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राष्ट्रीय भवन निर्माण मानक 2026

NATIONAL BUILDING CONSTRUCTION STANDARDS 2026

खण्ड 1 / VOLUME 1

भारतीय मानक ब्यूरो
BUREAU OF INDIAN STANDARDS

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FOREWORD

The Planning Commission, soon after the Third Five Year Plan, decided that the whole gamut of operations involved in construction, such as administrative, organizational, financial and technical aspects, be studied in depth. For this study, a Panel of Experts was appointed in 1965 by the Planning Commission and its recommendations are available in the 'Report on Economies in Construction Costs', published in 1968.

This study resulted in a recommendation that a National Building Code (NBC) be prepared to unify the building regulations throughout the country for use by government departments, municipal bodies and other construction agencies. The then Indian Standards Institution (now Bureau of Indian Standards) was entrusted by the Planning Commission with the responsibility of preparation of the National Building Code. Accordingly, Bureau of Indian Standard (then ISI) brought out first NBC in 1970. The same was subsequently revised in 1983, 2005 and 2016.

The third revision of NBC was published in 2016. This was a result of large scale changes in the building construction activities, such as change in nature of occupancies with prevalence of high rises and mixed occupancies, greater dependence and complicated nature of building services, development of new/innovative construction materials and technologies, greater need for preservation of environment and recognition of need for planned management of existing buildings and built environment, resulting in a paradigm shift in building construction scenario. A comprehensive revision was therefore brought out to address all these aspects and also reflect the changes incorporated in various standards which were considerably utilized in the 2016 revision.

Since the last revision, many things have changed, including per capita land availability, growing infrastructure requirements, socio-economic conditions, and technological advancements, to name a few. Hence, in these changing times, the requirements for development have also changed. The faster economic growth that India needs at present is possible if the ongoing reforms help the infrastructure sector to operate efficiently and compete cost-effectively by utilizing the available land optimally and reducing excessive compliance burden.

Based on the current realities, socio-economic scenario existing in the country, and the requirement of rapid urbanization and income growth, the total building stock is projected to increase exponentially in the coming years. Further, construction and infrastructure development are poised to determine the speed and scale of progress across every sector. These enable mobility, clean water, energy, and urban growth, which all directly contribute and shape economic opportunity and quality of life.

Construction, being a state subject, the contribution of the state government and local authorities are significant in infrastructure growth of the country. However, the land resources are limited. Maximum utilization of land resources is of prime importance in the current scenario. Further, India has come a long way from the time of first publication of the building code to the present day status. The requirements of standards and codes have also changed from a prescriptive regime, where the States and Local Authorities required hand holding, to now a more performance oriented outlook giving ample scope for innovation and decision making. The knowledge base in the country has also increased exponentially with exposure to the global best practices and adapting them to suit the country specific needs. Consequently, the contents of the current version, in which this publication has been renamed as the National Building Construction Standards (NBCS), have undergone substantial changes from those of the earlier NBC.

Thus in the current (fifth) version, this NBCS has been restructured substantially with a more safety and performance oriented approach rather than a prescriptive one. This includes leaving the adminstrative provisions land use requirements, like floor area ratio (FAR), development control rules (DCRs) and parking norms, which falls under the domain of local and state government, from the ambit of this NBCS. Open spaces, ground coverage, space utilization and land use related matters are primarily the state subject, and as such, these items have also not been included in this NBCS. As regards, other aspects covering construction practices, landscaping, sustainability aspects of built environment and asset and facility management, which are not directly linked to the design requirements vis-a-vis safety of the built environment have been taken out of this NBCS and will be published separately as guidelines of good practices mainly addressing management, sustainability, maintenance and practices to improve the working and efficiency of the built environment. The portions covering building materials, structural design aspects, building services and plumbing services are retained as part of this NBCS.

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The fire and life safety aspect, a state subject, but due to its importance towards the safety and well-being of the occupants as well as the common public, has also been included in this NBCS.

The current version of this NBCS with the restructuring is as follows:

a) The current version consists of

Foreword

Part A Introduction

Part B Building Materials

Part C Structural Design

Part D Building Services

Part E Plumbing Services

Part F Fire and Life Safety

Index of Terms

b) The other Parts of NBC 2016, namely Part 7 'Construction Management, Practices and Safety'; Part 10 'Landscape Development, Signs and Outdoor Display Structures'; Part 11 'Approach to Sustainability'; and Part 12 'Asset and Facility Management', have also been taken out of this NBCS. However, these will be brought out separately as special publication in the form of handbooks of good practices.

Towards making this NBCS more comprehensive, forward-looking and easy to implement, various technical aspects have been introduced apart from updating the existing provisions. The key points that have been addressed in this version of NBCS are:

1) Innovative building materials.
2) Review of blast-resistant design of buildings.
3) Detailed review of ground improvement techniques and inclusion of combined piled-raft foundations.
4) Inclusion of design parameters of new and imported timber species.
5) Construction methodologies using various bamboo species.
6) Inclusion of earthquake resistant systems of masonry.
7) Inclusion of the structural safety aspect of tall concrete buildings.
8) Updating the provisions on structural steel based design; and inclusion of a new section on composite construction.
9) Detailed provisions on glazing systems covering therein bullet-resistant glass and blast-resistant glass.
10) Charging infrastructure for electric vehicles.
11) Inclusion of provisions relating to renewable energy sources for buildings, such as rooftop solar photovoltaic (PV) systems, building-integrated photovoltaic (BIPV) systems, and solar water heating systems.
12) Inclusion of new section on power-driven parking systems for vehicles in buildings.
13) Focus on water-efficient fixtures in buildings; bio-digesters and packaged sewage treatment plants or decentralized STPs.

Notwithstanding the above, the repeated onslaught of natural calamities such as earthquakes, cyclones, landslides, floods, faced by the country has brought home the ever-increasing need for a common guiding document to enable monitoring and implementation of sound technical provisions and comprehensive features built into such a document. This NBCS fulfills this need apart from bringing at one place comprehensive provisions relating to selection of building materials, resilient design principles, various building services and plumbing services, etc, all ensuring to follow a unified approach thereby promoting resource efficiency, environmental responsibility and long-term resilience. This NBCS also captures the global best practices and technological developments in setting a robust roadmap towards establishing a more efficient mechanism promoting ease of doing business.

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As the regulation of land development and buildings is a State subject and is dealt by the States and local bodies such as urban (and rural) local bodies within their jurisdiction through building regulatory documents like building regulations, development control rules building byelaws, and fire regulations, this document is voluntary in nature and is non-binding. The implementation depends on adoption or adaptation by concerned parties or stipulations in a contract or by appropriate adoption by the concerned authorities.

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Panel for Building Materials, CED 46:P3

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Central Public Works Department, New Delhi	CHIEF ENGINEER, D&DM SE(C), D&DM (*Alternate*)
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CSIR-Central Building Research Institute, Roorkee	DR PRADEEP K. RAMANCHARLA DR AJAY CHOURASIA (*Alternate*)
CSIR-Structural Engineering Research Centre, Chennai	DR P. HARIKRISHNA DR KEERTHANA M. (*Alternate* I) DR J. PRAWIN (*Alternate* II)
Delhi Metro Rail Corporation, New Delhi	REPRESENTATIVE

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Organization	*Representative*(*s*)
Geological Survey of India, Kolkata	REPRESENTATIVE
Himachal Pradesh State Disaster Management Authority, Shimla	REPRESENTATIVE
India Meteorological Department, New Delhi	DR DEVENDRA PRADHAN SHRI K. N. MOHAN (*Alternate*)
Indian Association of Structural Engineers, New Delhi	SHRI MANOJ K. MITTAL PROF SHILPA PAL (*Alternate*)
Indian Geotechnical Society, New Delhi	DR A BOOMINATHAN DR G. R. DODAGOUDAR (*Alternate* I) DR DIVYAPRIYA BALASUBRAMANI (*Alternate* II)
Indian Institute of Technology Delhi, New Delhi	DR VASANT MATSAGAR DR DIPTI RANJAN SAHOO (*Alternate*)
Indian Institute of Technology Madras, Chennai	DR RUPEN GOSWAMI
Indian Institute of Technology Bombay, Mumbai	DR RAVI SINHA
Larsen & Toubro Ltd, Chennai	SHRI SURYA PRAKASH KARRI SHRI KRISHNA SOMARAJU (*Alternate* I) SHRI PRAVEEN KUMAR RAI (*Alternate* II)
National Council for Cement and Building Materials, Ballabgarh	SHRI P. N. OJHA DR BRIJESH SINGH (*Alternate*)
National Institute of Disaster Management, New Delhi	DR AMIR ALI KHAN
National Remote Sensing Centre, Hyderabad	REPRESENTATIVE
Odisha State Disaster Management Authority, Bhubaneswar	REPRESENTATIVE
The Institution of Engineers (India), Kolkata	DR ANIL JOSEPH MS SHILPI RANJAN (*Alternate*)
Uttarakhand State Disaster Management Authority, Dehradun	REPRESENTATIVE
Tandon Consultants Pvt Ltd, New Delhi	SHRI MAHESH TANDON SHRI VINAY GUPTA (*Alternate*)
In Personal Capacity (*B-03-3*, *Raheja Atlantis*, *NH 8*, *Sec 31-32A*, *Gurugram – 122001*)	DR PREM KRISHNA
In Personal Capacity (*Isa Vasyam*, *TC-15/1675*, *B-8/2-Lakshmi Nagar*, *Kesavadaspuram*, *Thiruvananthapuram – 695004*)	SHRI V. SURESH

Panel for Soils and Foundations, CED 46:P5

Organization	*Representative*(*s*)
In Personal Capacity (*473*, *Vinayak Apartments, BHEL Housing Society*, *Plot No. C58/19*, *Sector 62*, *Noida — 201301*)	SHRI C. PUSHPAKARAN **(*Convener*)**
AFCONS Infrastructure Limited, Mumbai	DR LAKSHMANA RAO MANTRI SHRI VIPIN PARIHAR (*Alternate* I) SHRI PRAJIT NAGARALE (*Alternate* II)

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Organization	*Representative*(*s*)
Bharat Heavy Electricals Ltd, Noida	SHRI T. MADHUSUDHAN RAO SHRI HITESH KUMAR (*Alternate*)
CSIR-Central Building Research Institute, Roorkee	SHRI MANOJIT SAMANTA DR ANINDYA PAIN (*Alternate* I) MS ASWATHY M. S. (*Alternate* II)
CENGRS Geotechnica Pvt Ltd, Noida	SHRI SANJAY GUPTA SHRI SORABH GUPTA (*Alternate*)
Central Public Works Department, New Delhi	SUPERINTENDING ENGINEER & PROJECT DIRECTOR, DPC-2 EE CUM SM (CIVIL)-1, DPC-2 (*Alternate*)
Creative Design Consultants & Engineers Pvt Ltd, Greater Noida	SHRI AMAN DEEP
Fluidyn India, Bengaluru	DR CHANDAN GHOSH
Indian Association of Structural Engineers, New Delhi	SHRI SUSHIL K. DHAWAN DR ABHAY GUPTA (*Alternate* I) DR ANIL JOSEPH (*Alternate* II)
Indian Geotechnical Society, New Delhi	DR A. P. SINGH DR NEELIMA SATYAM (*Alternate* I) DR ADITYA SINGH (*Alternate* II)
Indian Institute of Technology Delhi, New Delhi	DR J. T. SHAHU DR BAPPADITYA MANNA (*Alternate*)
Larsen & Toubro Limited, Chennai	DR G. D. RAJU SHRI VISAKAN R. (*Alternate*)
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	SHRI AJAY KUMAR SINGH SHRI AMAN GARG (*Alternate*)
NTPC Ltd, New Delhi	SHRI MANI BHUSHAN SHRI DEVENDER SINGH (*Alternate*)
National Capital Region Transport Corporation, New Delhi	SHRI JITENDRA KUMAR
National Council for Cement and Building Materials, Ballabgarh	SHRI P. N. OJHA SHRI AMIT TRIVEDI (*Alternate*)
National Institute of Disaster Management, New Delhi	DR AMIR ALI KHAN
The Institution of Engineers (India), Kolkata	ER VIJAY SHANKAR DR ALOK KUMAR (*Alternate*)
In Personal Capacity (*H-105*, *Casagrand Aristo*, *No. 5*, *Noble Street*, *Alandur*, *Chennai – 600016*)	SHRI A. VIJAYARAMAN

Panel for Timber and Bamboo, CED 46:P6

Organization	*Representative*(*s*)
In Personal Capacity [*Pratap Nursery Lane*, *(Near Gurudwara Sahib)*, *Panditwari*, *Dehradun – 248007*]	SHRI K. S. PRUTHI **(*Convener*)**
Bamboo Society of India, Bengaluru	SHRI PUNATI SRIDHAR SHRI K. N. MURTHY (*Alternate*)

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Organization	*Representative*(*s*)
Building Materials and Technology Promotion Council, New Delhi	DR SHAILESH KR AGRAWAL SHRI C. N. JHA (*Alternate*)
CEPT University, Ahmedabad	DR DHARA SHAH
CSIR-Central Building Research Institute, Roorkee	DR RAJESH KUMAR VERMA DR NEERAJ JAIN (*Alternate* I) SHRI PRASANTA KAR (*Alternate* II)
Central Public Works Department, New Delhi	SUPERINTENDING ENGINEER, DELHI CIRCLE-1 EXECUTIVE ENGINEER, L- DIV (*Alternate*)
Centre for Green Building Materials & Technology, Bengaluru	MS NEELAM MANJUNATH
Creative Design Consultants & Engineers Pvt Ltd, Greater Noida	SHRI AMAN DEEP
Forest Research Institute, Dehradun	SHRI RAJESH BHANDARI SHRI R. S. TOPWAL (*Alternate* I) SHRI ASHWATH HEGDE (*Alternate* II)
Forum of Scientists, Engineers and Technologists, Kolkata	SHRI RABI MUKHOPADHYAY DR PARTHASARATHI MUKHOPADHYAY (*Alternate*)
Housing and Urban Development Corporation Ltd, New Delhi	DR DEEPAK BANSAL MS POOJA NANDY (*Alternate* I) SHRIMATI NIRMAL BATRA (*Alternate* II)
Indian Association of Structural Engineers, New Delhi	DR DULAL GOLDAR DR VISALAKSHI TALAKOKULA (*Alternate*)
Indian Institute of Technology Delhi, New Delhi	DR SURESH BHALLA
ICFRE – Institute of Wood Science and Technology, Bengaluru	SHRI ANAND NANDANWAR SHRI AMITAVA SIL (*Alternate* I) DR NARASIMHAMURTHY (*Alternate* II)
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	SHRIMATI S. GAYATRI MUKHERJEE SHRI SHEKHAR GUPTA (*Alternate*)
Mutha Industries Pvt Ltd, Mumbai	SHRI ANIL MUTHA SHRI NEERAJ MUTHA (*Alternate* I) SHRI SUDIP CHAKRABORTY (*Alternate* II)
North East Centre for Technology Application and Reach, Shillong	SHRI KRISHNA KUMAR MS LYNGKSIAR KHONGWIR (*Alternate*)
The Institution of Engineers (India), Kolkata	SHRI MANOJ KUMAR SINGH ER KUMAR ARVIND (*Alternate*)
Windsor Wood (India) Pvt Ltd, Mumbai	SHRI PRAKASH NARMADASHANKAR SUTHAR
Wonder Grass Initiative Pvt Ltd, Nagpur	SHRI VAIBHAV KALEY
In Personal Capacity (*103/II*, *Vasant Vihar*, *P. O. New Forest*, *Dehradun – 248006*)	SHRI S. S. RAJPUT

Panel for Masonry, CED 46:P7

Organization	*Representative*(*s*)
In Personal Capacity (*131*, *Sector11D*, *Faridabad — 121006*)	SHRI V. V. ARORA **(*Convener*)**
Adlakha Associates Pvt Ltd, New Delhi	SHRI P. K. ADLAKHA SHRI V. K. SETHI (*Alternate*)

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Organization	*Representative*(*s*)
Association of Consulting Civil Engineers (India), Bengaluru	SHRI UMESH B. RAO DR RAGHUNATH S. (*Alternate*)
Bhabha Atomic Research Centre, Trombay	SHRI K. SRINIVAS SHRI H. E. IYER (*Alternate*)
Building Materials & Technology Promotion Council, New Delhi	DR SHAILESH KR AGRAWAL SHRI PANKAJ GUPTA (*Alternate*)
CSIR-Central Building Research Institute, Roorkee	DR AJAY CHOURASIA MS HINA GUPTA GHOSH (*Alternate*)
CSIR-Structural Engineering Research Centre, Chennai	DR A. RAMACHANDRA MURTHY DR S. R. BALASUBRAMANIAN (*Alternate* I) SHRI FARVAZE AHMED A. K. (*Alternate* II)
Central Public Works Department, New Delhi	SUPERINTENDING ENGINEER & PROJECT DIRECTOR, DPC-1 EE CUM SM (CIVIL)-1 (*Alternate*)
Indian Association of Structural Engineers, New Delhi	DR DEEPAK BANSAL MS SANGEETA WIJ (*Alternate*)
Indian Institute of Science, Bengaluru	DR B. V. VENKATARAMA REDDY DR K. S. NANJUNDA RAO (*Alternate*)
Indian Institute of Technology Delhi, New Delhi	DR DIPTI RANJAN SAHOO DR SRINIVAS MOGILI (*Alternate*)
Indian Institute of Technology Guwahati, Guwahati	DR HEMANT B. KAUSHIK
Indian Institute of Technology Hyderabad, Kandi	DR KOLLURU V. L. SUBRAMANIAM
Indian Institute of Technology Madras, Chennai	DR ARUN MENON
Indian Institute of Technology Patna, Patna	DR VAIBHAV SINGHAL
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	SHRI SAMEER PANDEY SHRI M. V. SHANKAR (*Alternate*)
National Council for Cement and Building Materials, Ballabgarh	SHRI AMIT TRIVEDI SHRI AMIT PRAKASH (*Alternate*)
The Institution of Engineers (India), Kolkata	SHRI SATYA PRAKASH SHRI MASARRAT NOOR KHAN (*Alternate*)

Panel for Plain, Reinforced and Prestressed Concrete, CED 46:P8

Organization	*Representative*(*s*)
In Personal Capacity (*Grace Villa*, *Kadamankulam PO*, *Thiruvalla – 689583*)	SHRI JOSE KURIAN **(*Convener*)**
Association of Consulting Civil Engineers (India), Bengaluru	SHRI M. S. SUDHARSHAN
Central Public Works Department, New Delhi	CHIEF ENGINEER (CENTRAL VISTA PZ-1) SE, CPVC-5 (*Alternate*)
Creative Design Consultants & Engineers Pvt Ltd, Greater Noida	SHRI AMAN DEEP
CSIR-Central Building Research Institute, Roorkee	DR S. K. SINGH SHRI SUBASH CHANDRA BOSE GURRAM (*Alternate* I) SHRI GOVIND GAURAV (*Alternate* II)
CSIR-Central Road Research Institute, New Delhi	REPRESENTATIVE

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Organization	*Representative*(*s*)
CSIR-Structural Engineering Research Centre, Chennai	DR B. H. BHARATKUMAR DR S. BHASKAR (*Alternate* I) SHRI THONDAMON V. (*Alternate* II)
Engineers India Limited, New Delhi	DR SUDIP PAUL MS RIYA DAS (*Alternate*)
Freyssinet Prestressed Concrete Company Limited, Mumbai	SHRI P. Y. MANJURE
Hindustan Construction Company Limited, Mumbai	SHRI SHAILESH THATTE
Indian Association of Structural Engineers, New Delhi	SHRI RAJEEV GOEL DR ABHAY GUPTA (*Alternate*)
Indian Concrete Institute, Chennai	SHRI A. K. JAIN SHRI K. P. ABRAHAM (*Alternate*)
Indian Institute of Technology Madras, Chennai	DR C. V. R. MURTY DR AMLAN K. SENGUPTA
Larsen & Toubro Limited, Chennai	SHRI V. SADASIVAM SHRI STHALADIPTI SAHA (*Alternate* I) SHRI J. RAJESH (*Alternate* II)
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	SHRI PRAMOD KUMAR SINGH SHRI NIRAJ KUMAR (*Alternate*)
Ministry of Road Transport and Highways, New Delhi	CHIEF ENGINEER, STANDARDS & RESEARCH (BRIDGES)
National Council for Cement and Building Materials, Ballabgarh	SHRI P. N. OJHA DR BRIJESH SINGH (*Alternate*)
Research, Designs and Standards Organization (Ministry of Railways), Lucknow	SHRI V. K. PANDEY SHRI SANDEEP SINGH (*Alternate*)
Tandon Consultants Pvt Ltd, New Delhi	SHRI MAHESH TANDON
The Institution of Engineers (India), Kolkata	SHRI MANOJ KUMAR SINGH DR M. A. CHAKRABARTI (*Alternate*)
VMS Consultants Private Limited, Mumbai	MS ALPA R. SHETH
In Personal Capacity (*131, Sector 11D, Faridabad – 121006*)	SHRI V. V. ARORA
In Personal Capacity (*M1F1 VGN Minerva Apartments*, *Guruswamy Road*, *Nolambur*, *Chennai – 600095*)	DR C. RAJKUMAR

Panel for Steel, CED 46:P9

Organization	*Representative*(*s*)
In Personal Capacity (*II*, *2A*, *Rani Meyyammai Towers*, *MRC Nagar*, *R A Puram*, *Chennai – 600028*)	DR V. KALYANARAMAN **(*Convener*)**
Association of Consulting Civil Engineers (India), Bengaluru	SHRI MANOJ KAWALKAR SHRI RAJKUMAR KACHARLA (*Alternate*)

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Organization	*Representative*(*s*)
Central Public Works Department, New Delhi	CHIEF ENGINEER & ED, (LLT DELHI) ZONE EE CUM SM (CIVIL)-1 (*Alternate*)
Creative Design Consultants & Engineers Pvt Ltd, Greater Noida	SHRI AMAN DEEP
CSIR-Central Building Research Institute, Roorkee	DR AJAY CHOURASIA DR R. SIVA CHIDAMBARAM (*Alternate* I) DR CHANCHAL SONKAR (*Alternate* II)
CSIR-Structural Engineering Research Centre, Chennai	DR G. S. PALANI DR NAPA PRASAD RAO (*Alternate* I) DR M. SARVANAN (*Alternate* II)
Engineers India Limited, New Delhi	SHRIMATI PAPIA MANDAL SHRI CHANDRA SHEKHAR SHARMA (*Alternate* I) SHRI SAPTADIP SARKAR (*Alternate* II)
Indian Association of Structural Engineers, New Delhi	DR HARSHAVARDHAN SUBBARAO DR ABHAY GUPTA (*Alternate*)
Indian Institute of Technology Madras, Chennai	DR S. ARUL JAYACHANDRAN DR P. S. LAKSHMI PRIYA (*Alternate*)
Institute for Steel Development and Growth, Kolkata	SHRI PYDI LAKSHMANA RAO SHRI ARIJIT GUHA (*Alternate* I) SHRI M. M. GHOSH (*Alternate* II)
Interarch Building Products Pvt Ltd, Noida	SHRI GAUTAM SURI SHRI NAVAZ MANIKKAL (*Alternate* I) SHRI SUNIL PULIKKAL (*Alternate* II)
Jindal Steel and Power Limited, New Delhi	SHRI SANJAY NANDANWAR SHRI BIJU MAHIMA (*Alternate*)
Kirby Building Systems India Limited, New Delhi	DR PADMAJA GOKARAJU SHRI VENUMADHAVA REDDY (*Alternate*)
Larsen & Toubro Limited, Chennai	SHRI T. VENKATESH RAO
MECON Limited, Ranchi	SHRI A. KRISHNA RAO SHRI C. KRISHNAM RAJU (*Alternate*)
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	SHRI M. V. SHANKAR BRIG Y. K. AHUJA (*Alternate*)
PEB Manufacturers' Association, Navi Mumbai	SHRI MANISH GARG
Research, Designs and Standards Organization (Ministry of Railways), Lucknow	SHRI RAJESH KUMAR SRIVASTAVA SHRI SRIJAN TRIPATHI (*Alternate*)
Tata Steel Ltd, Jamshedpur	SHRI HARIHARAPUTHIRAN H.
The Institution of Engineers (India), Kolkata	DR S. SENTHIL SELVAN DR P. R. KANNAN RAJKUMAR (*Alternate*)

Panel for Prefabrication and Systems Buildings, CED 46:P10

Organization	*Representative*(*s*)
Larsen & Toubro Ltd, Chennai	SHRI K. SENOU **(*Convener*)**

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Organization	*Representative*(*s*)
Adlakha Associates Pvt Ltd, New Delhi	SHRI P. K. ADLAKHA
Anna University, Chennai	DR K. P. JAYA
B.G. Shirke Construction Technology Pvt Ltd, Pune	SHRI YOGESH P. KAJALE COL SANJAY M. ADSAR (*Alternate*)
Building Materials and Technology Promotion Council, New Delhi	DR SHAILESH KUMAR AGRAWAL SHRI C. N. JHA (*Alternate*)
Central Public Works Department, New Delhi	CHIEF ENGINEER CUM ED, RPZ EE CUM SM (CIVIL)-1 (*Alternate*)
Creative Design Consultants & Engineers Pvt Ltd, Greater Noida	SHRI AMAN DEEP
CSIR-Central Building Research Institute, Roorkee	DR AJAY CHAURASIA SHRI ASHISH KAPOOR (*Alternate* I) SHRI SIDDHARTH BEHERA (*Alternate* II)
CSIR-Structural Engineering Research Centre, Chennai	DR J. PRABAKAR DR K. N. LAKSHMI KANDHAN (*Alternate* I) SHRI A. M. SARATH (*Alternate* II)
Indian Institute of Technology Hyderabad, Kandi	DR SURIYA PRAKASH S.
Indian Institute of Technology Madras, Chennai	DR RADHAKRISHNA G. PILLAI DR NIKHIL BUGALIA (*Alternate*)
Indian Association of Structural Engineers, New Delhi	SHRI MAHESH TANDON SHRI ALOK BHOWMICK (*Alternate*)
Innovela Building Solutions, Pune	SHRI SAURABH PURANDARE
Interarch Building Products Pvt Ltd, Noida	SHRI GAUTAM SURI SHRI MANISH KUMAR GARG (*Alternate*)
Kirby Building Systems India Ltd, Hyderabad	DR PADMAJA GOKARAJU SHRI G. VENUMADHAVA REDDY (*Alternate*)
Larsen & Toubro Ltd, Chennai	SHRI C. K. SOMARAJU SHRI AMIT BARDE (*Alternate* I) SHRI APARUP BISWAL (*Alternate* II)
Lloyd Insulations (India) Ltd, New Delhi	SHRI AJAY SINGH SHRI JITENDRA KUMAR JAIN (*Alternate*)
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	BRIG Y. K. AHUJA COL DEEPAK SINGH (*Alternate*)
National Council for Cement and Building Materials, Ballabgarh	SHRI AMIT TRIVEDI DR B. P. RANGA RAO (*Alternate*)
PEB Manufacturers' Association, Navi Mumbai	REPRESENTATIVE
Precast India, Pune	SHRI AJIT VASANT BHATE SHRI KAPILESH BHATE (*Alternate* I) DR K. SHANTHARAJU (*Alternate* II)
Precision Precast Solutions, Pune	SHRI DHANANJAY BHOSALE SHRI RAJENDRA R. KULKARNI (*Alternate*)
Tata Bluescope Steel Ltd, Pune	SHRI KRISHNAKANT RANE

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Organization	*Representative*(*s*)
The Indian Institute of Architects, Mumbai	SHRI CHAMARTHI RAJENDRA RAJU SHRI KURIAN GEORGE (*Alternate* I) SHRIMATI VIBHA SHRIVASTAVA (*Alternate* II)
The Institution of Engineers (India), Kolkata	DR R. K. BHANDARI SHRI HEMANT S. VADALKAR (*Alternate*)
TRC Engineering India Pvt Ltd, Bengaluru	SHRI ARJUN MUDAMBI
In Personal Capacity (*132*, *Marigold*, *Serene County*, *Telecom Nagar*, *Gachibowli – 500032*)	SHRI M. P. NAIDU
In Personal Capacity (*303*, *Tower 4*, *Lotus Panache*, *Sector 110*, *Noida – 201304*)	SHRI J. K. PRASAD

Panel for Lighting and Natural Ventilation, CED 46:P12

Organization	*Representative*(*s*)
CEPT University, Ahmedabad	DR RAJAN RAWAL **(*Convener*)**
AB Initio, Thane	SHRI SHIRISH DESHPANDE
Ashok B Lall Architects, New Delhi	SHRI ASHOK B. LALL MS GIRISHA SETHI (*Alternate*)
Bureau of Energy Efficiency, Ministry of Power, New Delhi	MS PRAVATANALINI SAMAL SHRI MAYANKRAJ PRAJAPAT (*Alternate*)
Central Public Works Department, New Delhi	CHIEF ENGINEER (T&R)-II SE (T)-ELECT (*Alternate*)
CEPT University, Ahmedabad	DR YASH SHUKLA DR MINU AGARWAL (*Alternate*)
CSIR-Central Building Research Institute, Roorkee	SHRI NAGESH BABU BALAM SHRI NAVEEN NISHANT (*Alternate* I) SHRI ANUP KUMAR PRASAD (*Alternate* II)
CSIR-National Physical Laboratory, New Delhi	DR PARAG SHARMA SHRI V. K. JAISWAL (*Alternate*)
Directorate General Factory Advice Service and Labour Institutes, Mumbai	SHRI VIPUL MISHRA SHRI SOURISH DATTA CHOUDHURY (*Alternate*)
Edspring Technologies, Noida	SHRI ANKUR KHANDELWAL
Indian Institute of Technology Delhi, New Delhi	DR DIBAKAR RAKSHIT
Indian Institute of Technology Madras, Chennai	DR BENNY RAPHAEL DR SHANMUGARAJ S. (*Alternate*)
Indian Institute of Technology Roorkee, Roorkee	DR E. RAJASEKAR
Indian Society for Lighting Engineers, New Delhi	DR PRAKASH BARJATIA DR GIRJA SHANKAR (*Alternate* I) DR S. M. ALI (*Alternate* II)
Kaleidoscope, Noida	SHRI AMOR KOOL
Larsen & Toubro Ltd (Sustainability Group), Chennai	DR S. RAJKUMAR
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	SHRI V. K. TIWARI SHRI H. K. GUPTA (*Alternate*)

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Organization	*Representative*(*s*)
Ministry of New and Renewable Energy, New Delhi	DR A. K. TRIPATHI SHRI H. C. BORAH (*Alternate*)
School of Planning and Architecture, New Delhi	DR V. K. PAUL DR ANIL DEWAN (*Alternate*)
Signify Innovations India Ltd, Hyderabad	SHRI NITISH POONIA SHRI SHRIKANT DATTATREYA PHANSE (*Alternate* I) MS MEGHA SONI (*Alternate* II)
Space Design Consultants, New Delhi	DR VINOD KUMAR GUPTA
The Indian Institute of Architects, Mumbai	SHRI GYANENDRA SINGH SHEKHAWAT MS RANEE VEDAMUTHU (*Alternate* I) SHRIMATI LEENA NIMBALKAR (*Alternate* II)
The Institution of Engineers (India), Kolkata	SHRI RAM NIWAS RAJPOOT
In Personal Capacity (*QRG Towers*, *2D*, *Sec - 126*, *Expressway Noida – 201304*)	SHRIMATI SUDHESHNA MUKHOPADHYAY

Panel for Electrical Installations, CED 46:P13

Organization	*Representative*(*s*)
In Personal Capacity (*C-20*, *Sector 47*, *Noida – 201301*)	SHRI N. NAGARAJAN **(*Convener*)**
Aeon Consultants, New Delhi	SHRI PUNEET GUPTA SHRI PRAVESH KUMAR PUNDIR (*Alternate*)
Bureau of Energy Efficiency, New Delhi	MS PRAVATANALINI SAMAL SHRI PANKAJ SHARMA (*Alternate*)
Cape Electric Pvt Ltd, Kanchipuram	SHRI S. GOPA KUMAR SHRI NANDU GOPAN (*Alternate*)
Central Electricity Authority, New Delhi	SHRI R. K. VERMA SHRI A. K. RAJPUT (*Alternate*)
Central Public Works Department, New Delhi	CHIEF ENGINEER CSQ (ELECT) SUPERINTENDING ENGINEER (ELECT), CSQ(E) (*Alternate*)
Chief Electrical Inspectorate, Govt of Tamil Nadu, Chennai	SHRI P. PALANI SHRI V. RAMAKRISHNAN (*Alternate*)
DEHN India Pvt Ltd, Gurugram	SHRI SAMIR SHANKAR SHRI MANU MUKUNDAN (*Alternate*)
Delhi Metro Rail Corporation, New Delhi	SHRI DEEPAK KUMAR GUPTA SHRI RAJEEV SAXENA (*Alternate*)
Energy Efficiency Services Limited, New Delhi	SHRI ASHISH SHARMA
Engineers India Ltd, New Delhi	SHRI HARISH KUMAR (*Alternate*)
Forum of Construction Utility Services, Bengaluru	SHRI MAXIM NORBET PINTO
International Copper Alliance, Mumbai	SHRI AMOL KALSEKAR SHRI MANAS KUNDU (*Alternate*)

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Organization	*Representative*(*s*)
Larsen & Toubro Ltd, Chennai	SHRI D. MAHESWARAN SHRI K. K. JEMBU KAILAS (*Alternate*)
Maple Engineering Design Services, Bengaluru	SHRI B. S. ASWATHNARAYAN SHRI PRASANNA KUMAR (*Alternate*)
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	SHRI VIMAL KUMAR TIWARI SHRI ARVIND KUMAR (*Alternate*)
Ministry of New and Renewable Energy, New Delhi	SHRI H. R. KHAN SHRI G. UPADHYAY (*Alternate*)
National Federation of Engineers for Electrical Safety, Chennai	SHRI S. APPAVOO SHRI ABHISHEK SINGH (*Alternate*)
OBO Bettermann, Chennai	DR K. JANAKI RAMAN SHRI ARUL PRAKASH (*Alternate*)
Proion Consultants Private Limited, New Delhi	SHRIMATI SHRUTI GOEL
Public Works Department (Electrical Division), Govt of Maharashtra, Mumbai	SHRI HEMANT SALI
Schneider Electric India Pvt Ltd, Gurugram	SHRI C. S. KORE SHRI DEBDAS GOSWAMI (*Alternate*)
Siemens Ltd, Chennai	SHRI VIJAY WANWE
Solar Energy Corporation of India, New Delhi	SHRI C. K. SINGH
Tata Motors Ltd, Mumbai	SHRI SENTHILNATHAN THANGAVELU
The Institution of Engineers (India), Kolkata	DR G. MADHUSUDANAN DR S. SENTHILKUMAR (*Alternate*)
In Personal Capacity (*Cedar 102, SJR Park Vista Aptts, Haralur Road, Ambalipura, Bengaluru – 560012*)	SHRI J. N. BHAVANI PRASAD
In Personal Capacity (*J-403*, *Second Floor*, *New Rajinder Nagar*, *New Delhi – 110060*)	SHRI CHAITANYA KUMAR VARMA

Panel for Air Conditioning, Heating and Mechanical Ventilation, CED 46:P14

Organization	*Representative*(*s*)
Indian Society of Heating, Refrigerating and Air Conditioning Engineers, New Delhi	SHRI ASHISH RAKHEJA **(*Convener*)**
AEON Integrated Building Design Consultants LLP, Noida	SHRI SAURABH GOYEL SHRI ANURAG JOSHI (*Alternate*)
ASHRAE India Chapter, New Delhi	SHRI INDRAJIT BHATTACHARYA
Blowtech Air Devices Pvt Ltd, Noida	SHRI SUSHIL K. CHOUDHURY
Blue Star Limited, Chennai	SHRI B. THIAGARAJAN SHRI SRINIVAS REDDY (*Alternate*)
Bureau of Energy Efficiency, New Delhi	SHRI SAURABH DIDDI SHRI PANKAJ SHARMA (*Alternate*)

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Organization	*Representative*(*s*)
Caire Consult, Greater Noida	SHRI ANKIT JAIN
Camfil India Pvt Ltd, Gurugram	SHRI ANIL CHOPRA
Carrier Air-conditioning and Refrigeration Ltd, Gurugram	SHRI BIMAL TANDON
CSIR-Central Building Research Institute, Roorkee	DR SHORAB JAIN DR TABISH ALAM (*Alternate* I) DR CHANDAN SWAROOP MEENA (*Alternate* II)
Central Public Works Department, New Delhi	CHIEF ENGINEER, CSQ (ELECT) SE (ELECT) CSQ (E) (*Alternate*)
CEPT University, Ahmedabad	DR RAJAN RAWAL DR YASH SHUKLA (*Alternate*)
Climaveneta Climate Technologies (P) Ltd, Bengaluru	SHRI ANIL DEV SHRI NANTHAGOPALAN T. K. (*Alternate* I) SHRI AKSHAY DEV (*Alternate* II)
Comfort Care System Pvt Ltd, New Delhi	SHRI YOGESH MALHOTRA
Daikin Air-conditioning India Pvt Ltd, Gurgram	SHRI KANWALJEET JAWA SHRI SANJAY GOYAL (*Alternate* I) SHRI GAURAV MEHTANI (*Alternate* II)
Delhi Fire Services, Govt of NCT of Delhi, New Delhi	DIRECTOR SHRI MUKESH VERMA (*Alternate*)
Delhi Metro Rail Corporation, New Delhi	SHRI ASHOK TEWARI SHRI RAJESH JAIN (*Alternate*)
Eskayem Consultants Pvt Ltd, Mumbai	SHRI GOUTHAM PITCHIKALA
Forum of Construction Utility Services, Bengaluru	SHRI K. D. SINGH SHRI RAMCHANDAR RAJU ADDHIPALLI (*Alternate* I) SHRI KAPIL KAPUR (*Alternate* II)
Indian Institute of Technology Delhi, New Delhi	DR MUKESH KHARE
Indian Society for Heating, Refrigeration & Air-Conditioning Engineers, New Delhi	SHRI K. RAMACHANDRAN SHRIMATI NIVEDITA JADHAV (*Alternate*)
International Ground Source Heat Pump Association, India Chapter, New Delhi	SHRI RICHIE MITTAL SHRI MADHUSUDHAN RAPOLE (*Alternate*)
Kanwal Industries Corporation, Noida	SHRI PAWANDEEP SINGH SHRI JAIVEER SINGH (*Alternate*)
Kirloskar Chillers Pvt Ltd, Pune	SHRI AVINASH MANJUL SHRI GAURANG DABHOLKAR (*Alternate* I) SHRI AMEY MAJGAONKAR (*Alternate* II)
Malaviya National Institute of Technology, Jaipur	DR JYOTIRMAY MATHUR
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	SHRI H. K. GUPTA SHRI ARVIND KUMAR (*Alternate*)
Proion Consultants Private Limited, New Delhi	SHRI SANDEEP GOEL
Schneider Electric India Pvt Ltd, Gurgram	SHRI HIMANSHU NIGAM

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Organization	*Representative*(*s*)
Shinryo Suvidha Engineers India Private Limited, Noida	SHRI ASHOK K. VIRMANI SHRI SANJAY MOOKERJEE (*Alternate*)
Sterling India Consulting Engineers, New Delhi	SHRI G. C. MODGIL MS KHUSHBOO MODGIL (*Alternate*)
The Institution of Engineers (India), Kolkata	SHRI PRADEEP CHATURVEDI SHRI ANIRBAN DATTA (*Alternate*)
The Chemours Company, Gurugram	DR AVINASH SHALIGRAM SHRI VIKAS MEHTA (*Alternate*)
UL India Pvt Ltd, Bengaluru	SHRI V. MANJUNATH SHRI SATISH KUMAR (*Alternate* I) SHRIMATI PALLAVI TRIPATHI (*Alternate* II)
Voltas Limited, New Delhi	SHRI ASHWANI SHARMA SHRI PANKAJ GOEL (*Alternate*)
In Personal Capacity (*C-9*, *9511*, *Vasant Kunj*, *New Delhi – 110070*)	DR R. S. AGARWAL
In Personal Capacity (*E 203*, *Belaire*, *Golf Course Road*, *Gurugram – 122022*)	SHRI PRABHAT GOEL
In Personal Capacity (*262*, *Solanipuram*, *Roorkee – 247667*)	DR P. K. BHARGAVA

Panel for Acoustics, Sound Insulation and Noise Control, CED 46:P15

Organization	*Representative*(*s*)
Interarch Building Products Private Limited, Noida	SHRI GAUTAM SURI **(*Convener*)**
All India Radio, New Delhi	MS ANURADHA AGGARWAL SHRI RAJESH KUMAR (*Alternate*)
Central Public Works Department, New Delhi	SENIOR ARCHITECT (ADG PROJECTS DELHI) ARCHITECT (ADG PROJECTS DELHI) (*Alternate*)
CSIR-Central Building Research Institute, Roorkee	SHRI SORAJ KUMAR PANIGARHI SHRI ANUP KUMAR PRASAD (*Alternate* I) SHRI KANTI LAL SOLANKI (*Alternate* II)
CSIR-National Physical Laboratory, New Delhi	DR NAVEEN GARG DR CHITRA GAUTAM (*Alternate*)
Indian Institute of Science, Bengaluru	DR M. L. MUNJAL
Indian Institute of Technology Kharagpur, Kharagpur	DR A. R. MOHANTY DR AMARNATH REDDY (*Alternate*)
Indian Institute of Technology Madras, Chennai	DR P. CHANDRAMOULI DR ABHIJIT SARKAR (*Alternate*)
Indian Institute of Technology Roorkee, Roorkee	DR E. RAJASEKAR

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Organization	*Representative*(*s*)
Military Engineer Services, E-in-C's Branch, Army HQ, New Delhi	SHRI SAMEER PANDEY SHRI NIRAJ KUMAR (*Alternate*)
P. S. Subramanian Associates, Chennai	DR S. KANDASWAMY DR P. SENTHIL KUMAR (*Alternate*)
School of Planning and Architecture, New Delhi	DR SHUVOJIT SARKAR
The Indian Institute of Architects, Mumbai	SHRI B. SUDHIR SHRI VIKAS DUBEY (*Alternate* I) SHRI SHITAL PRASHANT NEMANE (*Alternate* II)
The Institution of Engineers (India), Kolkata	SHRI P. K. ADLAKHA
In Personal Capacity (*H109/2*, *7th Avenue*, *Besant Nagar*, *Chennai – 600090*)	DR S. NARAYANAN

Panel for Installation of Lifts, Escalators and Moving Walks, CED 46:P16

Organization	*Representative*(*s*)
TAK Consulting Pvt Ltd, Mumbai	SHRI T. A. K. MATHEWS **(*Convener*)**
Central Public Works Department, New Delhi	CHIEF ENGINEER (T&R)-LL SE (T) - ELECT (*Alternate*)
Chief Electrical Inspectorate, Govt of Tamil Nadu, Chennai	SHRI G. JOSEPH AROCKIADOSS SHRI P. PALANI (*Alternate*)
Delhi Development Authority, New Delhi	SHRI SANJAY KUMAR KHARE SHRI K. P. YADAV (*Alternate*)
Delhi Fire Services, Govt of NCT of Delhi, New Delhi	DIRECTOR
Directorate of Maharashtra Fire & Emergency Services, Mumbai	SHRI SANTOSH WARICK SHRI KIRAN HATYAL (*Alternate*)
ECE Industries Limited, Ghaziabad	SHRI AMIT KACHROO SHRI ANURAG MANGLIK (*Alternate*)
Elevator and Escalator Component Manufacturing Association of India (EECMAI), Chennai	SHRI HARISH A. NAKMAN SHRI ABEY GEORGE (*Alternate*)
Electrical Inspectorate, Govt of NCT of Delhi, New Delhi	SHRI MUKESH KUMAR SHARMA SHRI JOGENDER SINGH (*Alternate*)
Forum of Construction Utility Services, Bengaluru	SHRI V. JAGADISH KUMAR
Fujitec India Pvt Ltd, Chennai	SHRI R. RAJESH SHRI S. MANOKAR (*Alternate*)
Indian Electrical and Electronics Manufacturers' Association (Elevator Division), Mumbai	SHRI P. K. DIXIT SHRI RUSHAD DIVECHA (*Alternate* I) SHRI UTTAM KUMAR (*Alternate* II)

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Organization	*Representative*(*s*)
Johnson Lifts Pvt Ltd, Chennai	SHRI T. SUBRAMANIAN SHRI V. KARTHIKEYAN (*Alternate* I) SHRI SACHIN MORE (*Alternate* II)
Kone Elevator India Limited, Chennai	SHRI U. VISWANATHAN SHRI BALAJI K. (*Alternate*)
Military Engineer Services, Engineer-in- Chief's Branch, Army HQ, New Delhi	SHRI H. K. GUPTA SHRI ARVIND KUMAR (*Alternate*)
NBCC (India) Ltd, New Delhi	SHIR RAM BACHAN SHRI TEJ PRAKASH DHABHAI (*Alternate*)
Otis Elevator Company (India) Limited, Bengaluru	SHRI ABHIJIT DANDEKAR SHRI SHRIHARI VISPUTE (*Alternate* I) SHRI PRAVEENA SIDDARAMANNA (*Alternate* II)
Public Works Department, Govt of Maharashtra, Mumbai	SHRI SANDEEP A. PATIL SHRI UDAY U. DAMBE (*Alternate*)
Schindler India Pvt Limited, Mumbai	SHRI NIMISH DESHPANDE SHRI NITIN VITHAL KADAM (*Alternate*)
TAK Consulting Pvt Ltd, Mumbai	SHRI GAURAV GHAG MS PRANALI PAWASKAR (*Alternate*)
TK Elevator India Pvt Ltd, New Delhi	SHRI VISHNU PARASHAR SHRI DEEPAK BALANI (*Alternate* I) SHRI SACHIN AGASHE (*Alternate* II)
The Indian Institute of Architects, New Delhi	SHRI PRAKASH DESHMUKH SHRI M. R. KUMARCHANDRA (*Alternate* I) MS ANAGHA VISHWAS KOTKAR (*Alternate* II)
The Institution of Engineers (India), Kolkata	DR R. K. DAVE
Wohr Parking Systems Pvt Ltd, Pune	SHRI SANDEEP KULKARNI SHRI VIKAS PATHAK (*Alternate*)
In Personal Capacity (*43*, *Juhu Gaurav*, *Gulmohar Cross Road No. 11*, *J.V.P.D. Scheme*, *Mumbai – 400049*)	SHRI P. M. TIPNIS

Panel for Plumbing Services, CED 46:P17

Organization	*Representative*(*s*)
Indian Plumbing Association, New Delhi	SHRI B. S. ASWATHNARAYAN **(*Convener*)**
Brihan Mumbai Licenced Plumbers' Association, Mumbai	SHRI BIJAL M. SHAH SHRI H. G. GANDHI (*Alternate*)
Central Ground Water Board, Faridabad	MEMBER (SML) REGIONAL DIRECTOR (*Alternate*)
Central Pollution Control Board, New Delhi	DR A. B. AKOLKAR DR SANJEEV AGRAWAL (*Alternate*)

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Organization	*Representative*(*s*)
Central Public Health and Environmental Engineering Organisation, New Delhi	DR RAMAKANT SHRI VIPIN KUMAR PATEL (*Alternate*)
Central Public Works Department, New Delhi	CHIEF ENGINEER (WORKS CUM TLQA) SE & PD, CIRCLE-1 (*Alternate*)
CSIR-Central Building Research Institute, Roorkee	DR KISHOR KULKARNI DR NAVEEN NISHANT (*Alternate* I) DR VEENA CHAUDHARY (*Alternate* II)
CSIR-National Environmental Engineering Research Institute, Nagpur	SHRI A. S. DUBEY SHRI M. KARTHIK (*Alternate*)
Delhi Development Authority, New Delhi	SHRI K. S. MEENA SHRI PANKAJ KUMAR VERMA (*Alternate*)
Delhi Jal Board, New Delhi	CHIEF ENGINEER (CENTRAL & NORTH)
Envirotech Instruments Pvt Ltd, New Delhi	SHRI S. K. GUPTA
Indian Plumbing Association, New Delhi	SHRI MUKESH ASIJA DR ANANTHA SIVA IYER (*Alternate*)
Indraprastha Gas Ltd, New Delhi	SHRI PANKAJ SHARMA SHRI AVIJIT NARAYAN (*Alternate*)
Mahanagar Gas Ltd, Mumbai	SHRIMATI PRATIBHA BIJAY
Maple Engineering Design Services (India) Pvt Ltd, Bengaluru	SHRI S. L. PRASANNA KUMAR
MKG Consultants, New Delhi	SHRI M. K. GUPTA SHRI NISHANT GUPTA (*Alternate* I) SHRI HIMANSHU PARIHAR (*Alternate* II)
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	SHRI H. K. GUPTA COL PANKAJ BANIYAL (*Alternate*)
Ministry of Jal Shakti, New Delhi	SHRI SAURABH GUPTA SHRI SANTOSH KUMAR (*Alternate*)
Municipal Corporation of Greater Mumbai, Mumbai	DY HYDRAULIC ENGINEER (PLANNING & CONTROL) EXECUTIVE ENGINEER (PLANNING & RESEARCH) (*Alternate*)
Nous Hospital Consultants Pvt Ltd, New Delhi	DR K. B. SOOD
Pratham Envirotech Solutions, Navi Mumbai	SHRI PURSHOTTAM YEMUL
Proion Consultants Private Limited, New Delhi	SHRI SANDEEP GOEL
Tata Consulting Engineers Limited, Mumbai	SHRI DHEERAJ KAUSHIK
The Indian Institute of Architects, New Delhi	SHRI UDAYA SHANKAR DONI SHRI RAJ KUMAR PRAJAPATI (*Alternate* I) MS SWOPNADUTTA MOHANTY (*Alternate* II)
The Institution of Engineers (India), Kolkata	DR D. ELANGO DR V. S. PRIYA (*Alternate*)
Water Management & Plumbing Skill Council, New Delhi	SHRI VINAY GUPTA SHRIMATI SHRUTI GOEL (*Alternate*)

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Organization	*Representative*(*s*)
In Personal Capacity (*Flat No 514*, *Charms Solitaire Apartment*, *Plot No 14*, *Mall Road*, *AK-2*, *Indirapuram*, *Ghaziabad – 201014*)	DR BHARAT BHUSHAN NAGAR

Panel for Glass and Glazing, CED 46:P20

Organization	*Representative*(*s*)
Central Public Works Department, New Delhi	SHRI PRADEEP GARG **(*Convener*)**
All India Glass Manufacturers' Federation, New Delhi	SHRI SHAILESH RANJAN SHRI RUPINDER SHELLY (*Alternate*)
Bureau of Energy Efficiency Ministry of Power, New Delhi	MS PRAVATANALINI SAMAL SHRI PANKAJ SHARMA (*Alternate*)
BES Consultants Pvt Ltd, Mumbai	SHRI RAJAN GOVIND SHRI VENUGOPAL B. (*Alternate*)
Central Public Works Department, New Delhi	SENIOR ARCHITECT (NATIONAL CPWD ACADEMY) ARCHITECT (NCA) (*Alternate*)
Creative Design Consultants & Engineers Pvt Ltd, Greater Noida	SHRI AMAN DEEP SHRI BARJINDER SINGH GHAI (*Alternate*)
CSIR-Central Building Research Institute, Roorkee	DR CHANDAN SWAROOP MEENA DR NAGESH BABU BALAM (*Alternate* I) DR KISHOR S. KULKARNI (*Alternate* II)
Dow Chemical International Private Limited, Mumbai	SHRI RAVISHANKAR SUBRAMANIAN
Facade India Testing Inc, Thane	SHRI V. S. RAVI
Glazing Society of India, Chennai	SHRI G. N. GOHUL DEEPAK MS DILNA SUBRAMANIAN (*Alternate* I) MS RANJANI. A (*Alternate* II)
GSC Glass Ltd, Greater Noida	SHRI C. J. SINGH
Hilti India Private Limited, New Delhi	SHRI SHOUNAK MITRA SHRI RAGHAVENDRA KUMAR VAKKALAGADDA (*Alternate*)
Indian Association of Structural Engineers, New Delhi	SHRI MAHESH TANDON DR S. P. ANCHURI (*Alternate*)
International Fenestration Forum, Noida	SHRI ARUN SHARMA
Indian Institute of Technology Madras (Structural Glass Research & Testing Facility), Chennai	DR S. ARUL JAYACHANDRAN DR ALAGAPPAN PONNALAGU (*Alternate*)
Indian Institute of Technology Roorkee, Roorkee	DR AVLOKITA AGRAWAL DR E. RAJASEKAR (*Alternate*)
Larsen & Toubro Ltd, Chennai	SHRI STHALADIPTI SAHA SHRI SURYA PRAKASH KARRI (*Alternate*)
Meinhardt Facade Technology India Pvt Ltd, Chennai	SHRI MAHESH ARUMUGAM
Municipal Corporation of Greater Mumbai (Mumbai Fire Brigade), Mumbai	CHIEF FIRE OFFICER DY CHIEF FIRE OFFICER (*Alternate*)

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Organization	*Representative*(*s*)
NBCC (India) Limited, New Delhi	SHRI ANUJ KUMAR GOYAL SHRI VINAY KUMAR KHULLAR (*Alternate*)
Procural Private Limited, Telangana	SHRI SATISH KUMAR SHRI SANTHANAM RENGASWAMY (*Alternate*)
Saint-Gobain Glass India Pvt Ltd, Chennai	SHRI SHOAIB SHAIKH SHRI U. GOPINATH (*Alternate* I) MS KAARNIKA M. (*Alternate* II)
Schueco India Pvt Ltd, Bengaluru	SHRI ANTONY JOHN V. S. SHRI KUSHAGRA AWASTHI (*Alternate*)
The Indian Institute of Architects, New Delhi	SHRI RUPINDER AHUJA SHRI ASHUTOSH KUMAR AGARWAL (*Alternate* I) SHRIMATI JYOTI RAJEEV PANSE (*Alternate* II)
The Institution of Engineers (India), Kolkata	ER M. NAGARAJ ER INDERDEEP SINGH OBEROI (*Alternate*)
UPVC Windows and Doors Manufacturers Association, New Delhi	SHRI ULLAS GULIANI SHRI MARIO SCHMIDT (*Alternate* I) SHRI SATISH KUMAR (*Alternate* II)
Winwall Technology India Pvt Ltd, Chennai	SHRI P. JOTHI RAMALINGAM

Panel for Information and Communication Enabled Installations, CED 46:P21

Organization	*Representative*(*s*)
In Personal Capacity (*C501*, *Shivam Apartments*, *Plot 14*, *Sector 12*, *Dwarka*, *New Delhi – 110078*)	SHRI A. K. MITTAL **(*Convener*)**
Broadband India Forum, New Delhi	SHRI T. V. RAMACHANDRAN BRIG ANIL TANDAN (*Alternate*)
Building Industry Consulting Service International, Mumbai	SHRI NINAD MOHAN DESAI
Cellular Operators Association of India, New Delhi	SHRI VIKRAM TIWATHIA SHRIMATI VERTIKA MISRA (*Alternate*)
Central Industrial Security Force, New Delhi	SHRI NARESH KUMAR SHRI SANJEEV KUMAR SADDI (*Alternate*)
Central Public Works Department, New Delhi	CHIEF ENGINEER (T&R)-LL SE (T)-ELECT (*Alternate*)
CMAI Association of India, New Delhi	DR N. K. GOYAL
Department of Telecommunication, New Delhi	SHRI ASHOK KUMAR SHRI RAJAT JAIN (*Alternate*)
Dept of Electronics & Information Technology, Ministry of Electronics and Information Technology, New Delhi	DR ANIL KUMAR KAUSHIK SHRI PRAKASH KUMAR (*Alternate*)
Digital Infrastructure Providers Association, New Delhi	SHRI MANOJ KUMAR SINGH SHRI RAMAKANT GUPTA (*Alternate* I) SHRI GREESH KUMAR (*Alternate* II)

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Organization	*Representative*(*s*)
Gujarat International Finance Tec City Company Limited, Gandhinagar	SHRI RAMAKANT JHA DR NILESH KUMAR PUREY (*Alternate*)
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	SHRI S. K. GUPTA SHRI BIJENDRA PAL SINGH (*Alternate*)
Ministry of Railways, New Delhi	SHRIMATI VINITA NARERA
Proion Consultants Private Limited, New Delhi	SHRIMATI SHRUTI GOEL
Roads and Buildings Department, Govt of Telangana, Hyderabad	SHRI G. RAMAKRISHNA KUMAR DEPUTY CHIEF ENGINEER (*Alternate*)
Shyam Spectra Private Limited, Gurugram	SHRI HEM CHAND RAJPUT
Sterlite Technologies Limited, Pune	SHRI S. N. GUPTA MD SHAHID KHAN (*Alternate*)
Telecom Equipment Manufacturers Association of India, New Delhi	SHRI BHUPESH YADAV SHRI RAHUL DUBEY (*Alternate*)
Telecom Engineering Centre, New Delhi	SHRI JASVEER SINGH PANESAR
The Indian Institute of Architects, Mumbai	DR JYOTI PANSE
The Institution of Engineers (India), Kolkata	DR LAXMI KANT MISHRA ER RAKESH KUMAR RATHORE (*Alternate*)
Town and Country Planning Organization, New Delhi	SHRI NARESH KUMAR DHIRAN SHRI PARESH KUMAR DURIA (*Alternate* I) SHRI R. SRINIVAS (*Alternate* II)
In Personal Capacity (*Cedar 102*, *SJR Park Vista Aptts*, *Haralur Road*, *Ambalipura*, *Bengaluru – 560012*)	SHRI J. N. BHAVANI PRASAD

WORKING GROUPS UNDER THE PANEL, CED 46:P2

Working Group for Smoke Management, CED 46:P2:WG1

Organization	*Representative*(*s*)
Directorate of Maharashtra Fire Services, Mumbai	SHRI SANTOSH S. WARICK **(*Convener*)**
Fire and Security Association of India, Chennai	SHRI SRINIVAS VALLURI
Indian Society of Heating, Refrigerating and Air Conditioning Engineers, New Delhi	SHRI K. RAMACHANDRAN

Working Group for Electrical Safety, CED 46:P2:WG2

Organization	*Representative*(*s*)
Fire and Security Association of India, Chennai	SHRI SRINIVAS VALLURI **(*Convener*)**
National Federation of Engineers for Electrical Safety, Chennai	SHRI S. GOPA KUMAR
In Personal Capacity (*27A*, *Tapovan Senior Citizens Foundation*, *Coimbatore – 641010*)	SHRI T. R. A. KRISHNAN

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Drafting Group of the Fire and Life Safety Chapter, CED 46:P2:WG3

Organization	*Representative*(*s*)
Fire Safe India Foundation, Mumbai	SHRI M. V. DESHMUKH **(*Convener*)**
Fire and Security Association of India, Chennai	SHRI SRINIVAS VALLURI
Kapse Consulting Engineers Private Limited, Thane	SHRI VINOD KAPSE
Directorate of Maharashtra Fire Services, Mumbai	SHRI SANTOSH S. WARICK SHRI KIRAN HATYAL (*Alternate*)
Proion Consultants Private Limited, New Delhi	SHRI SANDEEP GOEL
TTS Consultant, Kolkata	SHRI TARAK CHAKRABORTY
In Personal Capacity (*Shop No. 15*, *Natraj Building*, *Shiv Srishti Complex*, *M.G.Link Road Mulund* (*W*) *Mumbai – 400080*)	SHRI SHASHIKANT L. JADHAV
In Personal Capacity (*Flat No. 9221*, *ATS Pristine*, *Sector 150*, *Noida – 201 310*)	DR G. C. MISRA
In Personal Capacity (*Ganeshkunj A-10 Chanakyapuri*, *New Sama Road*, *Baroda – 390008*)	SHRI ABHAY D. PURANDARE

Advisory Group for Revision of Part 4, CED 46:P2:WG4

Organization	*Representative*(*s*)
In Personal Capacity (*K-33-A*, *Green Park*, (*First Floor*), *New Delhi – 110016*)	SHRI S. K. DHERI **(*Convener*)**
Delhi Fire Services, New Delhi	DIRECTOR
In Personal Capacity (*27A*, *Tapovan Senior Citizens Foundation*, *Coimbatore – 641010*)	SHRI T. R. A. KRISHNAN
In Personal Capacity (*D-317*, *IInd Floor*, *Nirman Vihar*, *New Delhi – 110 092*)	SHRI R. C. SHARMA
In Personal Capacity *('Isa Vasyam' TC-15/1675*, *B-8/2-Lakshmi Nagar*, *Kesavadaspuram – 695004*)	SHRI V. SURESH

Metro Related Provisions, CED 46:P2/WG5

Organization	*Representative*(*s*)
In Personal Capacity (*Flat No. 9221*, *ATS Pristine*, *Sector 150*, *Noida – 201310*)	DR G. C. MISRA **(*Convener*)**
Chennai Metro Rail Limited, Chennai	SHRIMATI RAJALAKSHMI. S
Delhi Metro Rail Corporation Limited, Delhi	SHRI ASHOK TEWARI SHRIMATI RASHMI BHARDWAJ (*Alternate* I) MS NABILA MARYAM (*Alternate* II)
National Capital Region Transport Corporation, New Delhi	SHRI GAUTAM BHARDWAJ
In Personal Capacity *(27A*, *Tapovan Senior Citizens Foundation*, *Coimbatore – 641010*)	SHRI T. R. A. KRISHNAN

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WORKING GROUP UNDER THE PANEL, CED 46:P3

Expert Group for Revision, CED 46:P3/WG1

Organization	*Representative*(*s*)
Central Public Works Department, New Delhi	SHRI DINESH K. UJJAINIA **(*Convener*)**
AEON Design & Development LLP, Noida	MS SHEETAL RAKHEJA
Builders' Association of India, Mumbai	REPRESENTATIVE
CSIR - Central Building Research Institute, Roorkee	DR NEERAJ JAIN
Indian Institute of Technology Madras, Chennai	DR AMLAN KUMAR SENGUPTA
Larsen and Toubro Construction, Chennai	SHRI PRAVEEN KUMAR RAI
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	SHRIMATI S. GAYATRI MUKHERJEE
National Council for Cement and Building Materials, Faridabad	SHRI P. N. OJHA
The Indian Institute of Architects, New Delhi	DR VENU SHREE

WORKING GROUPS UNDER THE PANEL, CED 46:P6

Working Group for Bamboo, CED 46:P6/WG1

Organization	*Representative*(*s*)
Wonder Grass Initiative Private Limited, Nagpur	SHRI VAIBHAV KALEY **(*Convener*)**
Bamboo Society of India, Bengaluru	SHRI PUNATI SRIDHAR
Centre for Green Building Materials & Technology, Bengaluru	MS NEELAM MANJUNATH
CSIR - Central Building Research Institute, Roorkee	ER MICKEY MECON DALBEHERA
Forest Research Institute, Dehradun	DR AJAY THAKUR

Working Group for Engineered Bamboo, CED 46:P6/WG2

Organization	*Representative*(*s*)
ICFRE-Institute of Wood Science and Technology, Bengaluru	SHRI ANAND NANDANWAR **(*Convener*)**
CEPT University, Ahmedabad	DR DHARA SHAH
Forest Research Institute, Dehradun	DR AJAY THAKUR
Indian Institute of Technology Delhi, New Delhi	SHRI SURESH BHALLA
ICFRE-Institute of Wood Science and Technology, Bengaluru	DR NARASIMHAMURTHY
Mutha Industries Pvt Ltd, Mumbai	SHRI ANIL MUTHA
North East Centre for Technology Application and Research, Shillong	SHRI KRISHNA KUMAR

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Working Group for Timber, CED 46:P6/WG3

Organization	*Representative*(*s*)
Forestry Innovation Consulting India Pvt Ltd, Mumbai	DR JIMMY THOMAS
Forest Research Institute, Dehradun	SHRI RAJESH BHANDARI
Forum of Scientists, Engineers and Technologists, Kolkata	DR PARTHASARATHI MUKHOPADHYAY
ICFRE-Institute of Wood Science and Technology, Bengaluru	SHRI ANAND NANDANWAR SHRI AMITAVA SIL
In Personal Capacity (*P.O. New Forest*, *Dehradun – 248006*)	DR SADHNA TRIPATHI

Working Group for Engineered Timber, CED 46:P6/WG4

Organization	*Representative*(*s*)
ICFRE-Institute of Wood Science and Technology, Bengaluru	SHRI ANAND NANDANWAR **(*Convener*)**
CEPT University, Ahmedabad	DR DHARA SHAH
Forest Research Institute, Dehradun	SHRI RAJESH BHANDARI
North East Centre for Technology Application and Research, Shillong	SHRI KRISHNA KUMAR
Windsor Wood (India) Pvt Ltd, Mumbai	SHRI PRAKASH NARMADASHANKAR SUTHAR

WORKING GROUPS UNDER THE PANEL, CED 46:P12

Working Group for Building Orientation, CED 46:P12/WG1

Organization	*Representative*(*s*)
Ashok B Lall Architects, New Delhi	SHRI ASHOK B. LALL (***Convener***)
Ministry of New and Renewable Energy, New Delhi	DR A. K. TRIPATHI

Working Group for Building Lightning, CED 46:P12/WG2

Organization	*Representative*(*s*)
CEPT University, Ahmedabad	DR MINU AGARWAL **(*Convener*)**
CSIR-Central Building Research Institute, Roorkee	SHRI ANUP KUMAR PRASAD

Working Group for Ventilation, CED 46:P12/WG3

Organization	*Representative*(*s*)
CEPT University, Ahmedabad	DR RAJAN RAWAL **(*Convener*)**
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	SHRI ASHOK KUMAR

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WORKING GROUPS UNDER THE PANEL, CED 46:P13

Working Group for Electric Vehicles, Battery Storage, and Load Factor, CED 46:P13/WG2

Organization	*Representative*(*s*)
In Personal Capacity (*Cedar 102*, *SJR Park Vista Aptts*, *Haralur Road*, *Ambalipura*, *Bengaluru – 560012*)	SHRI J. N. BHAVANI PRASAD **(*Convener*)**
Automotive Research Association of India, Pune	SHRI A. A. DESHPANDE
Cape Electric Private Limited, Kancheepuram	SHRI S. GOPA KUMAR
Central Public Works Department, New Delhi	SHRI VIKAS RANA
Directorate of Maharashtra Fire Services, Mumbai	SHRI SANTOSH S. WARICK
India Energy Storage Alliance, Pune	SHRI DEBI PRASAD DASH
Indian Institute of Technology Madras, Chennai	DR ASHOK JHUNJHUNWALA
Tata Motors Limited, Pune	SHRI SENTHILNATHAN THANGAVELU
In Personal Capacity (*J-403 Second Floor*, *New Rajinder Nagar*, *New Delhi – 110060*)	SHRI CHAITANYA KUMAR VARMA

Working Group for Revision of the Chapter, CED 46:P13/WG3

Organization	*Representative*(*s*)
Cape Electric Private Limited, Kancheepuram	SHRI S. GOPA KUMAR **(*Convener*)**
Aeon Integrated Building Design Consultants, Noida	SHRI PUNEET GUPTA
Central Electricity Authority, New Delhi	SHRI ASHOK KUMAR RAJPUT
Central Public Works Department, New Delhi	SHRI VIKAS RANA
Government of Tamil Nadu, Chief Electrical Inspectorate, Chennai	SHRI P. PALANI
Ministry of New and Renewable Energy, New Delhi	SHRI A. K. TRIPATHI
National Federation of Engineers for Electrical Safety, Chennai	SHRI ABHISHEK SINGH
Obo Bettermann India Private Limited, Oragadam	SHRI K. JANAKI RAMAN
ProtegoPlus Electrotech Private Limited, Mumbai	SHRI KRISH THEOBALD
Proion Consultants Private Limited, New Delhi	SHRIMATI SHRUTI GOEL
Public Works Department, Government of Maharashtra, Mumbai	SHRI HEMANT M. SALI
Schneider Electric India Private Limited, Gurugram	SHRI DEBDAS GOSWAMI

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Working Group for Solar Energy and Power from Multiple Sources and Related Protection and Earthing, CED 46:P13/WG4

Organization	*Representative*(*s*)
Proion Consultants Private Limited, New Delhi	SHRIMATI SHRUTI GOEL **(*Convener*)**
Cape Electric Private Limited, Kancheepuram	SHRI S. GOPA KUMAR
Central Public Works Department, New Delhi	SHRI VIKAS RANA
Government of Tamil Nadu, Chief Electrical Inspectorate, Chennai	SHRI P. PALANI
Ministry of New and Renewable Energy, New Delhi	DR A. K. TRIPATHI
Obo Bettermann India Private Limited, Oragadam	SHRI K. JANAKI RAMAN
Public Works Department, Government of Maharashtra, Mumbai	SHRI HEMANT M SALI
Solar Energy Corporation of India Limited, New Delhi	SHRI C. K. SINGH
The Institution of Engineers (India), Kolkata	DR S. SENTHILKUMAR

WORKING GROUP UNDER THE PANEL, CED 46:P14

Core Group for Revision of HVAC Chapter, CED 46:P14/WG1

Organization	*Representative*(*s*)
ACR Project Consultants Private Limited, Pune	SHRI HARSHAL SURANGE
Adani Energy Solution Pvt Ltd, Ahmedabad	SHRI PRAMOD PRABHAKAR
Anna University, Chennai	DR R. SARAVANAN
Blowtech Air Devices Private Limited, Noida	SHRI SUSHIL K. CHOUDHURY
Breathe Easy Labs, New Delhi	SHRI BARUN AGARWAL
CBRE India, Gurugram	SHRI ANEESH KADYAN
CEPT University, Ahmedabad	DR YASH SHUKLA
Degree Day Private Limited, Indore	SHRI NISHANT GUPTA
Dynacraft Air Controls, Mumbai	SHRI V. KRISHNAN
ELM Enterprises Private Limited, Delhi	SHRI PRASHANT DESAI
Enova HVAC & MEP Consulting, Mumbai	MS SANGITA JHANGIANI
Eskayem Consultants Private Limited, Mumbai	SHRI GOUTHAM PITCHIKALA
Envirocon Engineering Services, New Delhi	SHRIMATI SAMTA BAJAJ

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Organization	*Representative*(*s*)
Genex Consultants, Hyderabad	SHRI CHANDRASEKAR NARAYANAN
Godrej & Boyce Manufacturing Company Limited, Mumbai	SHRI MUKESH SUTHAR
Impec Filters Private Limited, Chennai	DR SHANKAR RAJASEKARAN
Indian Society of Heating, Refrigerating and Air Conditioning Engineers, New Delhi	SHRI ASHISH RAKHEJA SHRI K. RAMACHANDRAN SHRIMATI NIVEDITA JADHAV
International Ground Source Heat Pump Association, New Delhi	SHRI MADHUSUDAN RAPOLE
Lead Consultancy and Engineering Services (India) Private Limited, Bengaluru	SHRI M. SELAVARASU
Malaviya National Institute of Technology, Jaipur	DR JYOTIRMAY MATHUR
Overdrive Engineering Private Limited, New Delhi	SHRI RICHIE MITTAL
Paharpur Cooling Towers Limited, Kolkata	SHRI VARUN SWARUP
Proion Consultants Private Limited, New Delhi	SHRI SANDEEP GOEL
Reliance Industries Limited, Bengaluru	SHRI AJAJ KAZI
3S Green SSS Consultants, Bengaluru	SHRI C. SUBRAMANIAM
Sanelac Consultants Private Limited, New Delhi	SHRI VARUN JAIN
Sterling India Consulting Engineers, New Delhi	SHRI G. C. MODGIL
The Chemours India Private Limited, Gurugram	SHRI VIKAS MEHTA
UL India Private Limited, Bengaluru	SHRI V. MANJUNATH
Univac Environment Systems Private Limited, Mumbai	SHRI VIKRAM MURTHY
Vision Electro Mechanical Consultants Private Limited, Pune	DR NITIN DEODHAR
In Personal Capacity (*E-203, Belaire, Golf Course Road, Gurugram – 122022*)	SHRI PRABHAT GOEL
In Personal Capacity (*Experiqs Pvt Ltd*, *7022*, *RBTIC Building*, *IIT Bombay*, *Powai*, *Mumbai – 400076*)	SHRI AVINASH G. SHALIGRAM

WORKING GROUP UNDER THE PANEL, CED 46:P15

Expert Group for Revision, CED 46:P15/WG1

Organization	*Representative*(*s*)
Interarch Building Products Private Limited, Noida	SHRI GAUTAM SURI **(*Convener*)**
CSIR-National Physical Laboratory, New Delhi	DR NAVEEN GARG

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Organization	*Representative*(*s*)
Indian Institute of Science, Bengaluru	DR M. L. MUNJAL
Indian Institute of Technology Madras, Chennai	DR S. NARAYANAN
Indian Institute of Technology Roorkee, Roorkee	DR E. RAJASEKAR
P. S. Subramanian Associates, Chennai	DR S. KANDASWAMY
The Indian Institute of Architects, New Delhi	SHRI B. SUDHIR

WORKING GROUP UNDER THE PANEL, CED 46:P16

Working Group for Revision of the Chapter, CED 46:P16/WG1

Organization	*Representative*(*s*)
TAK Consulting Private Limited, Mumbai	SHRI T. A. K. MATHEWS **(*Convener*)**
Directorate of Maharashtra Fire Services, Mumbai	SHRI SANTOSH S. WARICK
ECE Industries Limited, New Delhi	SHRI AMIT KACHROO SHRI ANURAG MANGLIK (*Alternate*)
Elevator and Escalator Component Manufacturers' Association of India, Chennai	SHRI HARISH A. NAKMAN SHRI ABEY GEORGE (*Alternate*)
Energy Department, Government of Maharashtra, Mumbai	SHRI SANDEEP A. PATIL SHRI UDAY U. DAMBE (*Alternate*)
Fujitec India Private Limited, Gurugram	SHRI R. RAJESH SHRI MANOKAR S. (*Alternate* I) SHRI ARUN SANKAR (*Alternate* II)
IEEMA Elevator Division, Mumbai	SHRI P. K. DIXIT SHRI RUSHAD DIVECHA (*Alternate* I) SHRI UTTAM KUMAR (*Alternate* II)
Johnson Lifts Pvt Limited, Chennai	SHRI V. KARTHIKEYAN SHRI SACHIN MORE (*Alternate*)
Kone Elevator India Private Limited, Chennai	SHRI U. VISWANATHAN SHRI BALAJI K. (*Alternate*)
Otis Elevator Company (India) Limited, Bengaluru	SHRI SHRIHARI VISPUTE SHRI ABHIJIT DANDEKAR (*Alternate* I) SHRI PRAVEENA SIDDARAMANNA (*Alternate* II)
Schindler India Private Limited, Mumbai	SHRI NIMISH DESHPANDE SHRI RAJAGOPALAN RENGANATHAN (*Alternate* I) SHRI NITIN VITHAL KADAM (*Alternate* II)
TAK Consulting Private Limited, Mumbai	SHRI GAURAV GHAG
TK Elevator India Private Limited, New Delhi	SHRI VISHNU PARASHAR SHRI DEEPAK BALANI (*Alternate*)
Wohr Parking Systems Pvt Ltd, Pune	SHRI SANDEEP KULKARNI

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Organization	*Representative*(*s*)
In Personal Capacity (*43*, *Juhu Gaurav*, *Gulmohar Cross Road No. 11*, *J.V.P.D. Scheme*, *Mumbai – 400 049*)	SHRI P. M. TIPNIS

WORKING GROUPS UNDER THE PANEL, CED 46:P17

Working Group for Water Supply, CED 46:P17/WG1

Organization	*Representative*(*s*)
Indian Plumbing Association, New Delhi	SHRI B. S. ASWATHNARAYAN **(*Convener*)**
CSIR-Central Building Research Institute, Roorkee	SHRI CHANDAN SWAROOP MEENA
D' Plumbing Consultants, Mumbai	SHRI DEEPAK
Indian Plumbing Association Maple Engineering - Design Services India Private Limited, Bengaluru	SHRI S. L. PRASANNAKUMAR
Indian Plumbing Association, New Delhi	SHRI MUKESH ASIJA
Ministry of Heavy Industries and Public Enterprises, New Delhi	SHRI VIJAY PRAKASH
Municipal Corporation of Greater Mumbai, Mumbai	DR DEEPAK NAGARSOGE
Nous Hospital Consultants Pvt Ltd, New Delhi	DR K. B. SOOD
Proion Consultants Private Limited, New Delhi	SHRI SANDEEP GOEL
Tata Consulting Engineers Limited, Navi Mumbai	SHRI DHEERAJ KAUSHIK

Working Group for Drainage and Sanitation, CED 46: P17/WG2

Organization	*Representative*(*s*)
MKG Consultants, New Delhi	SHRI M. K. GUPTA **(*Convener*)**
Brihan Mumbai Licensed Plumbers' Association, Mumbai	SHRI H. G. GANDHI
Central Public Works Department, New Delhi	SHRI DINESH K. UJJAINIA
Maple Engineering Design Services (India) Private Limited, Bengaluru	SHRI S. L. PRASANNA KUMAR
Military Engineer Services, Engineer-in-Chief's Branch, Army HQ, New Delhi	SHRI N. K. GOEL
The Indian Institute of Architects, New Delhi	SHRI RAJ KUMAR PRAJAPATI
In Personal Capacity (*610*, *Technology Apartments*, *24*, *Patparganj*, *New Delhi – 110002*)	SHRI SUBIR PAUL

Working Group for Solid Waste Management, CED 46: P17/WG3

Organization\	*Representative*(*s*)
Central Public Health Environmental Engineering Organisation, New Delhi	DR RAMAKANT **(*Convener*)**

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Organization\	*Representative*(*s*)
CSIR-Central Building Research Institute, Roorkee	DR AJAY CHOURASIA
Central Pollution Control Board, New Delhi	DR A. B. AKOLKAR DR SANJEEV AGRAWAL (*Alternate*)
Indian Institute of Technology Bombay, Mumbai	DR HARIHARAN S. SHANKAR
The Institution of Engineers (India), Kolkata	SHRI D. ELANGO
In Personal Capacity (*377*, *Church View Apartment*, *Sector 29*, *Noida – 201303*)	CAPT MANJU MINHAS

Working Group for Gas Supply, CED 46:P17/WG4

Organization	*Representative*(*s*)
Indian Plumbing Association, New Delhi	SHRI B. S. ASWATHNARAYAN **(*Convener*)**
Indraprastha Gas Limited, New Delhi	SHRI PANKAJ SHARMA
CSIR-National Environmental Engineering Research Institute, Nagpur	SHRI A. S. DUBEY
Ministry of Heavy Industries and Public Enterprises, New Delhi	SHRI VIJAY PRAKASH
Tata Consulting Engineers Limited, Navi Mumbai	SHRI GANESH KUMAR SINGH

WORKING GROUPS UNDER THE PANEL, CED 46:P20

Working Group for Energy and Thermal Performance, CED 46:P20/WG1

Organization	*Representative*(*s*)
Indian Institute of Technology Roorkee, Roorkee	DR E. RAJASEKAR **(*Convener*)**
Bureau of Energy Efficiency, New Delhi	SHRI SAURABH DIDDI
Glazing Society of India, Chennai	SHRI GOHUL DEEPAK G. N.
International Fenestration Forum, New Delhi	SHRI ARUN SHARMA
Larsen & Toubro Ltd, Chennai	SHRI SURYA PRAKASH KARRI
Schueco India Private Limited, Bengaluru	SHRI KUSHAGRA AWASTHI

Working Group for Special Glass, CED 46:P20/WG2

Organization	*Representative*(*s*)
Meinhardt Facade Technology India Private Limited, Chennai	SHRI MAHESH ARUMUGAM **(*Convener*)**
Central Public Works Department, New Delhi	SHRI PRADEEP GARG

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Organization	*Representative*(*s*)
Defence Research and Development Organisation, Terminal Ballistics Research Laboratory, Chandigarh	SHRIMATI SAKSHI ARORA DR PREETI JAIN (*Alternate*)
Glazing Society of India, Chennai	SHRI GOHUL DEEPAK G. N.
International Fenestration Forum, New Delhi	SHRI ARUN SHARMA
Larsen & Toubro Ltd, Chennai	SHRI SURYA PRAKASH KARRI
Saint-Gobain Glass India Private Limited, Chennai	SHRI AVINASH GOWRA
Schueco India Private Limited, Bengaluru	SHRI ANTONY JOHN V. S.

Working Group for Fire Safety Aspects, CED 46:P20/WG3

Organization	*Representative*(*s*)
Indian Institute of Technology Gandhinagar, Gandhinagar	DR GAURAV SRIVASTAVA **(*Convener*)**
Aadit Fire Testing Laboratory Private Limited, Gummidipoondi	SHRI P. JOTHI RAMALINGAM DR MANIGANDAN SHRI AKHIL CHACKO
All India Glass Manufacturers' Federation, New Delhi	SHRI NAGENDRA KUMAR
BES Consultants Private Limited, Mumbai	SHRI RAJAN GOVIND
CSIR-Central Building Research Institute, Roorkee	SHRI S. K. NEGI
Glazing Society of India, Chennai	SHRI GOHUL DEEPAK G. N.
Hilti India Private Limited, New Delhi	SHRI RAGHAVENDRA KUMAR VAKKALAGADDA
Indian Institute of Technology Tirupati, Yerpedu	DR JASHNAV PANCHETI

Working Group for Structural Design Aspects, CED 46:P20/WG4

Organization	*Representative*(*s*)
BES Consultants Private Limited, Mumbai	SHRI RAJAN GOVIND **(*Convener*)**
Glazing Society of India, Chennai	SHRI GOHUL DEEPAK G. N.
Hilti India Private Limited, New Delhi	SHRI SHOUNAK MITRA
Indian Association of Structural Engineers, New Delhi	SHRI MAHESH TANDON
Indian Institute of Technology Madras, Chennai	DR ALAGAPPAN PONNALAGU
Larsen & Toubro Ltd, Chennai	SHRI SURYA PRAKASH KARRI
Schueco India Private Limited, Bengaluru	SHRI ANTONY JOHN V. S.

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Working Group for Performance and Testing, CED 46:P20/WG5

Organization	*Representative*(*s*)
Schueco India Private Limited, Bengaluru	SHRI ANTONY JOHN V. S. **(*Convener*)**
Eminent International Testing Centre Private Limited, Hyderabad	SHRI REJI BHAMI
Facade India Testing Inc, Thane	SHRI V. S. RAVI
Glazing Society of India, Chennai	SHRI GOHUL DEEPAK G. N.
Winwall Technology India Private Limited, Chennai	SHRI P. JOTHI RAMALINGAM

WORKING GROUPS UNDER THE PANEL, CED 46:P21

Working Group for Indoor and Outdoor Plants, CED 46:P21/WG1

Organization	*Representative*(*s*)
ITS India Forum, New Delhi	DR SHIV KUMAR **(*Convener*)**
Broadband India Forum, New Delhi	SHRI ATUL JAIN
Building Industry Consulting Service International, Mumbai	SHRI NINAD MOHAN DESAI
Ranext Technologies Private Limited, Gurugram	SHRI JAGMOHAN SINGH
Sterlite Technologies Limited, Pune	SHRI SHAHID KHAN
Telecom Regulatory Authority of India, New Delhi	SHRI VIRENDRA SINGH RAJPUT

Working Group for Wireless Systems, CED 46:P21/WG2

Organization	*Representative*(*s*)
In Personal Capacity (*C501*, *Shivam Apartments*, *Plot 14*, *Sector 12*, *Dwarka*, *New Delhi – 110078*)	SHRI A. K. MITTAL **(*Convener*)**
Broadband India Forum, New Delhi	SHRI DEBASHISH BHATTACHARYA
Cellular Operators Association of India, New Delhi	SHRI VIKRAM TIWATHIA SHRI DHANANJAY GAWANDE
Crest Digitel Private Limited, Gurugram	SHRI VINAY KUMAR
Digital Infrastructure Providers Association, New Delhi	SHRI MANOJ KUMAR SINGH
Telecom Engineering Centre, New Delhi	SHRI JASVEER SINGH PANESAR
Telecom Regulatory Authority of India, New Delhi	SHRI TEJPAL SINGH

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART A INTRODUCTION

BUREAU OF INDIAN STANDARDS

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART A INTRODUCTION

This Part covers guidelines to be followed for judicious implementation of the provisions of various Parts/Sections of the National Building Construction Standards (NBCS), following a systematic approach.

In order to provide safe and resilient habitat, careful consideration needs to be paid to the building construction activity. Building planning, designing and construction activities have developed over the centuries. Large number of ancient monuments and historical buildings all over the world bear testimony to the growth of civilization from the prehistoric era with the extensive use of manual labour and simple systems as appropriate to those ages to the present day mechanized and electronically controlled operations for designing and constructing buildings and for operating and maintaining systems and services. Technological and socio-economic developments in recent times have led to remarkable increase in demand for more and more sophistication in buildings resulting in ever increasing complexities. For achieving these performance demand high levels of inputs from professionals of different disciplines such as architecture, civil engineering, structural engineering, functional and life safety services including special aspects relating to utilities, landscaping in conceptualization, spatial planning, design and construction of buildings of various material and technology streams, and various services including operation, maintenance, repairs and rehabilitation aspects throughout the service life of the building.

Present day construction scenario enables freedom of action to adopt appropriate practices and provides for building planning, designing and construction for absorbing traditional practices as well as latest developments in knowledge in the various disciplines as relevant to a building including computer aided and/or other modern sensors aided activities in the various stages of conceptualization, planning, designing, constructing, maintaining and repairing the buildings. India being a large country with substantial variations from region to region, this document has endeavoured to meet the requirements of different regions of the country, both urban and rural, by taking into consideration factors such as climatic and environmental conditions, geographical terrain, vulnerability to natural disasters, ecologically appropriate practices, use of eco-friendly materials, use of appropriate conventional and alternative technologies, and also socio-economic considerations, towards the creation of sustainable human settlements and built environment.

Buildings can be classified based on occupancy and types of construction as below:

Occupancy classification

a) Residential;

b) Educational;

c) Institutional;

d) Assembly;

e) Business;

f) Mercantile;

g) Industrial;

h) Storage; and

j) Hazardous.

Types of construction — Based on fire resistance rating for different set of components (suitably covered under Part F — Fire and Life Safety) are:

i) Type 1;

ii) Type 2;

iii) Type 3; and

iv) Type 4.

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Buildings have various parts (such as plinth, habitable rooms, kitchen, bathroom and water closets, ledges, mezzanine, store room, garage, basement, parapet, cabin, boundary wall, wells, septic tanks, meter rooms, staircases, roofs) and specific requirements such as essential habitable sizes/ heights, distance between well and septic tanks, to name a few that ensure a liveable environment thereby addressing health, safety and public safety. The safe distance between buildings and electric lines is also essential towards occupant safety. Accessibility for persons with disabilities and the elderlies in specific public buildings and low income housing are also essential towards inclusivity in the built spaces. Open spaces within a plot also play a crucial part for harnessing natural lighting and ventilation, apart from movement space for fire tenders/emergency vehicles. Commensurate vehicular parking facilities within a built environment are also important so as to not burden the public spaces like roads/right of way. Such aspects may vary from place to place be it in the plains, hills and coastal areas; for which the state/local authorities should specify the specific criteria.

Based on their complexity, buildings may be classified as high rise buildings or special buildings in which the number of inmates and the requirement for their safe and habitable living are to be addressed. Importantly, the constructed buildings need to be well maintained and periodically audited covering structural audit, electrical audit, fire audit, accessibility audit to be carried out over the useful life of a building, say every 3 to 5 years.

A building project and the built facility in each stage necessarily requires professionals of many disciplines who should work together as a well-coordinated team to achieve the desired product delivery with quality, safety and other objectives, in an effective manner.

In large/complex projects, appropriate multi-disciplinary teams may be constituted to successfully meet the requirements of different stages. These may be design team, project management/execution team and operation and maintenance teams. Each team may comprise need-based professionals out of the following depending upon the nature, magnitude and complexity of the project:

1) architect;
2) civil engineer;
3) structural engineer;
4) geotechnical engineer;
5) electrical engineer;
6) plumbing engineer;
7) fire protection engineer;
8) heating, ventilation and air conditioning engineer;
9) lift, escalator and moving walk specialist;
10) acoustics specialist;
11) information/communication technology engineer;
12) health, safety and environment specialist;
13) environment/sustainability specialist;
14) town planner;
15) urban designer;
16) landscape architect;
17) security system specialist;
18) interior designer;

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19) quantity surveyor;

20) project/construction manager;

21) project planning and construction digitization specialist;

22) accessibility and universal design specialist; and

23) other subject specialist(s).

This NBCS (Part A) gives an overall direction for understanding the provisions of different types of buildings and built environment, and proposes guidance for utilizing appropriate knowledge and experience of qualified professionals, right from the conceptualization through construction and completion stages of a building project and indeed during the entire life cycle.

This part has been so formulated as to necessitate guidance to ensure quality and safety in the whole gamut of activities, primarily divided into planning, design and execution of a building or built environment. This may be ensured by obtaining assistance from the registered professionals involved in planning, design, execution, and supervision. Apart from these, various other resource groups are involved whose contribution may be duly taken into account depending on the type, nature, magnitude and complexity of the project, such as security system specialist, environment specialist/sustainability and green building specialist, accessibility specialist, project management consultant, interior designer in proper construction/erection, commissioning and operation of buildings and built environment. In this endeavour, a guide for qualification and competence of the associated professionals, is included in Annex A.

The aim of this Part is to get the maximum benefit from the building and its services in terms of objectives such as quality, timeliness and cost-effectiveness by following a team approach. In the team approach, which is an essential pre-requisite for a unified approach, the aim clearly is to maximize the efficiency of the total system through appropriate optimization of each of its sub-systems, and ensuring coordination among various disciplines and agencies that are involved during planning, design, construction, operation and maintenance of buildings and the associated infrastructure. In other words, in the team, the timely inputs from each of the professional disciplines have to be so optimized that the total system's efficiency becomes maximum. These teams may be design team, construction/execution team, and operation and maintenance team.

It is important that leaders and members of the teams, depending on the size and complexity of the project, are carefully selected considering their qualification, experience and expertise in these fields.

Tools like building information modelling (BIM) may be adopted in all major and complex building projects as they help in managing and sharing information throughout the life cycle of the project enabling better collaboration, planning and decision making. In such projects, BIM may be strategically adopted to enhance the multi-disciplinary approach in design and execution and as a fundamental tool in the planning and development stages.

After the useful service life of a building is over, or for other reasons such as redevelopment and proposed change in use of the land and built facility, it may be required to deconstruct a building. Such a deconstruction is preceded by an organized decommissioning. The decommissioning and deconstruction needs to be well planned and coordinated among concerned building professionals so as to ensure safety during such operations, as also retrieval of appropriate components and systems, for their possible reuse or recycling, or disposal as may be appropriate. This may in turn require a comprehensive decommissioning and deconstruction (including demolition) plan, which may be prepared during the initial stages of the project and kept available for use at the end of life cycle of the same.

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ANNEX A

GUIDE FOR THE QUALIFICATIONS AND COMPETENCE OF REGISTERED BUILDING PROFESSIONALS

A-1 ESSENTIAL REQUIREMENTS

Every building/development work should be planned, designed and supervised by registered professionals. The registered professionals for carrying out the various activities should be: (a) architect, (b) engineer, (c) structural engineer, (d) geotechnical engineer, (e) supervisor, (f) town planner, (g) landscape architect, (h) urban designer, and (j) utility services engineer. Requirements of registration for various professionals by the Authority or by the body governing such profession and constituted under a statute, as applicable to practice within the local body's jurisdiction, are given in A-2.1 to A-2.8. The competence of such registered personnel to carry out various activities is also indicated in A-2.1.1 to A-2.8.1.

The qualification and competence of the engineers for utility services and of builder/constructor is prescribed in A-2.9 and A-2.10, respectively. The qualification and competence of the professionals involved in peer reviewing/ proof checking with respect to structural design is prescribed in A-2.11.

A-2 REQUIREMENTS FOR REGISTRATION AND COMPETENCE OF BUILDING PROFESSIONALS

A-2.1 Architect

The minimum qualifications for an architect should be the qualifications as provided for in the *Architects Act*, 1972 for registration with the Council of Architecture.

A-2.1.1 *Competence*

The registered architect should be competent to carry out the work related to the building/development permit as given below:

a) Preparation of all plans and information connected with building permit except engineering services of high rise/special buildings;

b) Issuing certificate of supervision and completion of all buildings pertaining to architectural aspects;

c) Preparation of subdivision/layout plans and related information connected with development permit of area up to 1 ha for metro-cities, and 2 ha for other places; and

d) Issuing certificate of supervision for development of land of area up to 1 ha for metro-cities, and 2 ha for other places.

A-2.2 Engineer

The minimum qualifications for an engineer should be graduate in civil engineering/architectural engineering of recognized Indian or foreign university, or the Corporate Member of Civil Engineering Division/Architectural Engineering Division of the Institution of Engineers (India) or the member of the statutory body governing such profession, as and when established.

A-2.2.1 *Competence*

The registered engineer should be competent to carry out the work related to the building/development permit as given below:

a) Preparation of all plans and information connected with building permit;

b) Structural details and calculations of buildings including sub-surface investigation on plot up to 500 m^2 and up to 5 storeys or 16 m in height;

c) Issuing certificate of supervision and completion for all buildings;

d) Preparation of subdivision/layout plans and related information connected with development permit of area up to 1 ha for metro-cities, and 2 ha for other places;

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e) Preparation of all service plans and related information connected with development permit; and

f) Issuing certificate of supervision for development of land for all area.

A-2.3 Structural Engineer

The minimum qualifications for a structural engineer should be graduate in civil engineering of recognized Indian or foreign university, or Corporate Member of Civil Engineering Division of Institution of Engineers (India), and with minimum 3 years' experience in structural engineering practice with designing and field work.

NOTE — The 3 years' experience may be relaxed to 2 years in the case of post graduate degree of recognized Indian or foreign university in the branch of structural engineering. In case of doctorate in structural engineering, the experience required would be one year.

A-2.3.1 *Competence*

The registered structural engineer should be competent to prepare the structural design, calculations and details for all buildings and carry out supervision.

A-2.3.1.1 In case of buildings having special structural features, as decided by the Authority, which are within the horizontal areas and vertical limits specified in **A-2.2.1**(b) and **A-2.5.1**(a) should be designed only by structural engineers.

A-2.4 Geotechnical Engineer

The minimum qualifications for a geotechnical engineer should be graduate in civil engineering of recognized Indian or foreign university, or Corporate Member of Civil Engineering Division of Institution of Engineers (India), and with minimum 3 years' experience in geotechnical engineering practice with designing and field work.

NOTE — The 3 years' experience may be relaxed to 2 years in the case of post graduate degree of recognized Indian or foreign university in the branch of geotechnical engineering. In case of doctorate in geotechnical engineering, the experience required would be one year.

A-2.4.1 *Competence*

The registered geotechnical engineer should be competent to carry out sub-surface investigations and give report thereof. These may *inter-alia* include performing various tests required to determine engineering properties of sub-strata and ground water and making recommendations about the type of foundation, soil bearing capacity and the depth at which the foundations shall be placed, considering the structural system and loads provided by the engineer/structural engineer.

A-2.4.1.1 In case of buildings to be founded on soils having special geotechnical conditions, as decided by the Authority, which are within the horizontal areas and vertical limits specified in **A-2.2.1**(b) and **A-2.5.1**(a) should be designed only by geotechnical engineers.

A-2.4.1.2 For specific information on pre-construction services and construction stage services to be provided by the geotechnical engineers, reference may be made to IS 19235 : 2025 'Geotechnical engineering services — Requirements'.

A-2.5 Supervisor

The minimum qualifications for a supervisor should be diploma in civil engineering or architectural assistantship, or the qualification in architecture or engineering equivalent to the minimum qualification prescribed for recruitment to non-gazetted service by the Government of India plus 5 years' experience in building design, construction and supervision.

A-2.5.1 *Competence*

The registered supervisor should be competent to carry out the work related to the building permit as given below:

a) All plans and related information connected with building permit for residential buildings on plot up to 200 m^2 and up to two storeys or 7.5 m in height; and

b) Issuing certificate of supervision for buildings as per (a).

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A-2.6 Town Planner

The minimum qualification for a town planner should be the Associate Membership of the Institute of Town Planners or graduate or post-graduate degree in town and country planning.

A-2.6.1 *Competence*

The registered town planner should be competent to carry out the work related to the development permit as given below:

a) Preparation of plans for land subdivision/layout and related information connected with development permit for all areas; and

b) Issuing of certificate of supervision for development of land of all areas.

NOTE — However, for land layouts for development permit above 5 ha in area, landscape architect may also be associated, and for land development infrastructural services for roads, water supplies, sewerage/drainage, electrification, the registered engineers for utility services may be associated.

A-2.7 Landscape Architect

The minimum qualification for a landscape architect should be the bachelor master's degree in landscape architecture or equivalent from recognized Indian or foreign university.

A-2.7.1 *Competence*

The registered landscape architect should be competent to carry out the work related to landscape design for building/development permit for land areas 5 ha and above. In case of metro-cities, this limit of land area may be 2 ha and above.

NOTE — For smaller areas below the limits indicated above, association of landscape architect may also be considered from the point of view of desired landscape development.

A-2.8 Urban Designer

The minimum qualification for an urban designer should be the master's degree in urban design or equivalent from recognized Indian or foreign university.

A-2.8.1 *Competence*

The registered urban designer should be competent to carry out the work related to the building permit for urban design for land areas more than 5 ha and campus area more than 2 ha. He/she should also be competent to carry out the work of urban renewal for all areas.

NOTE — For smaller areas below the limits indicated above, association of urban designer may be considered from the point of view of desired urban design.

A-2.9 Engineers for Utility Services

The work of building and plumbing services should be executed under the planning, design and supervision of competent personnel. The qualification for registered mechanical engineer (including HVAC), electrical engineer and plumbing engineers for carrying out the work of air conditioning, heating and mechanical ventilation, electrical installations, lifts and escalators and water supply, drainage, sanitation and gas supply installations respectively should be as given in Part D 'Building Services' and Part E 'Plumbing Services' of this NBCS or as decided by taking into account practices of the national professional bodies dealing with the specialist engineering services.

Such an approach may be followed for association of other/multi-disciplinary professionals for taking inputs and associating with their areas of specialization. For example, specific to fire (as covered under Part F 'Fire and Life Safety' of this NBCS), the role of fire protection engineer is important, who could be, any of the following:

a) a graduate in fire protection engineering (or a related discipline like fire engineering, fire safety engineering, fire technology and safety engineering, of recognized Indian or foreign university);

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b) a graduate in mechanical/civil/electrical/electronics engineering and having a post graduate degree in fire protection engineering;

c) a graduate in mechanical/civil/electrical/electronics engineering and having at least 10 years of experience in the field of planning, designing, and commissioning of fire protection work;

d) an advance diploma in fire engineering (Divisional Officers Course) from National Fire Service College, Nagpur and having atleast five years' experience in the field of fire prevention, fire protection and fire safety matters; and

e) a diploma in fire engineering from National Fire Service College, Nagpur and having atleast ten years' experience in the field of fire prevention, fire protection and fire safety matters.

NOTE — The existing fire protection professionals having graduation in science or diploma in engineering having experience of at least 10 years may be allowed for the activity.

A-2.10 BUILDER/CONSTRUCTOR

The minimum qualification for the builder/constructor or his representative for execution of respective works should be as given in **A-2.1**, **A-2.2**, **A-2.3**, **A-2.4**, **A-2.5**, **A-2.6**, **A-2.7**, **A-2.8** and **A-2.9** for the concerned professional.

A-2.10.1 Competence

The qualified builder/constructor or his representative should be competent to carry out execution of work, which should have the same extent as for supervision by such professional as prescribed in **A-2.1.1**, **A-2.2.1**, **A-2.3.1**, **A-2.4.1**, **A-2.5.1**, **A-2.6.1**, **A-2.7.1**, **A-2.8.1** and **A-2.9**.

A-2.11 PEER REVIEWER/PROOF CHECKER (FOR STRUCTURAL DESIGN)

The individual peer reviewer/proof checker or the team leader of the peer review/proof checking organization should have the following qualification, experience and competence:

a) *For buildings up to the height of 15 m*

B.E./B.Tech (Civil) with 7 years of experience in structural engineering practice with designing and field work of relevant structures. The 7 years of experience should comprise a minimum of 5 years exclusively in structural designing.

b) *For buildings of height more than 15 m and up to 50 m*

B.E./B.Tech (Civil) with 10 years of experience in structural engineering practice with designing and field work of relevant structures. The 10 years of experience should comprise a minimum of 7 years exclusively in structural designing.

c) *For buildings of height more than 50 m and specialized structures*

1) Master's degree with major in structural engineering and 10 years of experience in structural engineering practice with designing and field work of relevant structures. The 10 years of experience should comprise a minimum of 7 years exclusively in structural designing; or

2) B.E./B.Tech (Civil) with 15 years of experience in structural engineering practice with designing and field work of relevant structures. The 15 years of experience should comprise a minimum of 10 years exclusively in structural designing.

d) Peer review may also be carried out by reputed academic institutions/research institutions (such as IITs, NITs, IISc, leading engineering colleges, CSIR-SERC, CBRI, CRRI).

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART B BUILDING MATERIALS

BUREAU OF INDIAN STANDARDS

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National Building Code Sectional Committee, CED 46

FOREWORD

This NBCS (Part B) lists Indian Standard specifications for materials used in building construction. The Indian Standards for methods of tests, to ensure the requirements demanded of the materials in the various situations, are also included. This is with a view to ensuring the quality and effectiveness of building materials used in the construction and of their storage, which are as important as other aspects of building activity like planning, designing and constructing the building itself.

Historically, choice of building materials was determined by what was locally available, appropriateness to geo-climatic conditions and affordability of users. In the recent past, different initiatives have been taken in the areas of research and development, standardization, and development and promotion of innovative materials. A review of the recent trends indicates that the growth in the area of building materials covers emerging trends and latest developments in the use of wastes, mineral admixtures in cement and concrete, substitutes to conventional timber, composite materials and recycling of wastes, at the same time ensuring desired response of materials in relation to fire safety, long term performance and durability. In addition to these, nowadays, development of specific materials is being researched which may be structured and designed to meet needs to specially developed construction technologies in time to come, such as, for disaster prone areas or aggressive climatic and industrial situations.

To encourage use of appropriate materials, it may be desirable to have to the largest extent possible, performance oriented approach for specifications rather than prescriptive approach. The approach has been already adopted in some cases in development of standards, wherever found possible.

Indian Standards cover the requirements for most of the materials in use. However, there may be a gap between development of new materials and techniques of application and formulation of standards. It, therefore, becomes necessary for the NBCS to be flexible to recognize building materials other than those for which Indian Standards are available. This Part of NBCS, therefore, since its first version, duly takes care of this aspect and explicitly provides for use of new or alternate building materials, provided it is proved by authentic tests that the new or alternative material is effective and suitable for the purpose intended. However, it is worthwhile that more and more emphasis is given to the satisfaction of performance requirements expected of a building material, so that a wide range of such new or alternate materials can be evaluated and used, if found appropriate.

As already emphasized, quality of material is quite important for their appropriate usage, whether it is a material for which an Indian Standard is available or a new or alternative material as defined in **4** of this Part of the NBCS. While continuing to emphasize on conformity of building materials to available Indian Standards, this Part recommends that the building regulating authority may also recognize the use of building materials conforming to other specifications and test methods (*see* **4.2**), in case Indian Standards are not available for particular materials. Third party certification schemes available in the country for quality assurance of above materials can be used with advantage to ensure the appropriateness of these materials.

It is well recognized that buildings during their construction and subsequently in their operation consume lot of resources. The holistic approach for ensuring proper and sustainable developments is to take care of the concerned aspects affecting sustainability during the conceptualization, planning, design, construction, and operation and maintenance stages. The building material production, transportation and utilization assume a substantial component of the embodied energy that goes into a building. It is therefore important that sustainability of building materials is well understood and ensured in a building project. This Part of the NBCS recognizes this aspect and covers appropriate provision therefore.

This Part of the Code was first published in 1970 as part of the National Building Code of India and subsequently revised in 1983, 2005 and 2016 as Part 5 of NBC 2016. The first and second revision of this Part incorporated an updated version of the list of Indian Standards given at the end of this Part of the Code. Also the list of building materials was completely revised to make it more user-friendly. The third revision of this Part included a new clause on sustainable building materials, expanded the clause on new/alternative materials to provide examples of some potential new/alternative materials, reorganized some of the existing categories and added new categories

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either under the existing principal categories or as separate principal categories. Additionally, the list of Indian Standards was updated.

In this revision, this publication has been renamed as National Building Construction Standards (NBCS).

This fourth revision of this Part, while basically retaining the structure of previous version, incorporates the following modifications:

a) The clause on new/alternative materials has been updated with examples of some potential new/alternative materials;

b) A clause on recycled materials has been added to address/encourage the usage of recycled materials in building construction; and

c) The list at the end of this Part has been updated to reflect the latest available Indian Standard specifications and methods of test.

A reference to SP 21 : 2005 'Summaries of Indian Standards for Building Materials' may be useful. This publication gives the summaries of Indian Standards covering various building materials, fittings and components except standards relating to paints.

All standards cross-referred in the main text of this Part, are subject to revision. The parties to agreement based on this Part are encouraged to investigate the possibility of applying the most recent editions of the standards.

As the regulation of land development and buildings is a State subject and is dealt by the States and local bodies such as urban local bodies within their jurisdiction through building regulatory documents like building regulations, building byelaws, and fire regulations, this document is voluntary in nature and is non-binding. The implementation depends on adoption by concerned parties or stipulations in a contract or by appropriate adoption by the concerned authorities.

For the purpose of deciding whether a particular requirement of this Part is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 2022 'Rules for rounding off numerical values (*second revision*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this Part.

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART B BUILDING MATERIALS

1 SCOPE

This NBCS (Part B) covers the requirements of building materials and components, and criteria for accepting new or alternative building materials and components.

2 MATERIALS

Every material used in fulfilment of the requirements of this Part, unless otherwise specified in this NBCS or approved, shall conform to the relevant Indian Standards. A list of Indian Standard specifications as the 'accepted standards' is given at the end of this Part of the NBCS. At the time of publication of this NBCS, the editions indicated were valid. All standards are subject to amendments and revisions. The Authority shall take cognizance of such amendments and revisions. The latest version of a specification shall, as far as possible, be adopted at the time of enforcement of this Part of the NBCS.

3 SUSTAINABLE MATERIALS

Choice of building materials is important in sustainable design because of the extensive network of activities such as extraction, processing and transportation steps required for making a material, and activities involved thereafter till building construction and even thereafter. These activities may pollute the air, soil and water, as well as destroy natural habitats and deplete natural resources. One of the most effective strategies for minimizing the environmental impacts of material usage is to reuse existing buildings. Rehabilitation of existing building, and their shell and non-shell components, not only reduces the volume of solid waste generated and its subsequent diversion to landfills but also the environmental impacts associated with the production, delivery and use or installation of new building materials. However, the use of sustainable building materials is one of the best strategies in the pursuit to sustainable buildings.

An ideal sustainable building material is not only environment friendly but also causes no adverse impact on health of occupants, is readily available, can be reclaimed and recycled, and is not only made from renewable raw material, but also uses predominantly renewable energy in its extraction, production, transportation, fixing and ultimate disposal. Practically, this kind of ideal material may not be available and hence, it may be best to choose materials which fulfil most of these criteria. Sustainable building materials also offer specific benefits to the building owner and building occupants, such as, reduced maintenance/replacement costs over the life of the building; energy conservation; improved occupant health and productivity; lower costs associated with changing space configurations; and greater design flexibility.

The use of used materials (*see* **6**) and recycled and waste materials (*see* **7**) contribute substantially towards sustainability of building material usage.

The selection of sustainable building materials may be made in accordance with relevant guidelines/best practices (Handbook on 'Sustainability in Buildings').

4 NEW OR ALTERNATIVE MATERIALS

4.1 The provisions of this Part are not intended to prevent the use of any material not specifically prescribed under **2**. Any such material may be approved by the Authority or an agency appointed by them for the purpose, provided it is established that the material is satisfactory for the purpose intended and the equivalent of that required in this Part or any other specification issued or approved by the Authority. The Authority or an agency appointed by them shall take into account the following parameters, as applicable to the concerned new or alternative building material:

a) Requirements of the material specified/expected in terms of the provisions given in the standards on its usage, including its applicability in geo-climatic condition;

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b) General appearance;

c) Dimension and dimensional stability;

d) Structural stability including strength properties;

e) Fire safety;

f) Durability;

g) Thermal properties;

h) Mechanical properties;

j) Acoustical properties;

k) Optical properties;

m) Biological effect;

n) Environmental aspects;

p) Working characteristics;

q) Ease of handling;

r) Consistency and workability;

s) UV resistance; and

t) Toxicity.

Some of such new/alternative materials may be decorative concrete, polymer concrete, plastic waste bricks, recycled plastic lumber, micro-concrete repair materials, dry-mix mortar, non-shrink grout, special materials for bunkers/blast resistant structures, artificial stones, nanotechnology based advanced materials, vermiculite based boards, exfoliated perlite, pervious concrete, high-pressure laminate sheet, aluminium honeycomb panels, zinc composite panels, stainless steel insulated water tanks, etc.

For establishing the performance of the material/component, laboratory/field tests, and field trials, as required, and study of historical data are recommended. For sampling and frequency of tests for new or alternate materials, similar product standards shall be referred to.

The above materials would no longer be treated as new/alternative materials as soon as Indian Standards for the same are established.

4.2 Approval in writing of the Authority or an agent appointed by them for the purpose of approval of material, shall be obtained by the owner or his agent before any new, alternative or equivalent material is used. The Authority or their agent shall base such approval on the principle set forth in **4.1** and shall require that tests be made (*see* **9.1**) or sufficient evidence or proof be submitted, at the expense of the owner or his agent, to substantiate any claim for the proposed material.

5 THIRD PARTY CERTIFICATION

For ensuring the conformity of materials for which Indian Standards exist and for new or alternative building materials, to requisite quality parameters the services under the third party certification schemes of the Government, may be utilized with advantage.

6 USED MATERIALS

Utilization of used materials and recycled materials may not be precluded provided these meet the requirements of this Part for new materials.

7 RECYCLED MATERIALS AND WASTE MATERIALS

7.1 A recycled building material refers to any product or material that has been previously utilized in another construction context. Various materials, such as brick, steel, timber, and even entire components like doors,

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windows and tiles, can be reused and repurposed. This term also encompasses building materials produced from waste, such as recycled plastic bricks, or concrete made from recycled aggregate from buildings, structures and pavements. Using suitable technological interventions, the natural and industrial by-products/ wastes can be gainfully utilized in the construction activity, not merely as fillers, but in combination with traditional materials or through repurposing. Some such potential materials include: fly ash, bottom ash, red mud, flue gas desulfurization (FGD) gypsum, phosphogypsum, fluorogypsum, blast furnace slag, Linz-Donawitz (LD) slag, textile sludge, marble waste (from cutting and processing), granite waste (from cutting and processing), iron tailings, mine tailings, copper tailings, copper slag, coal mining waste (coal tailings), rice husk ash, jarosite, broken glass and ceramics, kiln dust, galvanizing waste, tannery waste, stone waste (from crusher units), metal plating industry sludge, lime sludge, vermiculite, perlite and bagasse and other agro-wastes. One of such recycled/waste derived materials is aggregate derived from construction and demolition (C&D) waste, as described in **7.1.1** and **7.1.2**.

7.1.1 *Recycled Mixed Aggregate and Recycled Concrete Aggregate*

Recycled mixed aggregate (RMA) is made from construction and demolition (C&D) waste which may comprise concrete, brick, tiles, stones, etc. C&D waste is generated from construction, renovation, repair, and demolition of buildings and other structures. Recycled concrete aggregate (RCA) on the other hand is derived from concrete waste after requisite processing. Recycled concrete aggregate (RCA) comprises not only the original aggregate, but also the hydrated cement paste adhering to its surface.

7.1.2 *Reclaimed Asphalt Pavement Aggregates*

Milling or demolition of distressed bituminous/flexible pavements generates reclaimed asphalt pavement (RAP) aggregates. These aggregates consist of natural stones fully coated with 2 percent to 7.5 percent (by mass) of bitumen/asphalt binder and are typically used in the construction of new bituminous roads. The use of RAP in cement concrete pavements is rather a new approach and has been found to enhance the workability and toughness characteristics. The strength properties should also be evaluated before permitting for use. Suitably, the coarser fraction of RAP can be used as part replacement (in the order of 10 percent to 20 percent) of natural aggregates for non-structural lean-concrete applications such as paver blocks, kerb stones, etc.

8 STORAGE OF MATERIALS

All building materials shall be stored on the building site in such a way as to prevent deterioration or the loss or impairment of their structural and other essential properties [see relevant guidelines/best practices (Handbook on ‘Construction Management, Practices and Safety in Buildings’)].

9 METHODS OF TEST

9.1 Every test of material as required in this Part or by the Authority shall be carried out in accordance with the methods of test prescribed in relevant Indian Standards. In cases of methods of tests for which Indian Standards are not available, the same shall conform to the methods of tests issued by the Authority or their agent. A list of Indian Standard methods of test is given at the end of this Part of the NBCS as the ‘accepted standards’. Laboratory tests shall be conducted by recognized laboratories acceptable to the Authority.

9.1.1 The manufacturer/supplier shall satisfy himself that materials conform to the requirements of the specifications and if requested shall supply a certificate to this effect to the purchaser or his representative. When such test certificates are not available, the specimen of the material shall be tested.

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LIST OF STANDARDS

Following are the Indian Standards for various building materials and components, to be complied with in fulfilment of the requirements of this NBCS.

In the following list, while enlisting the Indian Standards, the materials have been categorized in such a way as to make the list user friendly. In the process, if so required, some of the standards have been included even in more than one category of products, such as in the category based on composition as well as on end application of the materials. The list has been arranged in alphabetical order of their principal category as given below:

1 Aluminium and other light metals and their alloys
2 Bitumen and tar products
3 Bricks, blocks and other masonry building units
4 Builder's hardware
5 Building chemicals
6 Building lime and products
7 Clay and stabilized soil products
8 Cement and concrete (including steel reinforcement for concrete)
9 Composite matrix products (including cement and resin matrix products)
10 Conductors and cables
11 Doors, windows and ventilators
12 Electrical wiring, fittings and accessories
13 Fillers, stoppers and putties
14 Floor covering, roofing and other finishes
15 Glass
16 Gypsum based materials
17 Mortar (including sand for mortar)
18 Paints and allied products
19 Polymers, plastics and geosynthetics/geotextiles
20 Sanitary appliances and water fittings
21 Steel and its alloys
22 Stones
23 Structural sections
24 Thermal insulation materials
25 Threaded fasteners, rivets and nails
26 Timber, bamboo and other lignocellulosic building materials
27 Unit weights of building materials
28 Waterproofing and damp-proofing materials
29 Welding electrodes and wires
30 Wire ropes and wire products (including wire for fencing)

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1 ALUMINIUM AND OTHER LIGHT METALS AND THEIR ALLOYS

IS No.	*Title*
IS 733 : 2025	Wrought aluminium and aluminium alloy bars, rods, sections, extruded tubes, hollow sections and profiles for general engineering purposes — Specification (*fourth revision*)
IS 734 : 1975	Specification for wrought aluminium and aluminium alloy forging stock and forgings (for general engineering purposes) (*second revision*)
IS 736 : 1986	Specification for wrought aluminium and aluminium alloys plate for general engineering purposes (*third revision*)
IS 737 : 2024	Wrought aluminium and aluminium alloys sheet and strip for general engineering purposes — Specification (*fifth revision*)
IS 738 : 1994	Wrought aluminium and aluminium alloys — Drawn tube for general engineering purposes — Specification (*third revision*)
IS 739 : 1992	Wrought aluminium and aluminium alloys — Wire for general engineering purposes — Specification (*third revision*)
IS 740 : 1977	Specification for wrought aluminium and aluminium alloys rivet stock for general engineering purposes (*second revision*)
IS 1254 : 2025	Corrugated aluminium sheet (*fifth revision*)
IS 1284 : 2024	Wrought aluminium alloys bolt and screw stock (for general engineering purposes) — Specification (*third revision*)
IS 1500	Metallic materials — Brinell hardness test:
(Part 1) : 2019	Test method (*fifth revision*)
(Part 2) : 2021	Verification and calibration of testing machines (*fifth revision*)
(Part 3) : 2019	Calibration of reference blocks (*fifth revision*)
(Part 4) : 2019	Table of hardness values (*fifth revision*)
IS 1501	Metallic materials — Vickers hardness test:
(Part 1) : 2025	Test method (*sixth revision*)
(Part 2) : 2020	Verification and calibration of testing machines (*fifth revision*)
(Part 3) : 2020	Calibration of reference blocks (*fifth revision*)
(Part 4) : 2020	Tables of hardness values (*fifth revision*)
IS 1586	Metallic materials — Rockwell hardness test:
(Part 1) : 2025/ ISO 6508-1 : 2023	Test method (*sixth revision*)
(Part 2) : 2025/ ISO 6508-2 : 2023	Verification and calibration of testing machines and indenters (*sixth revision*)

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IS No.	*Title*
IS 1586 (Part 3) : 2025/ ISO 6508-3 : 2023	Metallic materials — Rockwell hardness test: Part 3 Calibration of reference blocks (*sixth revision*)
IS 1716 : 2023	Metallic materials wire reverse bend test (*third revision*)
IS 1717 : 2018	Metallic materials — Wire — Simple torsion test (*fourth revision*)
IS 1757	Metallic materials — Charpy pendulum impact test:
(Part 1) : 2020	Test method (*fourth revision*)
(Part 2) : 2020	Verification of testing machines (*fourth revision*)
(Part 3) : 2020	Preparation and characterization of charpy V-notch test pieces for indirect verification of pendulum impact machines (*fourth revision*)
IS 1772 : 2024	Electroplated coatings of copper — Specification (*second revision*)
IS 1828 (Part 1) : 2022	Metallic materials — Calibration and verification of static uniaxial testing machines: Part 1 Tension/Compression testing machines — Calibration and verification of the force-measuring system (*fifth revision*)
IS 2258 : 2023	Rolled zinc plate, sheet and strip — Specification (*third revision*)
IS 2328 : 2018	Metallic materials — Tube — Flattening test (*third revision*)
IS 2479 : 1981	Colour code for the identification of aluminium and aluminium alloys for general engineering purposes (*second revision*)
IS 2605 : 2023	Zinc anodes for electroplating — Specification (*second revision*)
IS 2676 : 1981	Dimensions for wrought aluminium and aluminium alloys, sheet and strip (*first revision*)
IS 2677 : 1979	Dimensions for wrought aluminium and aluminium alloys, plates and hot rolled sheets (*first revision*)
IS 2782 : 2023	Refined nickel — Specification (*first revision*)
IS 3577 : 1992	Wrought aluminium and its alloys — Rivet, bolt and screw stock — Dimensions and tolerances (*first revision*)
IS 3711 : 2020	Steel and steel products — Location and preparation of samples and test pieces for mechanical testing (*third revision*)
IS 3965 : 1981	Dimensions for wrought aluminium and aluminium alloys bar, rod and section (*first revision*)
IS 4258 : 2018	Metallic materials — Conversion of hardness values (*third revision*)
IS 5075 : 2023	Metallic materials — Rotating bar bending fatigue testing (*second revision*)
IS 5528 : 2024/ ISO 9227 : 2022	Corrosion tests in artificial atmospheres — Salt spray tests (*second revision*)
IS 6477 : 1983	Dimensions for wrought aluminium and aluminium alloys extruded hollow sections (*first revision*)

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IS No.	*Title*
IS 6885	Metallic materials — Knoop hardness test:
(Part 1) : 2025/ ISO 4545-1 : 2023	Test method (*third revision*)
(Part 2) : 2020	Verification and calibration of testing machines (*second revision*)
(Part 3) : 2020	Calibration of reference blocks (*second revision*)
(Part 4) : 2020	Tables of hardness values (*second revision*)
IS 7811 : 2019	Phosphor bronze rods and bars (*second revision*)
IS 8632 : 2023	Metallic materials — Designation of test specimen axes in relation to product texture (*first revision*)
IS 10175 : 2018	Metallic materials — Sheet and strip — Erichsen cupping test (*third revision*)
IS 11999 : 2022	Metallic materials — Sheet and strip — Determination of plastic strain ratio (*second revision*)
IS 12260 : 2018	Metallic materials — Tube — Ring —Expanding test (*first revision*)
IS 12278 : 2018	Metallic materials — Tube — Ring tensile test (*second revision*)
IS 12444 : 2020	Copper wire rods for electrical applications — Specification (*first revision*)
IS 12871 : 2022	Copper, lead, zinc and nickel concentrates — Sampling procedures for determination of metal and moisture content (*second revision*)
IS 14712 : 2025	Wrought aluminium and its alloys — Chequered/tread sheets for general engineering purposes — Specification (*first revision*)
IS 15756 : 2022	Metallic materials — Sheet and strip — Determination of tensile strain hardening exponent (*first revision*)
IS 15965 : 2012	Pre-painted aluminium zinc alloy metallic coated steel strip and sheet (Plain)
IS 16842 : 2022	Metallic materials — Fatigue testing — Fatigue crack growth method (*first revision*)
IS 16843 : 2018	Metallic materials — Fatigue testing — Strain-controlled thermomechanical fatigue testing method
IS 17143 : 2023	Metallic materials torque — Controlled fatigue testing (*first revision*)
IS 17144	Metallic materials — Instrumented indentation test for hardness and materials parameters:
(Part 1) : 2019	Test method
(Part 2) : 2019	Verification and calibration of testing machines
(Part 3) : 2019	Calibration of reference blocks
(Part 4) : 2019	Test method for metallic and non-metallic coatings

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IS No.	*Title*
IS 17145	Metallic materials — Fatigue testing — Variable amplitude fatigue testing:
(Part 1) : 2019	General principles, test method and reporting requirements
(Part 2) : 2019	Cycle counting and related data reduction methods
IS 17149	Metallic materials — Leeb hardness test:
(Part 1) : 2019	Test method
(Part 2) : 2019	Verification and calibration of the testing devices
(Part 3) : 2019	Calibration of reference test blocks
IS 17151 : 2023	Metallic materials unified method of test for the determination of quasistatic fracture toughness (*first revision*)
IS 17413	Metallic materials — Tensile testing at high strain rates:
(Part 1) : 2020	Elastic-bar-type systems
(Part 2) : 2020	Servo-hydraulic and other systems
IS 17415 : 2023	Metallic materials torsion test at room temperature (*first revision*)
IS 17416 : 2024/ ISO 14556 : 2023	Metallic materials — Charpy V-Notch pendulum impact test — Instrumented test method (*first revision*)
IS 17418 : 2020	Metallic materials — Sheet and strip — Biaxial tensile testing method using a cruciform test piece
IS 17146 (Part 1) : 2023	Metallic materials determination of forming-limit curves for sheet and strip: Part 1 Measurement and application of forming-limit diagrams in the press shop (*first revision*)
IS 17417	Metallic materials — Dynamic force calibration for uniaxial fatigue testing:
(Part 1) : 2020	Testing systems
(Part 2) : 2020	Dynamic calibration device (DCD) instrumentation
IS 17419 : 2025	Metallic materials — Ductility testing — High speed compression test for porous and cellular metals (*first revision*)
IS 17679 : 2021	Metallic materials — Method of test for the determination of quasistatic fracture toughness of welds
IS 17682 : 2021	Aluminium composite panel specification
IS 17795 : 2025	Metallic materials — Uniaxial creep testing in tension — Method of test (*first revision*)
IS 17915 : 2022	Method for evaluation of tensile properties of metallic superplastic materials
IS 17937 : 2022	Mechanical testing of metals ductility testing compression test for porous and cellular metals
IS 18649 : 2024/	Corrosion of metals and alloys — Alternate immersion test in salt solution

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IS No.	*Title*
ISO 11130 : 2017	
IS 18650 : 2024/ ISO 26146 : 2012	Corrosion of metals and alloys — Method for metallographic examination of samples after exposure to high-temperature corrosive environments

2 BITUMEN AND TAR PRODUCTS

IS No.	*Title*
IS 73 : 2013	Paving bitumen — Specification (*fourth revision*)
IS 212 : 2021	Crude coal tar for general use (*third revision*)
IS 216 : 2006	Coal tar pitch — Specification (*second revision*)
IS 217 : 2018	Cutback bitumen — Specification (*third revision*)
IS 218 : 1983	Specification for creosote oil for use as wood preservatives (*second revision*)
IS 334 : 2023	Glossary of terms relating to bitumen and tar (*fourth revision*)
IS 702 : 2022	Specification for industrial bitumen (*third revision*)
IS 1201 to IS 1220	Methods for testing tar and bituminous materials:
IS 1201 : 2021	Sampling (*second revision*)
IS 1202 : 2021	Determination of specific gravity (*second revision*)
IS 1203 : 2022	Determination of penetration (*second revision*)
IS 1204 : 2022	Determination of residue of specified penetration (*second revision*)
IS 1205 : 2022	Determination of softening point — Ring and ball apparatus (*second revision*)
IS 1206	Determination of viscosity:
(Part 1) : 2023	By volumetric flow rate method
(Part 2) : 2022	Absolute viscosity
(Part 3) : 2021	Kinematic viscosity
IS 1207 : 2022	Determination of equiviscous temperature EVT (*second revision*)
IS 1208	Methods for Testing Tar and Bituminous Materials
(Part 1) : 2023	Determination of ductility (*second revision*)
(Part 2) : 2023	Determination of elastic recovery (*second revision*)
IS 1209 : 2021	Determination of flash point and fire point (*second revision*)
IS 1210 : 2020	Float test (*second revision*)
IS 1211 : 2022	Determination of water content dean and stark method (*second revision*)
IS 1212 : 2022	Determination of loss on heating (*second revision*)

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IS No.	*Title*
IS 1213 : 2025	Tar and bituminous materials — Distillation test — Methods of test (*third revision*)
IS 1215 : 2021	Determination of matter insoluble in toluene (*second revision*)
IS 1216 : 2023	Determination of solubility in trichloroethylene (*second revision*)
IS 1217 : 2024	Tar and bituminous materials — Determination of mineral matter (ash) — Methods of test (*second revision*)
IS 1218 : 2024	Tar and bituminous materials — Methods for test — Determination of phenols (*second revision*)
IS 1219 : 2024	Tar and bituminous materials — Methods for test — Determination of naphthalene (*second revision*)
IS 1220 : 2024	Tar and bituminous materials — Methods for test — Determination of volatile matter content *(second revision)*
IS 3117 : 2004	Bitumen emulsion for roads and allied applications (anionic type) — Specification (*first revision*)
IS 8887 : 2018	Bitumen emulsion for roads cationic type — Specification *(third revision)*
IS 9381 : 2024	Tar and bituminous materials — Determination of Fraass breaking point of bitumen — Methods of test (*first revision*)
IS 9382 : 2022	Methods for testing tar and bituminous materials — Determination of effect of heat and air by thin film oven test (*first revision*)
IS 10511 : 2024	Method for determination of asphaltenes in bitumen by precipitation with normal heptane (*first revision*)
IS 10512 : 2003	Method for determination of wax content in bitumen (*first revision*)
IS 13758	Coal tar pitch — Methods of test:
(Part 1) : 1993	Determination of matter insoluble in quinoline
(Part 2) : 1993	Determination of coking value
IS 14982 : 2022	Anti-Stripping agents for bitumen addition — Specification (*second revision*)
IS 15172 : 2002	Methods for testing tar and bituminous materials — Determination of curing index for cutback bitumens
IS 15173 : 2002	Methods for testing tar and bituminous materials — Determination of breaking point for cationic bitumen emulsion
IS 15174 : 2002	Methods for testing tar and bituminous material — Determination of breaking point for anionic bitumen emulsion
IS 15462 : 2019	Polymer modified bitumen (PMB) specification (*first revision*)
IS 15799 : 2021	Methods for testing tar and bituminous materials — Ageing of bitumen (RTFOT and PAV) (*first revision*)
IS 15808 : 2008	Multigrade bitumen for use in pavement construction — Specification
IS 17016 : 2018	Cationic modified bitumen emulsion — Specification

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IS No.	*Title*
IS 17052 : 2018	Rejuvenating agents for hot mix recycling of bituminous surfacing — Specification
IS 17079 : 2019	Rubber modified bitumen (RMB) — Specification
IS 17116 : 2019	Determination of resistance of compacted bituminous mixture to moisture — Induced damage
IS 17124 : 2019	Crack sealants — Specification
IS 17127 : 2019	Determination of stability, flow and bulk density of bituminous mixes by Marshall method
IS 17335 : 2020	Determination of apparent viscosity of bituminous binders at elevated temperature by rotational viscometer
IS 18104 : 2022	Methods for testing tar and bituminous materials — Determination of draindown characteristics of uncompacted bitumen mixtures
IS 18130 : 2023	Method for recovery of bitumen from solution using rotary evaporator
IS 18883 : 2025	Trackless emulsion — Specification
IS 19008 : 2023	Determination of bitumen content in bituminous paving mixtures

3 BRICKS, BLOCKS AND OTHER MASONRY BUILDING UNITS

IS No.	*Title*
IS 1077 : 2025	Common burnt clay building bricks — Specification (*sixth revision*)
IS 1725 : 2023	Stabilized soil blocks used in general building construction — Specification (*third revision*)
IS 2180 : 1988	Specification for heavy duty burnt clay building bricks (*third revision*)
IS 2185	Specification for concrete masonry units:
(Part 1) : 2005	Hollow and solid concrete blocks (*third revision*)
(Part 2) : 1983	Hollow and solid lightweight concrete blocks (*first revision*)
(Part 3) : 1984	Autoclaved cellular (aerated) concrete blocks (*first revision*)
(Part 4) : 2008	Preformed foam cellular concrete blocks
IS 2222 : 1991	Specification for burnt clay perforated building bricks (*third revision*)
IS 2691 : 2017	Burnt clay facing bricks — Specification (*third revision*)
IS 2849 : 1983	Specification for non-load bearing gypsum partition blocks (solid and hollow types) (*first revision*)
IS 3115 : 1992	Specification for lime based blocks (*second revision*)
IS 3316 : 1974	Specification for structural granite (*first revision*)
IS 3495	Burnt clay building bricks method of tests:

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IS No.	*Title*
(Part 1) : 2019	Determination of compressive strength (*fourth revision*)
(Part 2) : 2019	Determination of water absorption (*fourth revision*)
(Part 3) : 2019	Determination of efflorescence (*fourth revision*)
(Part 4) : 2019	Determination of warpage (*fourth revision*)
(Part 5) : 2021	Determination of initial rate of absorption
(Part 6) : 2022	Determination of modulus of rupture
IS 3583 : 1988	Specification for burnt clay paving bricks (*second revision*)
IS 3620 : 1979	Specification for laterite stone block for masonry (*first revision*)
IS 3952 : 2013	Burnt clay hollow bricks and blocks for walls and partitions — Specification (*third revision*)
IS 4139 : 1989	Specification for calcium silicate bricks (*second revision*)
IS 4860 : 1968	Specification for acid resistant bricks
IS 4885 : 1988	Specification for sewer bricks (*first revision*)
IS 5454 : 2024	Burnt clay bricks and burnt clay tiles — Methods of sampling (*second revision*)
IS 5751 : 1984	Specification for precast concrete coping blocks (*first revision*)
IS 5779 : 1986	Specification for burnt clay soling bricks (*first revision*)
IS 6165 : 1992	Dimensions for special shapes of clay bricks (*first revision*)
IS 9893 : 1981	Specification for precast concrete blocks for lintels and sills
IS 10360 : 1982	Specification for lime pozzolana concrete blocks for paving
IS 12440 : 1988	Specification for precast concrete stone masonry blocks
IS 12894 : 2002	Specification for pulverized fuel ash lime bricks (*first revision*)
IS 13757 : 1993	Burnt clay fly ash building bricks — Specification
IS 15658 : 2021	Concrete paving blocks — Specification (*first revision*)
IS 16720 : 2018	Pulverized fuel ash-cement bricks — Specification
IS 17165 : 2020	Manufacture of stabilized soil blocks — Guidelines

4 BUILDER'S HARDWARE

IS No.	*Title*
IS 204	Specification for tower bolts:
(Part 1) : 1991	Ferrous metals (*fifth revision*)
(Part 2) : 1992	Non-ferrous metals (*fifth revision*)

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IS No.	*Title*
IS 205 : 1992	Specification for non-ferrous metal butt hinges (*fourth revision*)
IS 206 : 2010	Specification for tee and strap hinges (*fifth revision*)
IS 208 : 2020	Door handles — Specification (*sixth revision*)
IS 281 : 2009	Specification for mild steel sliding door bolts for use with padlocks (*fourth revision*)
IS 362 : 1991	Specification for parliament hinges (*fifth revision*)
IS 363 : 1993	Specification for hasps and staples (*fourth revision*)
IS 364 : 1993	Specification for fanlight catch (*third revision*)
IS 452 : 1973	Specification for door springs, rat-tail type (*second revision*)
IS 453 : 1993	Specification for double-acting spring hinges (*third revision*)
IS 729 : 2021	Drawer locks cupboard locks and box locks — Specification (*fourth revision*)
IS 1019 : 1974	Specification for rim latches (*second revision*)
IS 1341 : 2018	Steel butt hinges — Specification (*sixth revision*)
IS 1823 : 1980	Specification for floor door stoppers (*third revision*)
IS 1837 : 1966	Specification for fanlight pivots (*first revision*)
IS 2681 : 1993	Specification for nonferrous metal sliding door bolts for use with padlocks (*third revision*)
IS 3564 : 2025	Hydraulically regulated door closers — Specification (*fifth revision*)
IS 3818 : 1992	Specification for continuous (piano) hinges (*third revision*)
IS 3828 : 1966	Specification for ventilator chains
IS 3843 : 1995	Steel back flap hinges — Specification (*second revision*)
IS 4621 : 1975	Specification for indicating bolts for use in public baths and lavatories (*first revision*)
IS 4948 : 2020	Welded steel wire fabric for general use — Specification (*third revision*)
IS 4992 : 2024	Door handles for mortice locks — Specification (*second revision*)
IS 5187 : 1972	Specification for flush bolts (*first revision*)
IS 5899 : 1970	Specification for bathroom latches
IS 6315 : 2025	Hydraulically regulated floor springs — Specification (*third revision*)
IS 6318 : 1971	Specification for plastic window stays and fasteners
IS 6343 : 1982	Specification for door closers (pneumatically regulated) for light door weighing up to 40 kg (*first revision*)
IS 7196 : 1974	Specification for hold fast

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IS No.	*Title*
IS 7197 : 1974	Specification for double action floor springs (without oil check) for heavy doors
IS 7534 : 1985	Specification for sliding locking bolts for use with padlocks (*first revision*)
IS 7881	Glossary of terms relating to builders' hardware:
(Part 1) : 2023	Locks (*first revision*)
(Part 2) : 2022	Latches (*first revision*)
IS 8756 : 1978	Specification for ball catches for use in wooden almirah
IS 9106 : 1979	Specification for rising butt hinges
IS 9131 : 2021	Rim locks and latches mechanically operated — Specification (*first revision*)
IS 9460 : 1980	Specification for flush drop handle for drawer
IS 9899 : 1981	Specification for hat coat and wardrobe hooks
IS 10019 : 1981	Specification for mild steel stays and fasteners
IS 10090 : 1982	Specification for numericals
IS 10342 : 1982	Specification for curtain rail system
IS 12817 : 2020	Stainless steel butt hinges — Specification (*third revision*)
IS 12867 : 1989	Specification for PVC hand rails covers
IS 14912 : 2001	Specification for door closers concealed type (hydraulically regulated)
IS 15833 : 2009	Stainless steel tower bolts — Specification
IS 15834 : 2020	Stainless steel sliding door bolts (Aldrops) for use with padlocks — Specification *(first revision)*
IS 16015 : 2022	Mortise locks and latches mechanically operated — Specification (*first revision*)
IS 16016 : 2013	Cylindrical locks with pin tumbler mechanism — Specification
IS 17296 : 2020	Stainless steel door stoppers — Specification
IS 17706 : 2022	Pull handles — Specification
IS 17954 : 2023	Telescopic ball bearing drawer slide — Specification
IS 18297 : 2023	Cabinet hinges — Specification

5 BUILDING CHEMICALS:

a) Anti-termite Chemicals

IS No.	*Title*
IS 5549 : 2023	Warfarin bait concentrates — Specification (*first revision*)
IS 7168 : 2023	Warfarin sodium salt water soluble powders — Specification (*first revision*)

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IS No.	*Title*
IS 8703 : 2023	Diuron water dispersible powder concentrates — Specification (*first revision*)
IS 8944 : 2005	Chlorpyrifos emulsifiable concentrates — Specification (*first revision*)
IS 16706 : 2018	Carpropamid, technical — Specification
IS 16955 : 2019	Spiromesifen, suspension concentrate (SC) — Specification

b) Chemical Admixture/Water Proofing Compounds

IS No.	*Title*
IS 2645 : 2003	Integral waterproofing compounds for cement mortar and concrete — Specification (*second revision*)
IS 6925 : 1973	Methods of test for determination of water soluble chlorides in concrete admixtures
IS 9103 : 1999	Specification for concrete admixtures (*first revision*)

c) Sealants/Fillers

IS No.	*Title*
IS 1834 : 1984	Specification for hot applied sealing compound for joint in concrete (*first revision*)
IS 1838	Specification for preformed fillers for expansion joint in concrete pavements and structures (non extruding and resilient type):
(Part 1) : 1983	Bitumen impregnated fibre (*first revision*)
(Part 2) : 1984	CNSL aldehyde resin and coconut pith
(Part 3) : 2011	Polymer based
IS 10566 : 1983	Methods of tests for preformed fillers for expansion joint in concrete paving and structural construction
IS 10959 : 2024/ ISO 6927 : 2021	Sealants for building purposes — Glossary of terms (*second revision*)
IS 11433	Specification for one-part gun — Grade polysulphide-based joint sealants:
(Part 1) : 1985	General requirements
(Part 2) : 1986	Methods of test
IS 12118	Specification for two parts polysulphide-based sealants:
(Part 1) : 1987	General requirements
(Part 2) : 1987	Methods of test
IS 13923 : 2023	Lead seal — Specification (*first revision*)

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d) Adhesives

IS No.	*Title*
IS 848 : 2025	Synthetic resin adhesives for plywood (phenolic, aminoplastic and bio-materials based) — Specification (*third revision*)
IS 851 : 1978	Specification for synthetic resin adhesives for construction work (non-structural) in wood (*first revision*)
IS 852 : 1994	Animal glue for general wood-working purposes — Specification (*second revision*)
IS 1508 : 1972	Specification for extenders for use in synthetic resin adhesives (urea-formaldehyde) for plywood (*first revision*)
IS 4835 : 1979	Specification for polyvinyl acetate dispersion-based adhesives for wood (*first revision*)
IS 5666 : 2024	Etch primer — Specification (*first revision*)
IS 9188 : 1979	Performance requirements for adhesive for structural laminated wood products for use under exterior exposure condition
IS 12830 : 1989	Rubber based adhesives for fixing PVC tiles to cement — Specification
IS 12994 : 1990	Epoxy adhesives, room temperature curing general purpose — Specification
IS 13238 : 2021	Epoxy based zinc phosphate primer (two pack) — Specification (*first revision*)
IS 15477 : 2019	Adhesives for use with ceramic tiles and mosaics (*first revision*)
IS 16239 : 2021	Polyurethane matt finish (two pack) — Specification
IS 16676 : 2024	Solventless liquid epoxy system for application on interior and exterior surfaces of steel water pipeline — Specification (*first revision*)
IS 16943 : 2019	Two components high build epoxy micaceous iron oxide (MIO) pigmented intermediate coat — Specification
IS 16944 : 2018	Two components high build epoxy titanium dioxide (TiO_2) pigmented intermediate coat — Specification
IS 17218 : 2019	Solar thermo-reflective exterior coating for cooling of roof top
IS 17545 : 2021	White cement based polymeric putty
IS 18837 : 2024	Solvent cements for chlorinated poly (vinyl chloride) (CPVC) plastic pipe and fittings

6 BUILDING LIME AND PRODUCTS

IS No.	*Title*
IS 712 : 1984	Specification for building limes (*third revision*)
IS 1624 : 1986	Method of field testing of building lime (*second revision*)
IS 2686 : 1977	Specification for cinder as fine aggregates for use in lime concrete (*first revision*)

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IS No.	*Title*
IS 3068 : 1986	Specification for broken brick (burnt-clay) coarse aggregates for use in lime concrete (*second revision*)
IS 3115 : 1992	Specification for lime based blocks (*second revision*)
IS 3182 : 1986	Specification for broken bricks (burnt clay) fine aggregates for use in lime mortar (*second revision*)
IS 4098 : 1983	Specification for lime-pozzolana mixture (*first revision*)
IS 4139 : 1989	Specification for calcium silicate bricks (*second revision*)
IS 6932	Method of tests for building limes:
(Part 1) : 2025	Determination of insoluble residue in dilute acid and alkali, loss on ignition, insoluble residue in hydrochloric acid, silicon dioxide, ferric and aluminium oxide, calcium oxide and magnesium oxide (*first revision*)
(Part 2) : 2025	Determination of carbon dioxide content (*first revision*)
(Part 3) : 2025	Determination of residue on slaking of quicklime (*first revision*)
(Part 4) : 2025	Determination of fineness of hydrated lime (*first revision*)
(Part 5) : 2025	Determination of unhydrated oxide (*first revision*)
(Part 6) : 2025	Determination of volume yield of quicklime (*first revision*)
(Part 7) : 2025	Determination of compressive and transverse strength (*first revision*)
(Part 8) : 2025	Determination of workability (*first revision*)
(Part 9) : 2025	Determination of soundness (*first revision*)
(Part 10) : 2025	Determination of popping and pitting of hydrated lime (*first revision*)
(Part 11) : 2025	Determination of setting time of hydrated lime (*first revision*)
IS 10360 : 1982	Specification for lime pozzolana concrete blocks for paving
IS 10772 : 1983	Specification for quick setting lime pozzolana mixture
IS 12894 : 2002	Specification for pulverized fuel ash lime bricks (*first revision*)
IS 15648 : 2006	Pulverized fuel ash for lime-pozzolana mixture applications — Specification

7 CLAY AND STABILIZED SOIL PRODUCTS

a) Blocks

IS No.	*Title*
IS 3952 : 2013	Burnt clay hollow bricks and blocks for walls and partitions — Specification (*third revision*)

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b) Stabilized Soil Products

IS No.	*Title*
IS 1725 : 2023	Stabilized soil blocks used in general building construction — Specification (*third revision*)

c) Bricks

IS No.	*Title*
IS 1077 : 2025	Common burnt clay building bricks — Specification (*sixth revision*)
IS 2180 : 1988	Specification for heavy duty burnt clay building bricks (*third revision*)
IS 2222 : 1991	Specification for burnt clay perforated building bricks (*third revision*)
IS 2691 : 2017	Burnt clay facing bricks — Specification (*third revision*)
IS 3495	Burnt clay building bricks — Method of tests
(Part 1) : 2019	Determination of compressive strength (*fourth revision*)
(Part 2) : 2019	Determination of water absorption (*fourth revision*)
(Part 3) : 2019	Determination of efflorescence (*fourth revision*)
(Part 4) : 2019	Determination of warpage (*fourth revision*)
(Part 5) : 2021	Determination of initial rate of absorption
(Part 6) : 2022	Determination of modulus of rupture
IS 3583 : 1988	Specification for burnt clay paving bricks (*second revision*)
IS 3952 : 2013	Burnt clay hollow bricks and blocks for walls and partitions — Specification (*third revision*)
IS 4885 : 1988	Specification for sewer bricks (*first revision*)
IS 5454 : 2024	Burnt clay bricks and burnt clay tiles — Methods of sampling (*second revision*)
IS 5779 : 1986	Specification for burnt clay soling bricks (*first revision*)
IS 6165 : 1992	Dimensions for special shapes of clay bricks (*first revision*)
IS 13757 : 1993	Burnt clay fly ash building bricks — Specification

d) Jallies

IS No.	*Title*
IS 7556 : 1988	Specification for burnt clay jallies (*first revision*)

e) Tiles

IS No.	*Title*
IS 654 : 2023	Clay roofing tiles Mangalore pattern — Specification (*fourth revision*)

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IS No.	*Title*
IS 1464 : 2024	Clay ridge and ceiling tiles — Specification (*third revision*)
IS 1478 : 2023	Clay flooring tiles — Specification (*third revision*)
IS 2690	Burnt clay flat terracing tiles — Specification
(Part 1) : 2023	Machine made (*third revision*)
(Part 2) : 2023	Hand made (*second revision*)
IS 3367 : 2023	Burnt clay tiles for use in lining irrigation and drainage works — Specification (*third revision*)
IS 3951	Hollow clay tiles for floor and roofs — Specification
(Part 1) : 2023	Filler type (*third revision*)
(Part 2) : 2023	Structural type (*third revision*)
IS 8920 : 1978	Methods for sampling of burnt clay tiles
IS 13317 : 2023	Clay roofing county tiles half round and flat tiles — Specification (*first revision*)
IS 17190 : 2020	Grouts for filling of joints of tiles and stones — Specification
IS 17897 : 2022	Stone-polymer composite flooring tiles and planks — Specification
IS 18387 : 2023	Gypsum ceiling tiles — Specification

8 CEMENT AND CONCRETE (INCLUDING STEEL REINFORCEMENT FOR CONCRETE)

a) Aggregates

IS No.	*Title*
IS 383 : 2016	Coarse and fine aggregate for concrete — Specification (*third revision*)
IS 4961 : 2022	Determination of particle size of powders by air elutriation methods (*first revision*)
IS 650 : 1991	Specification for standard sand for testing of cement (*second revision*)
IS 1542 : 1992	Specification for sand for plaster (*second revision*)
IS 2116 : 1980	Specification for sand for masonry mortars (*first revision*)
IS 2386	Methods of test for aggregates for concrete:
(Part 1) : 1963	Particle size and shape
(Part 2) : 1963	Estimation of deleterious materials and organic impurities
(Part 3) : 1963	Specific gravity, density, voids, absorption and bulking
(Part 4) : 1963	Mechanical properties
(Part 5) : 1963	Soundness

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IS No.	*Title*
(Part 6) : 1963	Measuring mortar making properties of fine aggregates
(Part 7) : 1963	Alkali aggregate reactivity
(Part 8) : 1963	Petrographic examination
IS 2430 : 1986	Methods of sampling of aggregates of concrete (*first revision*)
IS 4879 : 2023	Particulate materials — Sampling and sample splitting technique (*first revision*)
IS 5282	Determination of particle size distribution by gravitational liquid sedimentation methods:
(Part 1) : 2023	General principles and guidelines (*first revision*)
(Part 2) : 2023	Fixed pipette method (*first revision*)
(Part 3) : 2023	X-Ray gravitational technique *(first revision)*
(Part 4) : 2023	Balance method *(first revision)*
IS 6579 : 1981	Specification for coarse aggregate for water bound macadam (*first revision*)
IS 9142	Artificial light-weight aggregates for concrete — Specification:
(Part 1) : 2018	For concrete masonry blocks and for applications other than for structural concrete (*first revision*)
(Part 2) : 2018	Sintered fly ash coarse aggregate (*first revision*)
IS 10086 : 2021	Specification for moulds for use in tests of cement and concrete (*first revision*)
IS 18657	Representation of results of particle size analysis:
(Part 1) : 2024/ ISO 9276-1 : 1998	Graphical representation
(Part 2) : 2024/ ISO 9276-2 : 2014	Calculation of average particle sizes/diameters and moments from particle size distributions
(Part 3) : 2024/ ISO 9276-3 : 2008	Adjustment of an experimental curve to a reference model
(Part 4) : 2024/ ISO 9276-4 : 2001	Characterization of a classification process
(Part 5) : 2024/ ISO 9276-5 : 2005	Methods of calculation relating to particle size analyses using logarithmic normal probability distribution
(Part 6) : 2024/ ISO 9276-6 : 2008	Descriptive and quantitative representation of particle shape and morphology

b) Cement

IS No.	*Title*
IS 269 : 2015	Ordinary portland cement — Specification (*sixth revision*)
IS 455 : 2015	Portland slag cement — Specification (*fifth revision*)

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IS 1489	Portland pozzolana cement — Specification:
(Part 1) : 2015	Flyash based (*fourth revision*)
(Part 2) : 2015	Calcined clay based (*fourth revision*)
IS 3466 : 2026	Masonry cement — Specification (*third revision*)
IS 6452 : 1989	Specification for high alumina cement for structural use (*first revision*)
IS 6909 : 2026	Supersulphated cement — Specification (*second revision*)
IS 8041 : 2026	Rapid hardening portland cement — Specification (*third revision*)
IS 8042 : 2015	White portland cement — Specification (*third revision*)
IS 8043 : 2026	Hydrophobic portland cement — Specification (*third revision*)
IS 12330 : 1988	Specification for sulphate resisting portland cement
IS 12600 : 1989	Portland cement, low heat — Specification
IS 16415 : 2015	Composite cement — Specification
IS 16993 : 2018	Microfine ordinary portland cement — Specification
IS 17908 : 2023	Fibre cement sheet — Lightweight concrete sandwich panels specification
IS 18189 : 2023	Portland calcined clay limestone cement — Specification

c) Supplementary Cementitious Materials (Mineral Admixtures and Pozzolanas)

IS No.	*Title*
IS 1344 : 1981	Specification for calcined clay pozzolana (*second revision*)
IS 1727 : 1967	Methods of test for pozzolanic materials (*first revision*)
IS 3812	Pulverized fuel ash — Specification:
(Part 1) : 2013	For use as pozzolana in cement, cement mortar and concrete (*third revision*)
(Part 2) : 2013	For use as admixture in cement mortar and concrete (*third revision*)
IS 6491 : 1972	Method of sampling of flyash
IS 12089 : 1987	Specification for granulated slag for manufacture of portland slag cement
IS 12870 : 1989	Methods of sampling calcined clay pozzolana
IS 15388 : 2003	Specification for silica fume
IS 16354 : 2015	Metakaolin for use in cement, cement mortar and concrete — Specification
IS 16714 : 2018	Ground granulated blast furnace slag for use in cement mortar and concrete — Specification
IS 16715 : 2018	Ultrafine ground granulated blast furnace slag — Specification
IS 19058 : 2024	Ultrafine fly ash — Specification

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d) Chemical Admixtures

IS No.	*Title*
IS 6925 : 1973	Methods of test for determination of water soluble chlorides in concrete admixtures
IS 9103 : 1999	Specification for admixtures for concrete (*first revision*)

e) Concrete

IS No.	*Title*
IS 456 : 2000	Code of practice for plain and reinforced concrete (*fourth revision*)
IS 1343 : 2012	Code of practice for prestressed concrete (*second revision*)
IS 4926 : 2003	Code of practice for ready-mixed concrete (*second revision*)

f) Cement and Concrete Sampling and Methods of Test

IS No.	*Title*
IS 516	Hardened concrete methods of test:
(Part 1/Sec 1) : 2021	Testing of strength of hardened concrete, Section 1 Compressive, flexural and split tensile strength (*first revision*)
(Part 2)	Properties of hardened concrete other than strength,
(Sec 1) : 2018	Density of hardened concrete and depth of water penetration under pressure (*first revision*)
(Sec 2) : 2020	Initial surface absorption (*first revision*)
(Sec 3) : 2022	Oxygen permeability index (*first revision*)
(Sec 4) : 2021	Determination of the carbonation resistance by accelerated carbonation method (*first revision*)
(Part 4) : 2018	Sampling preparing and testing of concrete cores (*first revision*)
(Part 5)	Non-destructive testing of concrete,
(Sec 1) : 2018	Ultrasonic pulse velocity testing (*first revision*)
(Sec 2) : 2021	Half-cell potentials of uncoated reinforcing steel in concrete (*first revision*)
(Sec 3) : 2021	Carbonation depth test (*first revision*)
(Sec 4) : 2020	Rebound hammer test (*first revision*)
(Part 6) : 2020	Determination of drying shrinkage and moisture movement of concrete samples (*first revision*)
(Part 8/Sec 1) : 2020	Determination of modulus of elasticity, Section 1 Static modulus of elasticity and poissons ratio in compression (*first revision*)
(Part 11) : 2020	Determination of portland cement content of hardened hydraulic cement concrete (*first revision*)

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IS No.	*Title*
IS 1199	Fresh concrete — Methods of sampling testing and analysis:
(Part 1) : 2018	Sampling of fresh concrete (*first revision*)
(Part 2) : 2018	Determination of consistency of fresh concrete (*first revision*)
(Part 3) : 2018	Determination of density of fresh concrete (*first revision*)
(Part 4) : 2018	Determination of air content of fresh concrete (*first revision*)
(Part 5) : 2018	Making and curing of test specimens (*first revision*)
(Part 6) : 2018	Tests on fresh self-compacting concrete (*first revision*)
(Part 7) : 2018	Determination of setting time of concrete by penetration resistance (*first revision*)
IS 2770 (Part 1) : 1967	Methods of testing bond in reinforced concrete: Part 1 Pullout test
IS 3085 : 1965	Methods of test for permeability of cement mortar and concrete
IS 3535 : 1986	Methods of sampling hydraulic cement (*first revision*)
IS 4031	Methods of physical tests for hydraulic cement:
(Part 1) : 1996	Determination of fineness by dry sieving (*second revision*)
(Part 2) : 1999	Determination of fineness by specific surface by Blaine air permeability method (*second revision*)
(Part 3) : 1988	Determination of soundness (*first revision*)
(Part 4) : 1988	Determination of consistency of standard cement paste (*first revision*)
(Part 5) : 1988	Determination of initial and final setting times (*first revision*)
(Part 6) : 1988	Determination of compressive strength of hydraulic cement (other than masonry cement) (*first revision*)
(Part 7) : 1988	Determination of compressive strength of masonry cement (*first revision*)
(Part 8) : 1988	Determination of transverse and compressive strength of plastic mortar using prism (*first revision*)
(Part 9) : 1988	Determination of heat of hydration (*first revision*)
(Part 10) : 1988	Determination of drying shrinkage (*first revision*)
(Part 11) : 1988	Determination of density (*first revision*)
(Part 12) : 1988	Determination of air content of hydraulic cement mortar (*first revision*)
(Part 13) : 1988	Measurement of water retentivity of masonry cement (*first revision*)
(Part 14) : 1989	Determination of false set
(Part 15) : 1991	Determination of fineness by wet sieving
IS 4032 : 1985	Methods of chemical analysis for hydraulic cement (*first revision*)
IS 5816 : 1999	Method of test for splitting tensile strength of concrete (*first revision*)

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IS No.	*Title*
IS 9013 : 1978	Method of making, curing and determining compressive strength of accelerated cured concrete test specimens
IS 9284 : 1979	Method of test for abrasion resistance of concrete
IS 12423 : 1988	Methods for colorimetric analysis of hydraulic cement
IS 12803 : 1989	Methods of analysis of hydraulic cement by X-ray fluorescence spectrometer
IS 12813 : 1989	Method of analysis of hydraulic cement by atomic absorption spectrophotometer
IS 13311	Methods of non-destructive testing of concrete:
(Part 2) : 1992	Rebound hammer
IS 14959	Method of test for determination of water soluble and acid soluble chlorides in mortar and concrete
(Part 1) : 2001	Fresh mortar and concrete
(Part 2) : 2001	Hardened mortar and concrete
IS 17161 : 2020	Flexural strength and toughness parameters of fibre reinforced concrete — Method of test
IS 17725 : 2022	Precast concrete circular manhole — Specification

g) Treatment of Concrete Joints

IS No.	*Title*
IS 1834 : 1984	Specification for hot applied sealing compound for joint in concrete (*first revision*)
IS 1838	Specification for preformed fillers for expansion joint in concrete pavements and structures (non extruding and resilient type):
(Part 1) : 1983	Bitumen impregnated fibre (*first revision*)
(Part 2) : 1984	CNSL aldehyde resin and coconut pith
(Part 3) : 2011	Polymer based
IS 10566 : 1983	Methods of test for preformed fillers for expansion joints in concrete paving and structural construction
IS 11433	Specification for one grade polysulphide based joint sealant:
(Part 1) : 1985	General requirements
(Part 2) : 1986	Methods of test
IS 12118	Specification for two parts polysulphide based sealants:
(Part 1) : 1987	General requirements
(Part 2) : 1987	Methods of test

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h) Steel Reinforcement for Concrete

IS No.	*Title*
IS 432	Specification for mild steel and medium tensile steel bars and hard drawn steel wire for concrete reinforcement:
(Part 1) : 1982	Mild steel and medium tensile steel bars (*third revision*)
(Part 2) : 1982	Hard drawn steel wire (*third revision*)
IS 1566 : 1982	Specification for hard drawn steel wire fabric for concrete reinforcement (*second revision*)
IS 1599 : 2023/ ISO 7438 : 2020	Metallic materials — Bend test (*fifth revision*)
IS 1608	Metallic materials — Tensile testing:
(Part 1) : 2022/ ISO 6892-1 : 2019	Method of test at room temperature (*fifth revision*)
(Part 2) : 2020/ ISO 6892-2 : 2018	Method of test at elevated temperature (*fourth revision*)
(Part 3) : 2018/ ISO 6892-3 : 2015	Method of test at low temperature
IS 1785	Specification for plain hard drawn steel wire for pre-stressed concrete:
(Part 1) : 1983	Cold drawn stress-relieved wire (*second revision*)
(Part 2) : 1983	As drawn wire (*first revision*)
IS 1786 : 2008	High strength deformed steel bars and wires for concrete reinforcement Specification (*fourth revision*)
IS 2090 : 1983	Specification for high tensile steel bars used in prestressed concrete (*first revision*)
IS 6003 : 2010	Indented wire for prestressed concrete — Specification (*second revision*)
IS 6006 : 2014	Uncoated stress-relieved strand for prestressed concrete — Specification (*second revision*)
IS 9417 : 2018	Welding of high strength steel bars for reinforced concrete construction — Recommendations (*second revision*)
IS 10790	Methods of sampling of steel for prestressed and reinforced concrete:
(Part 1) : 1984	Prestressing steel
(Part 2) : 1984	Reinforcing steel
IS 12594 : 1988	Hot-dip zinc coating on structural steel bars for concrete reinforcement — Specification
IS 13620 : 1993	Specification for fusion bonded epoxy coated reinforcing bars
IS 14268 : 2022	Uncoated stress relieved low relaxation seven-wire ply strand for prestressed concrete — Specification (*second revision*)

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IS No.	*Title*
IS 16172 : 2023	Reinforcement couplers for mechanical splices of steel bars in concrete — Specification (*first revision*)
IS 16644 : 2018	Stress-Relieved low relaxation steel wire for prestressed concrete — Specification
IS 16651 : 2017	High strength deformed stainless steel bars and wires for concrete reinforcement — Specification
IS 18255 : 2023	Fibre-Reinforced Polymer FRP bars for concrete reinforcement — Methods of test
IS 18256 : 2023	Solid round glass fibre reinforced polymer GFRP bars for concrete reinforcement — Specification
IS 19068 : 2025	Continuous Hot-dip galvanized steel bars for concrete reinforcement — Specification

9 COMPOSITE MATRIX PRODUCTS

a) Cement Matrix Products

i) *Precast Concrete Products*

IS No.	*Title*
IS 2174 : 1962	Specification for reinforced concrete dust bins
IS 2185	Specification for concrete masonry units:
(Part 1) : 2005	Hollow and solid concrete blocks (*third revision*)
(Part 2) : 1983	Hollow and solid lightweight concrete blocks (*first revision*)
(Part 3) : 1984	Autoclaved cellular (aerated) concrete blocks (*first revision*)
(Part 4) : 2008	Preformed foam cellular concrete blocks
IS 4996 : 1984	Specification for reinforced concrete fence posts (*first revision*)
IS 5751 : 1984	Specification for precast concrete coping blocks (*first revision*)
IS 5758 : 2020	Precast concrete kerbs channels edging quadrants and other associated fittings — Specification (*second revision*)
IS 5820 : 2024	Precast concrete cable covers — Specification (*first revision*)
IS 6072 : 2023	Autoclaved reinforced cellular concrete wall slabs — Specification (*first revision*)
IS 6073 : 2006	Specification for autoclaved reinforced cellular concrete floor and roof slabs (*first revision*)
IS 6441	Methods of test for autoclaved cellular concrete products:
(Part 1) : 1972	Determination of unit weight or bulk density and moisture content
(Part 2) : 1972	Determination of drying shrinkage

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IS No.	*Title*
(Part 4) : 1972	Corrosion protection of steel reinforcement in autoclaved cellular concrete
(Part 5) : 1972	Determination of compressive strength
(Part 6) : 1973	Strength, deformation and cracking of flexural members subject to bending-short duration loading test
(Part 7) : 1973	Strength, deformation and cracking of flexural members subject to bending-sustained loading test
(Part 8) : 1973	Loading tests for flexural members in diagonal tension
(Part 9) : 1973	Jointing of autoclaved cellular concrete elements
IS 6523 : 1983	Specification for precast reinforced concrete door and window frames (*first revision*)
IS 9375 : 1979	Specification for precast reinforced concrete plant guards
IS 9872 : 1981	Specification for precast concrete septic tanks
IS 9893 : 1981	Specification for precast concrete blocks for lintels and sills
IS 12440 : 1988	Specification for precast concrete stone masonry blocks
IS 12592 : 2002	Specification for precast concrete manhole covers and frames (*first revision*)
IS 13356 : 1992	Specification for precast ferrocement water tanks (250 litres to 10 000 litres capacity)
IS 13990 : 1994	Specification for precast reinforced concrete planks and joists for flooring and roofing
IS 14143 : 1994	Specification for prefabricated brick panel and partially precast concrete joist for flooring and roofing
IS 14201 : 1994	Specification for precast reinforced concrete channel unit for construction of floors and roofs
IS 14241 : 1995	Specification for precast L-Panel units for roofing
IS 18661 : 2024	Precast concrete grating — Specification
IS 18889 : 2024	Precast concrete paving flags — Specification
IS 18894 : 2024	Precast concrete paving grids and grass pavers — Specification

ii) *Asbestos Fibre Cement Products*

IS No.	*Title*
IS 459 : 1992	Specification for corrugated and semi-corrugated asbestos cement sheets (*third revision*)
IS 1592 : 2003	Specification for asbestos cement pressure pipes and joints (*fourth revision*)
IS 1626	Specification for asbestos cement building pipes and pipe fittings, gutters and gutter fittings and roofing fittings:

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IS No.	*Title*
(Part 1) : 1994	Pipes and pipe fittings (*second revision*)
(Part 2) : 1994	Gutters and gutter fittings (*second revision*)
(Part 3) : 1994	Roofing fittings (*second revision*)
IS 2096 : 1992	Specification for asbestos cement flat sheets (*first revision*)
IS 2098 : 1997	Specification for asbestos cement building boards (*first revision*)
IS 5913 : 2003	Methods of test for asbestos cement products (*second revision*)
IS 6908 : 1991	Specification for asbestos cement pipes and fittings for sewerage and drainage (*first revision*)
IS 7639 : 1975	Method of sampling asbestos cement products
IS 9627 : 1980	Specification for asbestos cement pressure pipes (light duty)
IS 13000 : 1990	Silica-asbestos-cement flat sheets — Specification
IS 13008 : 1990	Specification for shallow corrugated asbestos cement sheets

iii) *Other Fibre Cement Products*

IS No.	*Title*
IS 14862 : 2000	Specification for fibre cement flat sheets
IS 14871 : 2000	Specification for products in fibre reinforced cement long corrugated or asymmetrical section sheets and fittings for roofing and cladding

iv) *Concrete Pipes and Pipes Lined/Coated with Concrete or Mortar*

IS No.	*Title*
IS 458 : 2021	Precast concrete pipes with and without reinforcement (*fifth revision*)
IS 784 : 2019	Prestressed concrete pipes including specials — Specification (*third revision*)
IS 1916 : 2018	Steel cylinder pipes with concrete lining and coating — Specification (*second revision*)
IS 3597 : 1998	Methods of test for concrete pipes (*second revision*)
IS 4350 : 1967	Specification for concrete porous pipes for under drainage
IS 7319 : 1974	Specification for perforated concrete pipes
IS 7322 : 1985	Specification for specials for steel cylinder reinforced concrete pipes (*first revision*)
IS 15155 : 2020	Barwire wrapped steel cylinder pipes with mortar lining and coating including specials specification (*first revision*)

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b) Resin Matrix Products

IS No.	*Title*
IS 1998 : 1962	Methods of test for thermosetting synthetic resin bonded laminated sheets
IS 2036 : 1995	Specification for phenolic laminated sheets (*second revision*)
IS 2046	Decorative thermosetting synthetic resin bonded laminated sheets — Specification:
(Part 1) : 2025/ ISO 4586-1 : 2018	Introduction and general information (*third revision*)
(Part 2) : 2025/ ISO 4586-2 : 2018	Determination of properties (*third revision*)
(Part 3) : 2025/ ISO 4586-3 : 2018	Classification and specifications for laminates less than 2 mm thick and intended for bonding to supporting substrates (*third revision*)
(Part 4) : 2025 ISO 4586-4 : 2018	Classification and specifications for compact laminates of thickness 2 mm and greater (*third revision*)
(Part 5) : 2025 ISO 4586-5 : 2018	Classification and specifications for flooring grade laminates less than 2 mm thick intended for bonding to supporting substrates (*third revision*)
(Part 6) : 2025 ISO 4586-6 : 2018	Classification and specifications for exterior-grade compact laminates of thickness 2 mm and greater (*third revision*)
(Part 7) : 2025 ISO 4586-7 : 2018	Classification and specifications for design laminates (*third revision*)
(Part 8) : 2025 ISO 4586-8 : 2018	Classification and specifications for alternative core laminates (*third revision*)

10 CONDUCTORS AND CABLES

IS No.	*Title*
IS 694 : 2010	Polyvinyl chloride insulated unsheathed and sheathed cables/cords with rigid and flexible conductor for rated voltages up to and including 450/750 V (*fourth revision*)
IS 1554	Specification for PVC insulated (heavy duty) electric cables:
(Part 1) : 1988	For working voltages up to and including 1 100 volts (*third revision*)
(Part 2) : 1988	For working voltages from 3.3 kV up to and including 11 kV (*second revision*)
IS 3961	Recommended current ratings for cables:
(Part 2) : 2017	PVC insulated and PVC sheathed heavy duty cables (*first revision*)
(Part 3) : 1968	Rubber insulated cables
(Part 5) : 1968	PVC insulated light duty cables
(Part 6) : 2016	Crosslinked polyethylene insulated PVC sheathed cables
(Part 7) : 2017	Crosslinked polyethylene insulated thermoplastic sheathed cables

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IS No.	*Title*
IS 4289	Specification for flexible cables for lifts and other flexible connections:
(Part 1) : 1984	Elastomer insulated cables (*first revision*)
(Part 2) : 2000	PVC insulated circular cables
IS 7098	Cross-linked polyethylene insulated sheathed cables — Specification
(Part 1) : 2025	For working voltage up to and including 1 100 V (*second revision*)
(Part 2) : 2011	For working voltages form 3.3 kV up to and including 33 kV (*second revision*)
(Part 3) : 1993	For working voltages from 66 kV upto and including 220 kV
IS 9968	Elastomer insulated cables — Specification:
(Part 1) : 2025	For working voltages up to and including 1 100 V (*second revision*)
(Part 2) : 2002	For working voltages from 3.3 kV up to and including 33 kV (*first revision*)
IS 10810	Methods of test for cables:
(Part 0) : 1984	General
(Part 1) : 1984	Annealing test for wires used in conductors
(Part 2) : 1984	Tensile test for aluminium wires
(Part 3) : 1984	Wrapping test for aluminium wires
(Part 4) : 1984	Persulphate test of conductor
(Part 5) : 1984	Conductor resistance test
(Part 6) : 1984	Thickness of thermoplastic and elastomeric insulation and sheath
(Part 7) : 1984	Tensile strength and elongation at break of thermoplastic and elastomeric insulation and sheath
(Part 8) : 1984	Breaking strength and elongation at break for impregnated paper insulation
(Part 9) : 1984	Tear resistance for paper insulation
(Part 10) : 1984	Loss of mass test
(Part 11) : 1984	Thermal ageing in air
(Part 12) : 1984	Shrinkage test
(Part 13) : 1984	Ozone resistance test
(Part 14) : 1984	Heat shock test
(Part 15) : 1984	Hot deformation test
(Part 16) : 1986	Accelerated ageing test by oxygen pressure method
(Part 17) : 1986	Tear resistance test for heavy duty sheath

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IS No.	*Title*
(Part 19) : 1984	Bleeding and blooming test
(Part 20) : 1984	Cold bend test
(Part 21) : 1984	Cold impact test
(Part 22) : 1984	Vicat softening point
(Part 23) : 1984	Melt-flow index
(Part 24) : 1984	Water soluble impurities test of insulating paper
(Part 25) : 1984	Conductivity of water extract test of insulating paper
(Part 26) : 1984	*p*H value of water extract test of insulating paper
(Part 27) : 1984	Ash content test of insulating paper
(Part 28) : 1984	Water absorption test (Electrical)
(Part 29) : 1984	Environmental stress cracking test
(Part 30) : 1984	Hot set test
(Part 31) : 1984	Oil resistance test
(Part 32) : 1984	Carbon content test for polyethylene
(Part 33) : 1984	Water absorption test (Gravimetric)
(Part 34) : 1984	Measurement of thickness of metallic sheath
(Part 35) : 1984	Determination of tin in lead alloy for sheathing
(Part 36) : 1984	Dimensions of armouring material
(Part 37) : 1984	Tensile strength and elongation at break of armouring materials
(Part 38) : 1984	Torsion test on galvanized steel wires for armouring
(Part 39) : 1984	Winding test on galvanized steel strips for armouring
(Part 40) : 1984	Uniformity of zinc coating on steel armour
(Part 41) : 1984	Mass of zinc coating on steel armour
(Part 42) : 1984	Resistivity test of armour wires and strips and conductance test of armour (wires/strips)
(Part 43) : 1984	Insulation resistance
(Part 44) : 1984	Spark test
(Part 45) : 1984	High voltage test
(Part 46) : 1984	Partial discharge test
(Part 47) : 1984	Impulse test

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IS No.	*Title*
(Part 48) : 1984	Dielectric power factor test
(Part 49) : 1984	Heating cycle test
(Part 50) : 1984	Bending test
(Part 51) : 1984	Dripping test
(Part 53) : 1984	Flammability test
(Part 54) : 1984	Static flexibility test
(Part 55) : 1986	Abrasion test
(Part 56) : 1987	Accelerated ageing by the air-pressure method
(Part 57) : 1987	Flexing test
(Part 58) : 1998	Oxygen index test
(Part 59) : 1988	Determination of the amount of halogen acid gas evolved during combustion of polymeric materials taken from cables
(Part 60) : 1988	Thermal stability of PVC insulation and sheath
(Part 61) : 1988	Flame retardant test
(Part 62) : 1993	Fire resistance test for bunched cables
(Part 63) : 1993	Smoke density of electric cables under fire conditions
(Part 64) : 2003	Measurement of temperature index
IS 12943 : 1990	Brass glands for PVC cables — Specification
IS 14763 : 2022	Conduit systems for cable management outside diameters of conduits for electrical installations and threads for conduits and fittings
IS 16205	Conduit systems for cable management:
(Part 1) : 2017	General requirements
(Part 21) : 2017	Particular requirements rigid conduit systems
(Part 22) : 2017	Particular requirements — Pliable conduit systems
(Part 23) : 2017	Particular requirements flexible conduit systems
(Part 24) : 2017	Particular requirements conduit systems buried under ground
IS 16783 : 2018	Cable cleats for electrical installations
IS 17048 : 2018	Halogen free flame retardant HFFR cables for working voltages up to and including 1 100 Volts — Specification
IS 17293 : 2020	Electric cables for photovoltaic systems for rated voltage 1 500 V d. c.

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IS No.	*Title*
IS 17505 (Part 1) : 2021	Specification for thermosetting insulated fire survival cables for fixed installation having low emission of smoke and corrosive gases when affected by fire for working voltages upto and including 1 100 V a.c. and 1 500 V d.c.: Part 1 Requirements for armoured cables
IS 18833 : 2024/ IEC 62895 : 2017	High voltage direct current (HVDC) power transmission — Cables with extruded insulation and their accessories for rated voltages up to 320 kV for land applications — Test methods and requirements
IS/IEC 60371-3-7 : 1995	Specification for insulating materials based on MICA: Part 3 Specifications for Individual materials, Section 7 Polyester film mica paper with an epoxy resin binder for single conductor taping
IS/IEC 60371-3-8 : 1995	Specification for insulating materials based on MICA: Part 3 Specification for individual materials, Section 8 Mica paper tapes for flame resistant security cables
IS/IEC 62275 : 2018	Cable management systems — Cable ties for electrical installations (*first revision*)

11 DOORS, WINDOWS AND VENTILATORS

a) Wooden Doors, Windows and Ventilators

IS No.	*Title*
IS 1003	Specification for timber panelled and glazed shutters
(Part 1) : 2003	Door shutters (*fourth revision*)
(Part 2) : 1994	Window and ventilator shutters (*third revision*)
IS 2191	Wooden flush door shutters (cellular, hollow and tubular core type) — Specification
(Part 1) : 2022	Plywood face panels (*fifth revision*)
(Part 2) : 2022	Particle board high density fibre board medium density fibre board and fibre hardboard face panels (*fourth revision*)
IS 2202	Wooden flush door shutters solid core type — Specification
(Part 1) : 2023	Plywood face panels (*seventh revision*)
(Part 2) : 2022	Particle board high density fibre board medium density fibre board and hardboard face panels (*fourth revision*)
IS 4020	Method of tests for door shutters:
(Part 1) : 1998	General (*third revision*)
(Part 2) : 1998	Measurement of dimensions and squareness (*third revision*)
(Part 3) : 1998	Measurement of general flatness (*third revision*)
(Part 4) : 1998	Local planeness test (*third revision*)
(Part 5) : 1998	Impact indentation test (*third revision*)

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IS No.	*Title*
(Part 6) : 1998	Flexure test (*third revision*)
(Part 7) : 1998	Edge loading test (*third revision*)
(Part 8) : 1998	Shock resistance test (*third revision*)
(Part 9) : 1998	Buckling resistance test (*third revision*)
(Part 10) : 1998	Slamming test (*third revision*)
(Part 11) : 1998	Misuse test (*third revision*)
(Part 12) : 1998	Varying humidity test (*third revision*)
(Part 13) : 1998	End immersion test (*third revision*)
(Part 14) : 1998	Knife test (*third revision*)
(Part 15) : 1998	Glue adhesion test (*third revision*)
(Part 16) : 1998	Screw withdrawal resistance test (*third revision*)
IS 4021 : 1995	Specification for timber door, window and ventilator frames (*third revision*)
IS 6198 : 1992	Specification for ledged, braced and battened timber door shutters (*second revision*)

b) Metal Doors, Windows Frames and Ventilators

IS No.	*Title*
IS 1038 : 1983	Specification for steel doors, windows and ventilators (*third revision*)
IS 1361 : 1978	Specification for steel windows for industrial buildings (*first revision*)
IS 1948 : 2024	Aluminium framed doors, windows and sliders — Specification (*first revision*)
IS 4351 : 2024	Steel Frames for Doors — Specification (*third revision*)
IS 6248 : 1979	Specification for metal rolling shutters and rolling grills (*first revision*)
IS 7452 : 1990	Specification for hot rolled steel sections for doors, windows and ventilators (*second revision*)
IS 10451 : 1983	Specification for steel sliding shutters (top hung type)
IS 10521 : 1983	Specification for collapsible gates
IS 18648 : 2024	Classification and performance requirements for doors, windows and sliders — Specification

c) Plastic Doors and Windows

IS No.	*Title*
IS 14856 : 2000	Specification for glass fibre reinforced plastic (GRP) panel type door shutters for internal use

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IS 15380 : 2023	Moulded high density fibre HDF panelled door shutters — Specification (*first revision*)
IS 15931 : 2012	Solid panel foam UPVC door shutters — Specification
IS 17953 : 2023	uPVC profiles for windows and doors — Specification
IS 18648 : 2024	Classification and performance requirements for doors, windows and sliders — Specification

d) Concrete Doors and Windows

IS No.	*Title*
IS 6523 : 1983	Specification for precast reinforced concrete door and window frames (*first revision*)

e) Other Composite Material Doors and Windows

IS No.	*Title*
IS 16073 : 2013	Bamboo-jute composite panel door shutter — Specification
IS 16074 : 2024	Steel flush door shutters — Specification (*first revision*)
IS 16096 : 2013	Bamboo-Jute composite hollow core door shutter — Specification

f) Fire Check Doors

IS No.	*Title*
IS 3614 : 2021	Fire doors and doorsets — Specification (*first revision*)

g) Mesh/Net for Mosquito/Vector Prevention

IS No.	*Title*
IS 1568 : 1970	Specification for wire cloth for general purposes (*first revision*)
IS 3150 : 1982	Specification for hexagonal wire netting for general purposes (*second revision*)
IS 11199 : 2024	Textiles — High density polyethylene (HDPE) monofilament twine door nets — Specification (*first revision*)

h) Doors and Windows — Method of tests

IS No.	*Title*
IS 4020	Method of tests for door shutters:
(Part 1) : 1998	General (*third revision*)
(Part 2) : 1998	Measurement of dimensions and squareness (*third revision*)
(Part 3) : 1998	Measurement of general flatness (*third revision*)
(Part 4) : 1998	Local planeness test (*third revision*)
(Part 5) : 1998	Part 5 Impact indentation test (*third revision*)

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IS No.	*Title*
(Part 6) : 1998	Flexure test (*third revision*)
(Part 7) : 1998	Edge loading test (*third revision*)
(Part 8) : 1998	Shock resistance test (*third revision*)
(Part 9) : 1998	Buckling resistance test (*third revision*)
(Part 10) : 1998	Slamming test (*third revision*)
(Part 11) : 1998	Misuse test (*third revision*)
(Part 12) : 1998	Varying humidity test (*third revision*)
(Part 13) : 1998	End immersion test (*third revision*)
(Part 14) : 1998	Knife test (*third revision*)
(Part 15) : 1998	Glue adhesion test (*third revision*)
(Part 16) : 1998	Screw withdrawal resistance test (*third revision*)
IS 12458 : 2019	Fire resistance of through penetration firestops — Method of test (*first revision*)
IS 17518 (Part 1) : 2022/ ISO 3008-1 : 2019	Fire resistance tests — Door and shutter assemblies: Part 1 General requirements
IS 17909 : 2022/ ISO 8274 : 2005	Windows and doors — Resistance to repeated opening and closing — Test method
IS 17910	Thermal performance of windows and doors — Determination of thermal transmittance by the hot-box method:
(Part 1) : 2022/ ISO 12567-1:2010	Complete windows and doors
(Part 2) : 2022/ ISO 12567-2 : 2005	Roof windows and other projecting windows
IS 17911 : 2022/ ISO 15099 : 2003	Thermal performance of windows, doors and shading devices — Detailed calculations
IS 17920 : 2022/ ISO 10077-2 : 2017	Thermal performance of windows, doors and shutters — Calculation of thermal transmittance — Numerical method for frames
IS 18268 : 2023/ ISO 9379 : 2005	Operating forces — Test methods — Doors
IS 18434 : 2023	Field measurement of air permeability and water penetration through installed curtain wall, windows, doors, sliders and skylights under static air pressure difference — Method of test
IS 18459 : 2024	Water penetration of curtain walls, windows, sliders, doors and skylights by uniform static air pressure difference — Method of test
IS 18473 : 2024	Wind resistance for curtain walls, windows, sliders, doors and skylights — Method of test

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IS No.	*Title*
IS 18647 : 2024	Water penetration of curtain walls, windows, sliders and doors by dynamic air pressure — Method of test
IS 18648 : 2024	Classification and performance requirements for doors, windows and sliders — Specification
IS 19006 : 2023/ISO 8248 : 1985	Windows and door height windows — Mechanical tests

12 ELECTRICAL WIRING, FITTINGS AND ACCESSORIES

IS No.	*Title*
IS 302 (Part 1) : 2024/ IEC 60335-1 : 2020	Household and similar electrical appliances — Safety: Part 1 General requirements (*seventh revision*)
IS 302	Safety of household and similar electrical appliances:
(Part 2)	Particular requirements,
(Sec 21) : 2024/ IEC 60335-2-21 : 2022	Storage water heaters (*third revision*)
(Sec 35) : 2017	Electric instantaneous water heaters (*second revision*)
(Sec 74) : 2024/ IEC 60335-2-74 : 2021	Portable immersion heaters
IS 302-2-80 : 2017	Household and similar electrical appliances — Safety: Part 2-80 Particular requirements for fans (*first revision*)
IS 369 : 2019	Household electric direct — Acting room heaters — Performance requirements (*fourth revision*)
IS 371 : 1999	Specification for ceiling roses (*third revision*)
IS 374 : 2019	Specification for electric ceiling type fans and regulators (*third revision*)
IS 418 : 2004	Specification for tungsten filament lamp for domestic and similar general lighting purposes (*fourth revision*)
IS 1258 : 2024/ IEC 61184 : 2017	Bayonet lampholders (*fifth revision*)
IS 1293 : 2019	Plugs and socket-outlets of rated voltage up to and including 250 volts and rated current up to and including 16 amperes — Specification (*fourth revision*)
IS 1554	PVC insulated (heavy duty) electric cables:
(Part 1) : 1988	For working voltages upto and including 1 100 V (*third revision*)
(Part 2) : 1988	For working voltages from 3.3 kV upto and including 11 kV (*second revision*)
IS 2082 : 2018	Stationary storage type electric water heaters — Specification (*fifth revision*)
IS 2086 : 1993	Specification for carriers and bases used in re-wirable type electric fuses up to 650 volts (*third revision*)
IS 2215 : 2006	Specification for starters for fluorescent lamps (*third revision*)
IS 2418	Tubular fluorescent lamps for general lighting service:

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IS No.	*Title*
(Part 1) : 2018	Safety requirements (*second revision*)
(Part 2) : 2018	Performance requirements (*second revision*)
(Part 3) : 1977	Dimensions of G-5 and G-13 bi-pin caps (*first revision*)
(Part 4) : 1977	Go and no-go gauges for G-5 and G-13 bi-pin caps (*first revision*)
IS 3323 : 1980	Specification for bi-pin lamp holders for tubular fluorescent lamps (*first revision*)
IS 3324 : 1982	Specification for holders for starters for tubular fluorescent lamps (*first revision*)
IS 3419 : 1988	Specification for fittings for rigid non-metallic conduits (*second revision*)
IS 3480 : 2024	Flexible steel conduits for electrical wiring — Specification (*first revision*)
IS 3724 : 2017	Cartridge type heating elements (non-embedded type) — Specification (*first revision*)
IS 3837 : 1976	Specification for accessories for rigid steel conduits for electrical wiring (*first revision*)
IS 3854 : 2023	Specification for switches for domestic and similar purposes (*third revision*)
IS 4159 : 2021	Mineral filled sheathed heating elements (*fourth revision*)
IS 4160 : 2005	Specification for interlocking switch socket outlets (*first revision*)
IS 4649 : 1968	Specification for adaptors for flexible steel conduits
IS 9385 (Part 1) : 2024/ IEC 60282-1 : 2020	High -voltage fuses: Part 1 Current-limiting fuses (*second revision*)
IS 9385 (Part 2) : 2018	High-voltage fuses: Part 2 Expulsion fuses (*first revision*)
IS 9385 (Part 6) : 2025/ IEC 60282-4 : 2020	High-voltage fuses: Part 6 Additional testing requirements for high-voltage expulsion fuses utilizing polymeric insulators
IS 9537	Specification for conduits for electrical installations:
(Part 1) : 1980	General requirements
(Part 2) : 1981	Rigid steel conduits
(Part 3) : 1983	Rigid plain conduits for insulating materials
(Part 4) : 1983	Pliable self-recovering conduits for insulating materials
(Part 5) : 2000	Pliable conduits of insulating materials
(Part 6) : 2000	Pliable conduits of metal or composite materials
(Part 8) : 2003	Rigid non threadable conduits of aluminium alloy
IS 9926 : 1981	Specification for fuse wires used in rewirable type electric fuses up to 650 volts

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IS No.	*Title*
IS 10027 : 2018	Composite units of air-break switches and rewirable type fuses for voltages not exceeding 650 V a.c. — Specification (*second revision*)
IS 10322	Specification for luminaires:
(Part 1) : 2014	General requirements and tests (*first revision*)
(Part 5)	Particular requirements,
(Sec 1) : 2012	General purpose luminaires (*first revision*)
(Sec 2) : 2012	Recessed luminaires (*first revision*)
(Sec 3) : 2012	Luminaires for road and street lighting (*first revision*)
(Sec 4) : 1987	Portable general purpose luminaires
(Sec 5) : 2013	Flood light (*first revision*)
(Sec 6) : 2013	Hand lamps
(Sec 7) : 2017/ IEC 60598- 2-20 : 2014	Lighting chains (*first revision*)
(Sec 8) : 2013	Emergency lighting
(Sec 9) : 2017/ IEC 60598-2-21 : 2014	Rope lights
IS 11037 : 2019	Electronic type fan regulators specification (*first revision*)
IS 12640	Residual current operated circuit-breakers without integral overcurrent protection for household and similar uses (RCCBs):
(Part 1) : 2024/ IEC 61008-1 : 2013	General rules (*third revision*)
(Part 2) : 2016/ IEC 61009-1 : 2012	General rules (*second revision*)
(Part 3) : 2018/ IEC 1008-2-1 : 1990	Applicability of the general rule to RCCB's functionally independent of line voltage
(Part 4) : 2018/ IEC 61008-2-2 : 1990	Applicability of the general rules to RCCB's functionally dependent on line voltage
IS 12776 : 2002	Specification for galvanized strand for earthing (*first revision*)
IS 13010 : 2002	AC watt-hour meters, Class 0.5, 1 and 2 (*first revision*)
IS 13779 : 2020	a.c. static watthour meters class 1 and 2-Specefication (*second revision*)
IS 13947 (Part 5/Sec 2) : 2004/ IEC 60947-5-2(1999)	Low-voltage switchgear and controlgear — Specification: Part 5 Control circuit devices and switching elements, Section 2 Proximity switches
IS 14614 : 2023	Residual current — operated protective devices (RCDs) for household and similar use — Electromagnetic compatibility (*first revision*)

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IS No.	*Title*
IS 14697 : 2021	a.c. Static transformer operated watthour meters (Class 0.2 S and 0.5 S) and Var-hour meters (Class 0.2 S, 0.5 S and 1 S) — Specification (*first revision*)
IS 14763 : 2022	Conduit systems for cable management outside diameters of conduits for electrical installations and threads for conduits and fittings (*first revision*)
IS 14768	Conduit fittings for electrical installations:
(Part 1) : 2000	General requirements
(Part 2) : 2003	Metal conduit fittings
IS 14772 : 2020	Boxes and enclosures for electrical accessories for household and similar fixed electrical installations general requirements (*first revision*)
IS 14897 : 2021	Glass bulb designation system for lamps (*first revision*)
IS 14927	Cable trunking systems and cable ducting systems for electrical installations:
(Part 1) : 2023	General requirements (*first revision*)
(Part 2/Sec 1) : 2023/ IEC 61084-2-1 : 2017	Cable trunking systems and cable ducting systems intended for mounting on walls and ceilings, Section 1 Particular requirements (*first revision*)
IS 15111	Self-Ballasted lamps for general lighting services:
(Part 1) : 2002	Safety requirements
(Part 2) : 2002	Performance requirements
IS 15368 : 2003	Cable reels for household and similar purposes
IS 15518 (Part 1) : 2004	Safety requirements for incandescent lamps: Part 1 Tungsten filament lamps for domestic and similar general lighting purposes
IS 15787 : 2025	Switched socket-outlets without interlock for fixed installations — Particular requirements (*first revision*)
IS 15968 : 2013	Ballasts for tubular fluorescent lamps — Performance requirements
IS 15974 : 2013	Auxiliaries for lamps — Starting devices (other than glow starters) — Performance requirements
IS 16102	Self-ballasted LED-lamps for general lighting services:
(Part 1) : 2026	Safety requirements (*first revision*)
(Part 2) : 2026	Performance requirements (*second revision*)
IS 16103	LED modules for general lighting:
(Part 1) : 2025/ IEC 62031 : 2018	Safety — Specifications (*first revision*)
(Part 2) : 2025/ IEC 62717 : 2019 CSV	Performance requirements *(first revision*)
IS 16148 : 2014/ IEC 61167 : 2011	Metal halide lamps — Performance specification

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IS No.	*Title*
IS 16444 (Part 2) : 2017	a.c. Static transformer operated watthour and var-hour smart meters, class 0.2 s, 0.5 s and 1.0 s: Part 2 Specification transformer operated smart meters
IS 16463 (Part 12) : 2026/ IEC 61643-12 : 2020	Low-voltage surge protective devices: Part 12 Surge protective devices connected to low-voltage power distribution systems — Selection and application principles (*first revision*)
IS 16614	Double-capped LED linear lamps:
(Part 1) : 2026	Safety requirements (*first revision*)
(Part 2) : 2022	Performance requirements
IS 17119 : 2019	Residual current devices with or without overcurrent protection for socket-outlets for household and similar uses
IS 17121 : 2019/IEC 62606 : 2017	General requirements for arc fault detection devices
IS 17122 : 2019	Electrical accessories — Circuit-Breakers and similar equipment for household use — Auxiliary contact units
IS 17345	Power track system:
(Part 1) : 2020	General requirement
(Part 21) : 2020	Particular requirements for power track systems intended for wall and ceiling mounting
IS 18751 : 2024	Polymer coated inner tanks for storage water heater — Specification
IS 18830 : 2024	Cord extension sets — Specifications
IS/IEC 60079	Explosive atmospheres:
(Part 0) : 2017	General requirements (*third revision*)
(Part 1) : 2014	Equipment protection by flameproof enclosures “d” (*first revision*)
(Part 7) : 2017	Equipment protection by increased safety “e” (*second revision*)
(Part 13) : 2017	Equipment protection by pressurized room “p” and artificially ventilated room “v” (*first revision*)
(Part 15) : 2017	Equipment protection by type of protection “n” (*second revision*)
(Part 19) : 2019	Equipment repair overhaul and reclamation (*second revision*)
(Part 25) : 2020	Intrinsically safe electrical systems (*second revision*)
(Part 26) : 2021	Equipment with separation elements or combined levels of protection (*second revision*)
(Part 31) : 2022	Equipment dust ignition protection by enclosure “t” (*second revision*)
(Part 39) : 2015	Intrinsically safe systems with electronically controlled spark duration limitation

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IS No.	*Title*
(Part 42) : 2019	Electrical safety devices for the control of potential ignition sources from ex-equipment
(Part 43) : 2017	Equipment in adverse service conditions
(Part 46) : 2017	Equipment assemblies
(Part 47) : 2021	Equipment Protection by 2-wire intrinsically safe ethernet concept (2-WISE)
IS/IEC 60127	Miniature fuses:
(Part 3) : 2015	Sub-miniature fuse-links
(Part 10) : 2001	User guide for miniature fuses
IS/IEC 60309	Plugs fixed or portable socket-outlets and appliance inlets for industrial purposes
(Part 1) : 2021	General requirements (*second revision*)
(Part 2) : 2021	Dimensional compatibility requirements for pin and contact-tube accessories (*second revision*)
IS/IEC 60320	Appliance couplers for household and similar general purposes
(Part 1) : 2021	General requirements (*first revision*)
(Part 2/Sec 3) : 2018	Particular requirement, Section 3 Appliance couplers with a degree of protection higher than IPX0 (*first revision*)
IS/IEC 60669 (Part 2/Sec 1) : 2021	Switches for household and similar fixed electrical installations: Part 2-1: Particular requirements electronic control devices (*first revision*)
IS/IEC 60794	Optical fibre cables:
(Part 1)	Generic specification,
(Sec 1) : 2015	General (*first revision*)
(Sec 2) : 2017	Basic optical cable test procedures — General guidance (*first revision*)
(Part 2) : 2017	Indoor cables, Sectional specification (*first revision*)
(Part 2/Sec 11) : 2020	Indoor cables, Section 11 Detailed specification for simplex and duplex cables for use in premises cabling (*first revision*)
(Part 4/Sec 20) : 2018	Sectional specification, Section 20 Aerial optical cables along electrical power lines family specification for ADSS all dielectric self-supported optical cables
IS/IEC 60898	Electrical accessories — Circuit-breakers for overcurrent protection for household and similar installations
(Part 1) : 2015	Circuit-breakers for a.c. operation (*first revision*)
(Part 2) : 2016	Circuit-breakers for a.c. and d.c. operation (*first revision*)
(Part 3) : 2019	Circuit-breakers for d.c. operation
IS/IEC 60947	Low-voltage switchgear and controlgear:

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IS No.	*Title*
(Part 1) : 2020	General rules (*second revision*)
(Part 2) : 2016	Circuit-breakers (*first revision*)
(Part 3) : 2020	Switches, disconnectors, switch-disconnectors and fuse-combination units (*first revision*)
(Part 4)	Contactors and motor-starters,
(Sec 1) : 2023	Electromechanical contactors and motor-starters (*third revision*)
(Sec 2) : 2020	Semiconductor motor controllers, starters and soft-starters (*second revision*)
(Sec 3) : 2020	Semiconductor controllers and semiconductor contactors for non-motor loads *(third revision*)
(Part 5)	Control circuit devices and switching elements,
(Sec 1) : 2024	Electromechanical control circuit devices (*third revision*)
(Sec 2) : 2019	Proximity switches (*first revision*)
(Sec 5) : 2016	Electrical emergency stop devices with mechanical latching function
(Part 6)	Multiple function equipment,
(Sec 1) : 2021	Transfer switching equipment (*first revision*)
(Sec 2) : 2020	Control and protective switching devices or equipment (CPS) (*first revision*)
(Part 7)	Ancillary equipment,
(Sec 1) : 2009	Terminal blocks for copper conductors
(Sec 2) : 2009	Protective conductor terminal blocks for copper conductors
IS/IEC 61058	Switches for appliances:
(Part 1)	Particular requirements,
(Sec 1) : 2016	Mechanical switches
(Sec 2) : 2016	Particular requirements for electronics switches
IS/IEC 61439	Low-voltage switchgear and controlgear assemblies:
(Part 1) : 2020	General rules (*first revision*)
(Part 2) : 2020	Power switchgear and controlgear assemblies (*first revision*)
IS/IEC 61757 : 2018	Fibre optic sensors — Generic specification
IS/IEC/TR 62655 : 2013	Tutorial and application guide for high-voltage fuses
IS/IEC 63008 : 2020	Household and similar electrical appliances accessibility of control elements, doors, lids, drawers and handles

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IS No.	*Title*
IS/ISO/IEC 80079	Explosive atmospheres:
(Part 20)	Material characteristics for gas and vapour classification,
(Sec 1) : 2017	Test methods and data
(Sec 2) : 2016	Combustible dusts test methods
(Part 36) : 2016	Non-electrical equipment for explosive atmospheres — Basic method and requirements
(Part 37) : 2016	Non-electrical equipment for explosive atmospheres — Non electrical type of protection constructional safety “c”, control of ignition source “b”, liquid immersion “k”

13 FILLERS, STOPPERS AND PUTTIES

IS No.	*Title*
IS 110 : 2017	Ready mixed paint brushing grey filler for enamels for use over primers — Specification (*third revision*)
IS 419 : 1967	Specification for putty for use on window frames (*first revision*)
IS 419 : 2023	Putty for use on window frames — Specification (*second revision*)
IS 423 : 1961	Specification for plastic wood, for joiner’s filler (*revised*)
IS 3709 : 1966	Specification for mastic cement for bedding of metal windows
IS 7164 : 1973	Specification for stopper
IS 13184 : 1991	Specification for mastic filler, epoxy based

14 FLOOR COVERING, ROOFING AND OTHER FINISHES

a) Concrete Flooring

IS No.	*Title*
IS 1237 : 2012	Specification for cement concrete flooring tiles (*second revision*)
IS 6073 : 2006	Autoclaved reinforced cellular concrete floor and roof slabs — Specification (*first revision*)
IS 13801 : 2013	Chequered cement concrete tiles — Specification (*first revision*)
IS 13990 : 1994	Specification for precast reinforced concrete planks and joists for flooring and roofing
IS 14201 : 1994	Precast reinforced concrete channel units for construction of floors and roofs — Specification
IS 15658 : 2021	Concrete paving blocks — Specification (*first revision*)

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b) Flooring Compositions

IS No.	*Title*
IS 657 : 1982	Specification for materials for use in the manufacture of magnesium oxychloride flooring compositions (*second revision*)
IS 9162 : 1979	Methods of tests for epoxy resin, hardeners and epoxy resin composition for floor topping
IS 9197 : 1979	Specification for epoxy resin, hardness and epoxy resin compositions for floor topping
IS 10132 : 1982	Method of test for materials for use in the preparation of magnesium oxychloride flooring composition

c) Linoleum Flooring

IS No.	*Title*
IS 653 : 1992	Specification for linoleum sheets and tiles (*third revision*)
IS 9704 : 1980	Methods of tests for linoleum sheets and tiles

d) Rubber Flooring

IS No.	*Title*
IS 809 : 1992	Specification for rubber flooring materials for general purposes (*second revision*)

e) Stone Flooring

IS No.	*Title*
IS 1128 : 1974	Specification for limestone (slab and tiles) (*first revision*)
IS 1130 : 1969	Specification for marble (blocks, slabs and tiles)
IS 3316 : 1974	Specification for structural granite (*first revision*)
IS 3622 : 1977	Specification for sand stone (slabs and tiles) (*first revision*)
IS 14223 (Part 1) : 2023	Polished building stones specification: Part 1 Granite and similar stones (*first revision*)

f) Bituminous Flooring

IS No.	*Title*
IS 1195 : 2002	Specification for bitumen mastic for flooring (*third revision*)
IS 5317 : 2002	Specification for pitch-mastic for bridge decking and roads (*second revision*)
IS 8374 : 1977	Specification for bitumen mastic, anti-static and electrically conducting grade
IS 9510 : 1980	Specification for bitumen mastic acid resisting grade
IS 13026 : 1991	Specification for bitumen mastic for flooring for industries handling LPG and other light hydrocarbon products

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IS 15194 : 2002	Specification for pitch-mastic flooring for industries handling heavy hydrocarbon products like kerosene, diesel and furnace oil

g) Plastic Flooring

IS No.	*Title*
IS 3461 : 1980	Specification for PVC asbestos floor tiles (*first revision*)
IS 3462 : 1986	Specification for unbacked flexible PVC flooring (*second revision*)
IS 3464 : 1986	Methods of test for plastic flooring and wall tiles (*second revision*)

h) Ceramic/Vitreous Flooring and Wall Finishing

IS No.	*Title*
IS 2333 : 2025	Plaster of Paris for ceramic and glass industry — Specification (*third revision*)
IS 4457 : 2007	Ceramic unglazed vitreous acid resisting tile — Specification (*second revision*)
IS 13630	Ceramic tiles methods of test sampling and basis for acceptance:
(Part 1) : 2019	Determination of dimensions and surface quality (*second revision*)
(Part 2) : 2019	Determination of water absorption and bulk density (*second revision*)
(Part 3) : 2019	Determination of moisture expansion using boiling water (*second revision*)
(Part 4) : 2019	Determination of linear thermal expansion (*second revision*)
(Part 5) : 2019	Determination of resistance to thermal shock (*second revision*)
(Part 6) : 2019	Determination of modulus of rupture and breaking strength (*second revision*)
(Part 7) : 2019	Determination of stain and chemical resistance of unglazed tiles (s*econd revision*)
(Part 8) : 2019	Determination of stain and chemical resistance of glazed tiles (*second revision*)
(Part 9) : 2019	Determination of crazing resistance of glazed tiles (*second revision*)
(Part 10) : 2019	Determination of frost resistance (*second revision*)
(Part 11) : 2019	Determination of resistance to surface abrasion of glazed tiles (*second revision*)
(Part 12) : 2019	Determination of resistance to deep abrasion of unglazed tiles (*second revision*)
(Part 13) : 2019	Determination of scratch hardness of surface according to Mohs scale (*second revision*)
(Part 14) : 2019	Determination of impact resistance by measurement of coefficient of restitution (*first revision*)
(Part 15) : 2019	Sampling and basis for acceptance (*first revision*)
(Part 16) : 2019	Determination of lead and cadmium given off by glazed tiles

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IS No.	*Title*
IS 13712 : 2019	Ceramic tiles definitions classifications characteristics and marking (*second revision*)
IS 15622 : 2017	Pressed ceramic tiles — Specification (*first revision*)
IS 17933 : 2022	Fine ceramics (advanced ceramics, advanced technical ceramics) — Test method for flexural strength of monolithic ceramics at room temperature
IS 17935 : 2022	Fine ceramics (advanced ceramics, advanced technical ceramics) — Determination of density and apparent porosity
IS 17975 : 2022	Fine ceramics (advanced ceramics, advanced technical ceramics) — test method for fracture resistance of silicon nitride materials for rolling bearing balls at room temperature by indentation fracture (IF) method
IS 17981 : 2022	Fine ceramics (advanced ceramics, advanced technical ceramics) — Test method for elastic moduli of monolithic ceramics at room temperature by sonic resonance
IS 17991 : 2022	Fine ceramics (advanced ceramics, advanced technical ceramics) — Test method for hardness of monolithic ceramics at room temperature
IS 18221 : 2023	Fine ceramics (Advanced ceramics, advanced technical ceramics) — Test method for linear thermal expansion of monolithic ceramics by push-rod technique

j) Clay Flooring

IS No.	*Title*
IS 1478 : 2023	Clay flooring tiles specification (*third revision*)
IS 3951	Hollow clay tiles for floors and roofs specification:
(Part 1) : 2023	Filler type (*third revision*)
(Part 2) : 2023	Structural type (*third revision*)

k) Roofing

IS No.	*Title*
IS 277 : 2018	Galvanized steel strips and sheets plain and corrugated — Specification (*seventh revision*)
IS 459 : 1992	Specification for corrugated and semi-corrugated asbestos cement sheets (*third revision*)
IS 654 : 2023	Clay roofing tiles mangalore pattern specification (*fourth revision*)
IS 1464 : 2024	Clay ridge and ceiling tiles — Specification (*third revision*)
IS 2690	Burnt clay flat terracing tiles — Specification:
(Part 1) : 2023	Machine made (*third revision*)
(Part 2) : 2023	Handmade (*third revision*)

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IS No.	*Title*
IS 3951	Hollow clay tiles for floors and roofs specification:
(Part 1) : 2023	Filler type (*third revision*)
(Part 2) : 2023	Structural type (*third revision*)
IS 6073 : 2006	Autoclaved reinforced cellular concrete floor and roof slabs (*first revision*)
IS 6250 : 1981	Specification for roofing slate tiles (*first revision*)
IS 10388 : 1982	Specification for corrugated coir wood wool cement roofing sheets
IS 12583 : 1988	Specification for corrugated bitumen roofing sheets
IS 12866 : 2021	Plastic translucent sheets made from thermo-setting polyester resin (glass fibre reinforced) — Specification (*first revision*)
IS 13317 : 2023	Clay roofing country tiles half round and flat tiles Specification (*first revision*)
IS 13990 : 1994	Specification for precast reinforced concrete planks and joists for flooring and roofing
IS 14201 : 1994	Precast reinforced concrete channel units for construction of floors and roofs — Specification
IS 14241 : 1995	Specification for precast L-Panel units for roofing
IS 14246 : 2024	Continuously pre-painted galvanized steel sheets and strips — Specification (*second revision*)
IS 15965 : 2012	Pre-Painted aluminium zinc alloy metallic coated steel strip and sheet (plain)
IS 18385 : 2023	Hot-Dip galvanized/galvannealed steel sheet, plate and strip for automotive applications — Specification

m) Other Floorings and Roofings

IS No.	*Title*
IS 4456	Methods of test for chemical resistant mortars:
(Part 1) : 2026	Silicate type and resin type (*first revision*)
(Part 2) : 2026	Sulphur type (*first revision*)
IS 4457 : 2007	Ceramic unglazed vitreous acid resisting tile — Specification (*second revision*)
IS 4832	Chemical resistant mortars specification:
(Part 1) : 2023	Silicate type (*first revision*)
(Part 2) : 2023	Resin type (*first revision*)
(Part 3) : 2023	Sulphur type (*first revision*)
IS 4860 : 1968	Specification for acid resistant bricks

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IS No.	*Title*
IS 14862 : 2000	Specification for fibre cement flat sheets
IS 14143 : 1994	Specification for prefabricated brick panel and partially precast concrete joist for flooring and roofing
IS 14871 : 2000	Products in fibre reinforced cement — Long corrugated or asymmetrical section sheets and fittings for roofing and cladding — Specification
IS 18433	Measurement of surface frictional properties — Methods of test:
(Part 1) : 2023	Pendulum test
(Part 2) : 2023	Ramp test

n) Wall Coverings/Finishing

IS No.	*Title*
IS 1542 : 1992	Specification for sand for plaster (*second revision*)
IS 3952 : 2013	Burnt clay hollow bricks and blocks for walls and partitions — Specification (*third revision*)
IS 4456	Methods of test for chemical resistant mortars:
(Part 1) : 1967	Silicate type and resin type
(Part 2) : 1967	Sulphur type
IS 4832	Chemical resistant mortars specification:
(Part 1) : 2023	Silicate type (*first revision*)
(Part 2) : 2023	Resin type (*first revision*)
(Part 3) : 2023	Sulphur type (*first revision*)
IS 15418 : 2003	Wall coverings in roll form for finished wall papers, wall vinyls and plastic wall coverings — Specification

15 GLASS

IS No.	*Title*
IS 2553 (Part 1) : 2018	Safety glass — Specification: Part 1 Architectural building and general uses (*fourth revision*)
IS 2835 : 1987	Specification for flat transparent sheet glass (*third revision*)
IS 3438 : 2023	Silvered glass mirrors for general purposes specification (*third revision*)
IS 5437 : 2024	Rolled glass patterned extra clear patterned wired and wired-patterned glass specification (*second revision*)
IS 9154 : 2020	Method for determination of alkali resistance of glass (*first revision*)
IS 12869	Methods for determination of viscosity and viscometric fixed points of glass:
(Part 1) : 2022	Standard test methods (*first revision*)

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IS No.	*Title*
(Part 2) : 2022	Determination of softening point (*first revision*)
IS 14900 : 2018	Transparent float glass — Specification (*first revision*)
IS 16526 : 2017	Atactic polypropylene (APP) modified bituminous waterproofing and damp-proofing membrane with glass fibre reinforcement — Specification
IS 16945 : 2018	Fire resistance test for glass walls
IS 16947 : 2018	Fire resistance tests for doors with glass panes, openable glass windows and sliding glass doors
IS 16978	Glass in building — Forced-entry security glazing:
(Part 1) : 2022	Test and classification by repetitive ball drop (*first revision*)
(Part 2) : 2018	Test and classification by repetitive impact of a hammer and axe at room temperature
(Part 3) : 2018	Test and classification by manual attack
(Part 4) : 2018	Test and classification by pendulum impact under thermally and fire stressed conditions
IS 16982 : 2018	Heat strengthened glass — Specification
IS 17004 : 2018	Testing methods for processed glass
IS 17346 : 2020	Insulating glazing unit — Specification
IS 18222 : 2023	Glass-Resistance to attack by hydrochloric acid at 100 °C — Flame emission or flame atomic absorption spectrometric method
IS 18518 : 2023	Bullet resistant security glass — Quality and performance requirements
IS 19328 : 2025	Fire resistant glass — Specification
IS 19331 : 2025	Tinted float glass — Specification

16 GYPSUM BASED MATERIALS

IS No.	*Title*
IS 2095	Specification for gypsum plaster boards:
(Part 1) : 2023	Plain gypsum plaster boards (*fourth revision*)
(Part 2) : 2022	Coated laminated gypsum plaster boards (*third revision*)
(Part 3) : 2022	Reinforced gypsum plaster boards tiles cornices and mouldings (*third revision*)
IS 2542	Gypsum plaster concrete and products methods of test:
(Part 1)	Plaster and concrete,
(Sec 1) : 2023	Determination of normal consistency of gypsum plaster (*second revision*)

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IS No.	*Title*
(Sec 2) : 2023	Determination of normal consistency of gypsum concrete (*second revision*)
(Sec 3) : 2023	Determination of setting time of plaster and concrete (*second revision*)
(Sec 4) : 2023	Determination of transverse strength of gypsum plaster (*second revision*)
(Sec 5) : 2023	Determination of compressive strength and dry set density of gypsum plaster (*second revision*)
(Sec 6) : 2023	Determination of soundness of gypsum plaster (*second revision*)
(Sec 7) : 2023	Determination of impact resistance of gypsum plaster by dropping ball test (*second revision*)
(Sec 8) : 2023	Determination of mass of coarse particles (*second revision*)
(Sec 9) : 2023	Determination of expansion of gypsum plaster (*second revision*)
(Sec 10) : 2023	Determination of sand in set gypsum plaster (*second revision*)
(Sec 11) : 2023	Determination of wood fibre content in wood fibre gypsum plaster (*second revision*)
(Sec 12) : 2023	Determination of dry bulk density (*second revision*)
(Sec 13) : 2023	Determination of free water (*second revision*)
(Sec 14) : 2023	Determination of fineness (*second revision*)
(Part 2)	Gypsum products,
(Sec 1) : 1981	Measurement of dimensions (*first revision*)
(Sec 2) : 1981	Determination of mass (*first revision*)
(Sec 3) : 1981	Determination of mass and thickness of paper surfacing (*first revision*)
(Sec 4) : 1981	Transverse strength (*first revision*)
(Sec 5) : 1981	Compressive strength (*first revision*)
(Sec 6) : 1981	Gypsum products, Section 6 Water absorption (*first revision*)
(Sec 7) : 1981	Moisture content (*first revision*)
(Sec 8) : 1981	Nail retention of precast reinforced gypsum slabs (*first revision*)
IS 2547	Specification for gypsum building plaster:
(Part 1) : 1976	Excluding premixed lightweight plaster (*first revision*)
(Part 2) : 1976	Premixed lightweight plaster (*first revision*)
IS 2849 : 1983	Specification for non-load bearing gypsum partition blocks (solid and hollow types) (*first revision*)
IS 8272 : 1984	Specification for gypsum plaster for use in the manufacture of fibrous plasterboards (*first revision*)

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IS No.	*Title*
IS 9498 : 1980	Specification for inorganic aggregates for use in gypsum plaster
IS 12679 : 2023	By-Product gypsum for construction — Specification (*second revision*)
IS 17400 : 2021	Glass fibre reinforced gypsum building panels – Specification
IS 18387 : 2023	Gypsum ceiling tiles — Specification

17 MORTAR (Including Sand for Mortar)

IS No.	*Title*
IS 2116 : 1980	Sand for masonry mortars — Specification (*first revision*)
IS 2250 : 1981	Code of practice for preparation and use of masonry mortars (*first revision*)
IS 3085 : 1965	Method of test for permeability of cement mortar and concrete
IS 4098 : 1983	Specification for lime-pozzolana mixture (*first revision*)
IS 4456	Methods of test for chemical resistant mortars:
(Part 1) : 1967	Silicate type and resin type
(Part 2) : 1967	Sulphur type
IS 4832	Chemical resistant mortars specification:
(Part 1) : 2023	Silicate type (*first revision*)
(Part 2) : 2023	Resin type (*first revision*)
(Part 3) : 2023	Sulphur type (*first revision*)
IS 13077 : 1991	Guide for preparation and use of mud mortar in masonry
IS 14959	Method of test determination of water soluble and acid soluble chlorides in mortar and concrete
(Part 1) : 2001	Fresh mortar and concrete
(Part 2) : 2001	Hardened mortar and concrete

18 PAINTS AND ALLIED PRODUCTS

a) Water Based Paints and Pigments

IS No.	*Title*
IS 427 : 2013	Distemper, dry, colour as required — Specification (*third revision*)
IS 428 : 2013	Washable distemper — Specification (*third revision*)
IS 5410 : 2013	Specification for cement paint (*second revision*)
IS 15489 : 2024	Paint, plastic emulsion — Specification (*second revision*)

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b) Ready Mixed Paints, Enamels and Powder Coatings

IS No.	*Title*
IS 101	Methods of sampling and test for paints, varnishes and related products:
(Part 1)	Tests on liquid paints general and physical,
(Sec 1) : 2023	Sampling (*fourth revision*)
(Sec 2) : 2023/ ISO 1513 : 2010	Preliminary examination and preparation of samples for testing (*fourth revision*)
(Sec 3) : 1986	Preparation of panels (*third revision*)
(Sec 4) : 2024	Brushing test (*fourth revision*)
(Sec 5) : 2024	Consistency (*fourth revision*)
(Sec 5) : 2024	Consistency (*fourth revision*)
(Sec 6) : 1987	Flash point (*third revision*)
(Sec 7) : 2020/ ISO 2811-1	Mass per 10 litres determination of density pycnometer method (*fourth revision*)
(Sec 8) : 2022/ ISO 787-9 : 2019	Pigments and extenders determination of *p*H value of an aqueous suspension (*first revision*)
(Part 2)	Test on liquid paints chemical examination,
(Sec 1) : 2018	Water content (*fourth revision*)
(Sec 2) : 2025	Determination of non-volatile matter content (*fourth revision*)
(Sec 3) : 2015/ ISO 11890-1	Determination of volatile organic compound VOC content — Difference method
(Sec 4) : 2022/ ISO 11890-2 : 2020	Determination of volatile organic compound VOC and or semi volatile organic compounds SVOC content gas-chromatographic (*first revision*)
(Sec 5) : 2017/ ISO 17895 : 2005	Determination of volatile organic compound content of low-VOC emulsion paints (In-Can VOC)
(Part 3)	Tests on paint film formation,
(Sec 1) : 1986	Drying time (*third revision*)
(Sec 2) : 2025/ ISO 2808 : 2019	Determination of film thickness (*fourth revision*)
(Sec 4) : 1987	Finish (*third revision*)
(Sec 5) : 2022/ ISO 1524 : 2020	Determination of fineness of grind (*fifth revision*)
(Sec 6) : 2024/ ISO 19403-3 : 2017	Determination of the surface tension of liquid using the pendant drop method
(Part 4)	Optical test,

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IS No.	*Title*
(Sec 1) : 1988	Opacity (*third revision*)
(Sec 2) : 2021/ ISO 3668 : 2017	Colour-visual comparison of colour of paints (*fourth revision*)
(Sec 3) : 1988	Light fastness test (*third revision*)
(Sec 4) : 2020/ ISO 2813 : 2014	Gloss determination of gloss value at 20°, 60° and 85° (*fourth revision*)
(Part 5)	Mechanical test on paint films,
(Sec 1) : 1988	Hardness tests (*third revision*)
(Sec 2) :1988	Flexibility and adhesion (*third revision*)
(Sec 3) : 2019/ ISO 6272-1 : 2011	Impact resistance — Falling-weight test large-area indenter (*fifth revision*)
(Sec 4) : 1988	Print free test (*third revision*)
(Sec 5) : 2019/ ISO 6272-2 : 2011	Impact resistance — Falling-weight test small-area indenter
(Sec 6) : 2024/ ISO 7784-1 : 2023	Determination of resistance to abrasion by abrasive-paper covered wheels and rotating test specimen
(Sec 7) : 2024/ ISO 7784-2 : 2023	Determination of resistance to abrasion by abrasive rubber wheels and rotating test specimen
(Part 6)	Durability tests,
(Sec 1) : 1988	Resistance to humidity under conditions of condensation (*third revision*)
(Sec 2) : 1989	Keeping properties (*third revision*)
(Sec 3) : 1990	Moisture vapour permeability (*third revision*)
(Sec 4) : 1991	Degradation of coatings (pictorial aids for evaluation)
(Sec 5) : 1997	Accelerated weathering test (*third revision*)
(Part 7)	Environmental tests on paint films,
(Sec 1) : 1989	Resistance to water (*third revision*)
(Sec 2) : 1990	Resistance to liquids (*third revision*)
(Sec 3) : 2020/ ISO 3248 : 2016	Determination of the effect of heat (*fourth revision*)
(Sec 4) : 1990	Resistance to bleeding of pigments (*third revision*)
(Part 8)	Tests for pigments and other solids,
(Sec 1) : 1989	Residue on sieve (*third revision*)
(Sec 2) : 1990	Pigments and non –volatile matter (*third revision*)

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IS No.	*Title*
(Sec 3):1993	Ash content (*third revision*)
(Sec 4) : 2015	Phthalic anhydride (*fourth revision*)
(Sec 5) : 2022	Lead restriction test (*fourth revision*)
(Sec 6) : 1993	Volume solids
(Part 9)	Tests for lacquers and varnish,
(Sec 1) : 1993	Acid value (*third revision*)
(Sec 2) : 1993	Rosin test (*third revision*)
(Part 10)	Instrumental analysis,
(Sec 1) : 2022/ ISO 13320 : 2020	Particle size analysis — Laser diffraction methods (*first revision*)
(Sec 2) : 2018/ ISO 16773-1 : 2016	Electrochemical impedance spectroscopy EIS on coated and uncoated metallic specimens — Terms and definitions
(Sec 3) : 2018/ ISO 16773-2 : 2016	Electrochemical impedance spectroscopy EIS on coated and uncoated metallic specimens — Collection of data
(Sec 4) : 2018/ ISO 16773-3 : 2016	Electrochemical impedance spectroscopy EIS on coated and uncoated metallic specimens — Processing and analysis of data from dummy cells
(Sec 5) : 2018/ ISO 16773-4	Electrochemical impedance spectroscopy EIS on coated and uncoated metallic specimens — Examples of spectra of polymer — Coated and uncoated specimens
(Part 11)	Evaluation of degradation of coatings — Designation of quantity and size of defects and of intensity of uniform changes in appearance,
(Sec 2) : 2024/ ISO 4628-2 : 2016	Assessment of degree of blistering
(Sec 5) : 2024/ ISO 4628-5 : 2022	Assessment of degree of flaking
IS 104 : 2017	Specification for ready mixed paint brushing zinc chrome priming (*third revision*)
IS 109 : 2017	Specification for ready mixed paint finishing priming plaster to Indian Standard colour no 361 light stone and no 631 light grey white and off — white (*second revision*)
IS 137 : 2017	Ready mixed paint brushing matt or eggshell flat finishing interior to Indian standard colour as required — Specification (*second revision*)
IS 151 : 2017	Ready mixed paint, spraying, finishing, stoving, enamel for general purposes, colour as required — Specification (*second revision*)
IS 158 : 2015	Ready mixed paint brushing bituminous black acid alkali and heat resisting — Specification (*fourth revision*)
IS 159 : 2024	Ready mixed paint, brushing, acid resisting — Specification (*second revision*)
IS 164 : 2023	Ready mixed paint for road marking — Specification (*third revision*)

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IS No.	*Title*
IS 168 : 2024	Ready mixed paint air drying for general purpose — Specification (*fifth revision*)
IS 341 : 2016	Black japan, types A, B and C — Specification (*second revision*)
IS 486 : 2023	Brushes, sash tool for paints and varnishes — Specification (*fifth revision*)
IS 2074 : 2023	Ready mixed paint air drying red oxide zinc chrome priming — Specification (*fourth revision*)
IS 2075 : 2017	Ready mixed paint stoving red oxide zinc chrome priming — Specification (*third revision*)
IS 2339 : 2013	Specification for aluminium paint for general purposes (*first revision*)
IS 2932 : 2025	Air drying enamel paint for architectural and industrial applications — Specification (*fifth revision*)
IS 3536 : 2016	Ready mixed paint, brushing, wood primer — Specification (*second revision*)
IS 3537 : 2017	Ready mixed paint, finishing, interior, for general purposes, to Indian Standard colours — Specification (*first revision*)
IS 3539 : 1966	Specification for ready mixed paint, undercoating, for use under oil finishes, to Indian Standard colours, as required
IS 3585 : 2018	Ready mixed paint, aluminium, brushing, priming, water resistant for woodwork — Specification (*first revision*)
IS 3678 : 2016	Ready mixed paint, thick white, for lettering — Specification (*first revision*)
IS 8687	Vitreous enamels and frits — Method of test:
(Part 1) : 2023	Sieve analysis (*first revision*)
(Part 2) : 2020	Fusion flow test (*first revision*)
IS 8709 : 2023	Colour retention of vitreous enamel coatings — Method of test (*first revision*)
IS 8767 : 2020	Calcium carbonate, precipitated, and activated, for paints —Specification (*first revision*)
IS 9862 : 2017	Specification for ready mixed paint brushing bituminous black acid alkali water and chlorine resisting (*first revision*)
IS 11883 : 2017	Ready mixed paint, brushing, red oxide, priming for metals – Specification (*first revision*)
IS 12744 (Part 1) : 2013	Ready mixed paint, air drying, red oxide — Zinc phosphate, priming – Specification: Part 1 For domestic and decorative applications (*fourth revision*)
IS 13183 : 1991	Specification for aluminium paints, heat resistant
IS 13213 : 2018	Polyurethane full gloss enamel (two pack) — Specification (*first revision*)
IS 13607 : 1992	Specification for ready mixed paint, finishing, general purposes, synthetic
IS 13871 : 2021	Powder coating — Specification (*first revision*)
IS 14506 : 1998	Epoxy redoxide zinc phosphate weldable primer, two component — Specification

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IS No.	*Title*
IS 14589 : 1999	Zinc priming paint, epoxy based, two pack — Specification
IS 16719 : 2022	Solvent free polyurethane coatings for exterior of steel pipe and fittings — Specification (*first revision*)
IS 16942 : 2018	Silicone soya alkyd exterior paint — Specification
IS 17044 : 2018	Intumescent fire retardant paint (water based) — Specification
IS 17680 (Part 1) : 2021/ ISO 18314-1 : 2015	Analytical colorimetry: Part 1 Practical colour measurement
IS 18563 : 2024	Nanomodified paints and coatings — Code of practice
IS 19003 : 2022	Re-dispersible polymer powder — Specification

c) Thinners and Solvents

IS No.	*Title*
IS 82:1973	Methods of sampling and test for thinners and solvents for paints (*first revision*)
IS 384	Flat brushes for paints and varnishes — Specification:
(Part 1) : 2023	Heavy duty (*seventh revision*)
(Part 2) : 2023	Household purposes *(seventh revision)*
IS 430 : 1972	Specification for paint remover, solvent type, non-flammable (*second revision*)
IS 431 : 1972	Specification for Paint remover, solvent type, flammable (*second revision*)
IS 486 : 2023	Brushes, sash tool for paints and varnishes — Specification (*fifth revision*)
IS 487 : 2023	Brushes, paints and varnishes – Oval, ferrule bound; and round, ferrule bound (*sixth revision*)
IS 533:2007	Specification for gum spirit of turpentine (oil of turpentine) (*third revision*)
IS 644 : 2023	Dipentene for paints — Specification (*third revision*)
IS 5667 : 1970	Specification for thinner for cellulose nitrate based paints and lacquers
IS 14314 : 1995	Specification for thinner general purposes for synthetic paints and varnishes

d) Varnishes and Lacquers

IS 344 : 2023	Varnish stoving — Specification (*second revision*)
IS 347 : 2023	Varnish shellac for general purposes — Specification (*second revision*)
IS 348 : 1968	Specification for French polish (*first revision*)
IS 524 : 2024	Varnish for general purpose — Specification (*third revision*)
IS 642 : 2024	Varnish medium for aluminium paint — Specification *(second revision)*

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IS 5691 : 2024	Lacquer, cellulose nitrate, pigmented, finishing, glossy — Specification (*first revision*)
IS 6127 : 2023	Varnish, spar and fungicidal — Specification (*first revision*)
IS 10018:1981	Specification for lacquer, cellulose, nitrate clear, finishing, glossy for wood

19 POLYMERS, PLASTICS AND GEOSYNTHETICS/GEOTEXTILES

IS No.	*Title*
IS 1998 : 1962	Methods of test for thermosetting synthetic resin bonded laminated sheets
IS 2036 : 1995	Specification for phenolic laminated sheets (*second revision*)
IS 2046 : 2025/ ISO 4586 : 2018	Decorative thermosetting synthetic resin bonded laminated sheets — Specification:
(Part 1) : 2025	Introduction and general information (*third revision*)
(Part 2) : 2025	Determination of properties (*third revision*)
(Part 3) : 2025	Classification and specifications for laminates less than 2 mm thick and intended for bonding to supporting substrates (*third revision*)
(Part 4) : 2025	Classification and specifications for compact laminates of thickness 2 mm and greater (*third revision*)
(Part 5) : 2025	Classification and specifications for flooring grade laminates less than 2 mm thick intended for bonding to supporting substrates (*third revision*)
(Part 6) : 2025	Classification and specifications for exterior-grade compact laminates of thickness 2 mm and greater (*third revision*)
(Part 7) : 2025	Classification and specifications for design laminates (*third revision*)
(Part 8) : 2025	Classification and specifications for alternative core laminates (*third revision*)
IS 2076 : 1981	Specification for unsupported polyvinyl chloride sheeting (*first revision*)
IS 2508 : 2024	Polyethylene films and sheets — Specification (*fourth revision*)
IS 6307 : 2023	Rigid PVC sheets — Specification (*second revision*)
IS 10889 : 2004	Specification for high density polyethylene films (*first revision*)
IS 12830:1989	Specification for rubber based adhesives for fixing PVC tiles to cement
IS 13162	Geotextiles — Methods of test:
(Part 2) : 2024	Determination of resistance to the exposure of ultraviolet light, moisture and heat (xenon-arc type apparatus) (*first revision*)
(Part 3) : 2021	Single layers (*first revision*)
(Part 4) : 2025	Dynamic perforation test (cone drop test) (*first revision*)
IS 13262 : 1992	Specification for pressure sensitive adhesive tapes with plastic base

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IS No.	*Title*
IS 13325 : 1992	Method of test for the determination to tensile properties of extruded polymer geogrids using the wide strip
IS 13326 (Part 1) : 2025/ ISO 12957-1 : 2018	Geosynthetics — Determination of friction characteristics: Part 1 Direct shear test (*first revision*)
IS 14182 : 1994	Specification for solvent cement for use with unplasticized polyvinylchloride plastic pipe and fittings
IS 14293 : 2024	Geotextiles — Method for determination of trapezoid tearing strength (*first revision*)
IS 14294 : 2024	Geotextiles — Method for determination of apparent opening size by dry sieving technique (*first revision*)
IS 14324 : 2025/ ISO 11058 : 2019	Geotextiles and geotextile-related products — Determination of water permeability characteristics normal to the plane, without load (*first revision*)
IS 14443 : 1997	Specification for polycarbonate sheets
IS 14500 : 1998	Linear low-density polyethylene (LLDPE) films — Specification
IS 14643 : 1999	Specification for unsintered polytetrafluoroethylene (PTFE) Tape for thread sealing applications
IS 14706 : 2024/ ISO 9862 : 2023	Geosynthetics — Sampling and preparation of test specimens (*first revision*)
IS 14714 : 2024	Geotextiles — Determination of abrasion resistance (*first revision*)
IS 14715	Jute geotextiles:
(Part 1) : 2016	Strengthening of sub-grade in roads — Specification (*second revision*)
(Part 2) : 2016	Control of bank erosion in rivers and waterways — Specification (*second revision*)
IS 14716 : 2021	Geosynthetics — Test method for the determination of mass per unit area of geotextiles and geotextile-related products (*first revision*)
IS 14739 : 2026/ ISO 13431 : 1999	Geotextiles and geotextile-related products: Determination of tensile creep and creep rupture behaviour (*second revision*)
IS 14753 : 1999	Specification for poly(methyl) methacrylate (PMMA) (acrylic) sheets
IS 14986 : 2001	Guidelines for application of jute geotextile for rain water erosion control in road and railway embankments and hill slopes
IS 15060 : 2018/ ISO 10321 : 2008	Geosynthetics tensile test for joint seams by wide-width strip method (*first revision*)
IS 15351 : 2015	Agro textiles laminated high density polyethylene HDPE woven geomembrane for water proof lining — Specification (*second revision*)
IS 15869 : 2020	Textiles-Open weave coir Bhoovastra — Specification (*first revision*)
IS 15909 : 2020	PVC geomembranes for lining — Specification (*second revision*)

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IS No.	*Title*
IS 16352 : 2020	Geosynthetics — High density polyethylene (HDPE) geomembranes for lining-specification (*first revision*)
IS 16362 : 2024	Geosynthetics — Geotextiles for subsurface drainage, subgrade separation, subgrade stabilization, filtration and erosion control (in hard armor systems) applications — Specification (*second revision*)
IS 16380 : 2020	Geosynthetics — Method of test for measuring pullout resistance of geosynthetics in soil (*first revision*)
IS 16483 : 2017	Geosynthetics — Method for microscopic evaluation of the dispersion of carbon black in polyolefin geosynthetics
IS 16493 : 2017	Geosynthetics — Method of test for determination of pyramid puncture resistance of unprotected and protected geomembranes
IS 16627 : 2017	Agro textiles — High density polyethylene (HDPE) laminated woven lay flat tube for use in mains and submains of drip irrigation system — Specification
IS 16635 : 2026	Geosynthetics — Wide-width tensile test (*first revision*)
IS 16653 : 2017	Geosynthetics — Needle punched nonwoven geobags for coastal and waterways protection — Specification
IS 16654 : 2017	Geosynthetics — Polypropylene multifilament woven geobags for coastal and waterways protection — Specification
IS 17179 : 2025/ ISO 12958 : 2020	Geotextiles and geotextile-related products — Determination of water flow capacity in their plane:
(Part 1)	Index text
(Part 2)	Performance test (*first revision*)
IS 17360 : 2020	Geosynthetics — Screening test method for determining the resistance of geotextiles and geotextile-related products to oxidation
IS 17363 : 2025	Geotextiles and geotextile-related products — Screening test method for determining the resistance to acid and alkaline liquids (*first revision*)
IS 17365 : 2025/ ISO/TS 20432 : 2022	Determination of the long-term strength of geosynthetics for soil reinforcement — Guidelines (*first revision*)
IS 17368 : 2020	Geosynthetics — Determination of damage to geosynthetic caused during installation
IS 17369	Geotextiles and geotextile-related products — Strength of internal structural junctions:
(Part 1) : 2020/ ISO 13426-1 : 2019	Geocells
(Part 2) : 2025/ ISO 13426-2 : 2005	Geocomposites (*first revision*)
IS 17371 : 2020	Geosynthetics — Geogrids for flexible pavements — Specification
IS 17372 : 2020	Geosynthetics — Polymeric strip/geostrip used as soil reinforcement in retaining structures — Specification

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IS No.	*Title*
IS 17373 : 2020	Geosynthetics — Geogrids used in reinforced soil retaining structures — Specification
IS 17374 : 2020	Geosynthetics — Reinforced HDPE membrane for effluents and chemical resistance lining — Specification
IS 17420 : 2020	Geosynthetics — Index test procedure for the evaluation of mechanical damage under repeated loading-damage caused by granular material (laboratory test method)
IS 17483	Geosynthetics — Geocells — Specification:
(Part 1) : 2020	Load bearing application
(Part 2) : 2020	Slope erosion protection application
IS 17544 : 2022	Multilayer (PE-AL-PE) plastic piping system for indoor gas installations with a maximum operating pressure up to and including 500 KPA (5 Bar) — Specification
IS 17728 : 2021	Agro textiles — High density polyethylene (HDPE) laminated woven lay flat tube and fittings for use in rain irrigation system — Specification
IS 17880 : 2022	Geosynthetics — Synthetic polymer rope gabions for coastal and waterways protection — Specification
IS 18309 : 2023	Geosynthetics — Prefabricated vertical drains for quick consolidation of very soft plastic soil-Specification
IS 18998 : 2025/ ISO 24187 : 2023	Principles for the analysis of microplastics present in the environment
IS 19050 : 2025/ ISO 17541 : 2014	Plastics — Quantitative evaluation of scratch-induced damage and scratch visibility
IS 19169 : 2025/ ISO 21475 : 2019	Plastics — Methods of exposure to determine the wavelength dependent degradation using spectrally dispersed radiation
IS 19104 : 2025	Geosynthetics — Geotextile tubes for coastal and waterways protection — Specification
IS 19161 : 2025/ ISO 16869 : 2008	Plastics — Assessment of the effectiveness of fungistatic compounds in plastics formulations
IS 19162 : 2025/ ISO 15314 : 2018	Plastics — Methods for marine exposure
IS 19165 : 2025/ ISO 22196 : 2011	Measurement of antibacterial activity on plastics and other non-porous surfaces
IS 19166 : 2025/ ISO 10640 : 2011	Plastics — Methodology for assessing polymer photoageing by FTIR and UV/visible spectroscopy
IS 19167 : 2025/ ISO 21702 : 2019	Measurement of antiviral activity on plastics and other non-porous surfaces
IS 19168 : 2025/ ISO 29664 : 2010	Plastics — Artificial weathering including acidic deposition

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IS No.	*Title*
IS 19218 : 2025	Textiles — Coir non-woven stitched composite geotextiles for erosion control applications — Specification

20 SANITARY APPLIANCES AND WATER FITTINGS

a) General

IS No.	*Title*
IS 804 : 1967	Specification for rectangular pressed steel tanks (*first revision*)
IS 906 : 2024	Portable and fixed type revolving branch for firefighting — Specification (*fourth revision*)
IS 1726 : 1991	Specification for cast iron manhole covers and frames (*third revision*)
IS 2963 : 2013	Copper alloy waste fittings and waste plug for wash-basins and sinks — Specification (*second revision*)
IS 5219 : 2013	Cast copper alloy traps — Specification (*first revision*)
IS 5455 : 1969	Specification for cast-iron steps for manholes
IS 5961 : 1970	Specification for cast iron gratings for drainage purposes
IS 9140 : 1996	Method of sampling of vitreous and fire clay sanitary appliances (*second revision*)
IS 9872 : 1981	Specification for precast concrete septic tanks
IS 12592 : 2002	Specification for precast concrete manhole covers and frames (*first revision*)
IS 12701 : 1996	Specification for rotational moulded polyethylene water storage tanks (*first revision*)
IS 13356 : 1992	Specification for precast ferrocement water tanks (250 to 10 000 litres capacity)
IS 14399	Hot press moulded thermosetting glass fibre reinforced (GRP) sectional water storage tanks:
(Part 1) : 1996	Specification for panels
(Part 2) : 1996	Guidelines for assembly, installation and testing
IS 17650	Water efficient plumbing products — Requirements:
(Part 1) : 2021	Sanitaryware
(Part 2) : 2021	Sanitary fittings
IS 18666 : 2024	Rotationally moulded polyethylene septic tanks — Specification
IS 18797 : 2024	Packaged sewage treatment plant — Specification
IS 19315 : 2025	Ball valves for water works purposes — Specification

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b) Pipes and Fittings excluding valves

i) *Brass and Copper Pipes and Fittings*

IS No.	*Title*
IS 407 : 1981	Specification for brass tubes for general purposes (*third revision*)
IS 2501 : 1995	Specification for solid drawn copper tubes for general engineering purposes (*third revision*)

ii) *Cast Iron Pipes and Fittings*

IS No.	*Title*
IS 1536 : 2001	Specification for centrifugally cast (spun) iron pressure pipes for water, gas and sewage (*fourth revision*)
IS 1536 : 2023	Centrifugally cast spun iron pressure pipes for water gas and sewage specification (*fifth revision*)
IS 1537 : 1976	Specification for vertically cast iron pressure pipes for water, gas and sewage (*first revision*)
IS 1538 : 1993	Specification for cast iron fittings for pressure pipes for water, gas and sewage (*third revision*)
IS 1729 : 2023	Sand cast iron spigot and socket pipes, fittings and accessories — Specification (*third revision*)
IS 1729 : 2023	Sand cast iron spigot and socket pipes fittings and accessories — Specification (*third revision*)
IS 1879 : 2010	Malleable cast iron pipe fittings — Specification (*third revision*)
IS 3989 : 2024	Centrifugally cast (spun) iron spigot and socket pipes, fittings and accessories — Specification (*fourth revision*)
IS 5531 : 2014	Cast iron specials for asbestos cement pressure pipes for water and sewage — Specification (*third revision*)
IS 6418 : 1971	Specification for cast iron and malleable cast iron flanges for general engineering purposes
IS 7181 : 1986	Specification for horizontally cast iron double flanged pipes for water, gas and sewage (*first revision*)
IS 8329 : 2000	Specification for centrifugally cast (spun) ductile iron pressure pipes for water, gas and sewage (*third revision*)
IS 8794 : 1988	Specification for cast iron detachable joints for use with asbestos cement pressure pipes (*first revision*)
IS 9523 : 2000	Ductile iron fittings for pressure pipes for water gas and sewage — Specification (*first revision*)
IS 10292 : 1988	Dimensional requirements for rubber sealing rings for C.I.D. Joints in asbestos cement piping (*first revision*)

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IS No.	*Title*
IS 10299 : 1982	Cast iron saddle pieces for service connection from asbestos cement pressure pipes
IS 11606 : 1986	Methods of sampling cast iron pipes & fittings
IS 12820 : 2004	Dimensional requirements of rubber gaskets for mechanical joints and push-on joints for use with cast iron pipes and fittings for carrying water gas and sewage (*first revision*)
IS 12987 : 1991	Cast iron detachable joints for use with asbestos cement pressure pipes (light duty) — Specification
IS 12988 : 1991	Rubber sealing rings for CID joints for light duty AC pipes — Dimensional requirements
IS 13382 : 2018	Cast iron specials for mechanical and push-on flexible joints for pressure pipes for water gas and sewage — Specification (*second revision*)
IS 15905 : 2024	Hubless centrifugally cast (spun) iron pipes, fittings and accessories — Spigot series — Specification (*first revision*)

iii) *Lead Pipes and Fittings*

IS No.	*Title*
IS 404 (Part 1) : 1993	Specification for lead pipes: Part 1 For other than chemical purpose (*third revision*)

iv) *Fibre Pipes and Fittings*

IS No.	*Title*
IS 11925 : 1986	Specification for pitch-impregnated fibre pipes and fittings for drainage purposes

v) *Plastic Pipes and Fittings*

IS No.	*Title*
IS 4984 : 2016	Polyethylene pipes for water supply — Specification (*fifth revision*)
IS 4985 : 2021	Unplasticized PVC pipes for potable water supplies — Specification (*fourth revision*)
IS 7834	Specification for injection moulded PVC socket fittings with solvent cement joints for water supplies:
(Part 1) : 1987	General requirements (*first revision*)
(Part 2) : 1987	Specific requirements for 45° elbows (*first revision*)
(Part 3) : 1987	Specific requirements for 90° elbows (*first revision*)
(Part 4) : 1987	Specific requirements for 90° tees (*first revision*)
(Part 5) : 1987	Specific requirements for 45° tees (*first revision*)

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IS No.	*Title*
(Part 6) : 1987	Specific requirements for sockets (*first revision*)
(Part 7) : 1987	Specific requirements for unions (*first revision*)
(Part 8) : 1987	Specific requirements for caps (*first revision*)
IS 8008 : 2022	Injection moulded machined polyethylene fittings for water supply — Specification (*second revision*)
IS 8360 : 2022	Fabricated polyethylene fittings for water supply — Specification (*first revision*)
IS 10124	Specification for fabricated PVC fittings for potable water supplies:
(Part 1) : 2009	General requirements (*second revision*)
(Part 2) : 2009	Specific requirements for sockets (*second revision*)
(Part 3) : 2009	Specific requirements of straight reducers (*second revision*)
(Part 4) : 2009	Specific requirements for caps (*second revision*)
(Part 5) : 2009	Specific requirements for equal tees (*second revision*)
(Part 6) : 2009	Specific requirements for flanged tail piece with metallic flanges (*second revision*)
(Part 7) : 2009	Specific requirements for threaded adaptors (*second revision*)
(Part 8) : 2009	Specific requirements for 90° bends (*second revision*)
(Part 9) : 2009	Specific requirements for 60° bends (*second revision*)
(Part 10) : 2009	Specific requirements for 45° bends (*second revision*)
(Part 11) : 2009	Specific requirements for 30° bends (*second revision*)
(Part 12) : 2009	Specific requirements for 22½° bends (*second revision*)
(Part 13) : 2009	Specific requirements for 11¼° bends (*second revision*)
IS 12235	Methods of test for unplasticised PVC pipes for potable water supplies:
(Part 1) : 2004	Measurement of dimensions (*first revision*)
(Part 2) : 2004	Determination of Vicat softening temperature (*first revision*)
(Part 3) : 2004	Test for opacity (*first revision*)
(Part 4) : 2004	Determining the detrimental effect on the composition of water (*first revision*)
(Part 5)	Longitudinal reversion:
(Sec 1) : 2004	Determination methods (*first revision*)
(Sec 2) : 2004	Determination parameters (*first revision*)
(Part 6) : 2004	Stress relief test (*first revision*)
(Part 7) : 2004	Resistance to sulphuric acid (*first revision*)

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IS No.	*Title*
(Part 8)	Resistance to internal hydrostatic pressure,
(Sec 1) : 2004	Resistance to internal hydrostatic pressure at constant internal water pressure (*first revision*)
(Sec 2) : 2004	Leak-tightness of elastomeric sealing ring type socket joints under positive internal pressure and with angular deflection (*first revision*)
(Sec 3) : 2004	Leak-tightness of elastomeric sealing ring type socket joints under negative internal pressure and with angular deflection (*first revision*)
(Sec 4) : 2004	Leak-tightness of elastomeric sealing ring type socket joints under positive internal pressure without angular deflection (*first revision*)
(Part 9) : 2004	Resistance to external blows (impact resistance) at 0 °C (Round-the-clock method)
(Part 10) : 2004	Determination of organotin as tin aqueous solution (*first revision*)
(Part 11) : 2004	Resistance to dichloromethane at specified temperature (*first revision*)
(Part 12) : 2004	Determination of titanium dioxide content
(Part 13) : 2004	Determination of tensile strength and elongation
(Part 14) : 2004	Determination of density/relative density (specific gravity)
(Part 15) : 2004	Determination of vinyl chloride monomer content
(Part 16) : 2004	High temperature test
(Part 17) : 2004	Determination of ash content and sulphated ash content
(Part 18) : 2004	Determination of ring stiffness
(Part 19) : 2004	Flattening test
IS 12709 : 1994	Specification for glass-fibre reinforced plastic (GRP) pipes joints and fittings for use for potable water supply (*first revision*)
IS 12818 : 2010	Specification for unplasticized PVC screen and casing pipes for bore/tubewell (*second revision*)
IS 13592:2013	Unplasticized polyvinyl chloride (PVC-U) pipes for soil and waste discharge systemfor inside and outside buildings including ventilation and rainwater system — Specification (*first revision*)
IS 14333 : 2022	Polyethylene pipes for sewerage and industrial chemicals and effluent — Specification (*first revision*)
IS 14402 : 1996	Specification for GRP pipes, joints and fittings for use for sewerage, industrial waste and water (other than potable)
IS14735 : 1999	Specification for unplasticized polyvinyl chloride (UPVC) injection moulded fittings for soil and waste discharge system for inside buildings including ventilation and rain water system

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IS No.	*Title*
IS 14885 : 2022	Polyethylene pipes for the supply of gaseous fuels — Specification (*first revision*)
IS 15328:2003	Specification for unplasticized non-pressure polyvinyl chloride (PVC-U) pipes for use in underground drainage and sewerage system
IS 15450 : 2022	Polyethylene-aluminium-polyethylene composite pressure pipes for hot and cold water supplies — Specification (*first revision*)
IS 15778 : 2007	Chlorinated polyvinyl chloride (CPVC) pipes for potable hot and cold water distribution supplies — Specification
IS 15801 : 2008	Polypropylene-random copolymer pipes for hot and cold water supplies — Specification
IS 15927	Specification for polyethylene fittings for use with polyethylene pipes for the supply of gaseous fuels:
(Part 1) : 2012	Fittings for sockets using heated tools
(Part 2) : 2012	Spigot fittings for butt fusion jointing or socket fusion using heated tools, spigot fittings for use with electro-fusion fittings
(Part 3) : 2011	Electro-fusion fittings
IS 16098	Structured — wall plastics piping systems for non-pressure drainage and sewerage — Specification
(Part 1) : 2013	Pipes and fittings with smooth external surface, Type A
(Part 2) : 2013	Pipes and fittings with non-smooth external surface, Type B
IS 16534 : 2017	Chlorinated polyvinyl chloride (CPCV) Pipe fittings for automatic sprinkler fire extinguishing system — Specification
IS 16647 : 2017	Oriented unplasticized polyvinyl chloride (PVC-O) pipes for water supply — Specification
IS 17546 : 2021	Chlorinated polyvinyl chloride (CPVC) fittings for potable hot and cold water distribution supplies — Specification

vi) *Steel Tubes, Pipes and Fittings*

IS No.	*Title*
IS 1239	Steel tubes tubulars and other wrought steel fittings — Specification:
(Part 1) : 2004	Steel tubes (*sixth revision*)
(Part 2) : 2011	Steel pipe fittings (*fifth revision*)
IS 3589 : 2001	Steel pipes for water and sewage 1 683 mm to 2 540 mm outside diameter — Specification (*third revision*)
IS 4270 : 2001	Steel tubes used for water wells — Specification (*third revision*)
IS 5504 : 2025	Spiral welded pipes — Specification (*second revision*)

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IS 6286 : 1971	Specification for seamless and welded steel pipe for sub-zero temperature service
IS 6392 : 2020	Steel pipes flanges — Specification (*first revision*)

vii) *Stoneware Pipes and Fittings*

IS No.	*Title*
IS 651 : 2007	Glazed stoneware pipes and fittings — Specification (*sixth revision*)
IS 3006 : 1979	Specification for chemically resistant glazed stoneware pipes and fittings (*first revision*)

viii) *Asbestos Cement Pipes*

[*See* 9 (a) (ii) under the category 'Composite Matrix Products']

ix) *Concrete Pipes and Pipes lined/coated with Concrete or Mortar*

[*See* 9 (a) (iv) under the category 'Composite Matrix Products']

c) Kitchen and Sanitary Appliances

IS No.	*Title*
IS 774 : 2021	Ceramic vitreous china flushing cisterns for water closets and urinals — Specification (*sixth revision*)
IS 2548	Specification for plastic seats and covers for water-closets:
(Part 1) : 1996	Thermoset seats and covers (*fifth revision*)
(Part 2) : 1996	Thermoplastic seats and covers (*fifth revision*)
IS 2556	Vitreous china sanitary appliances — Specification:
(Part 1) : 2021	General requirements (*fourth revision*)
(Part 2) : 2024	Specific requirements of washdown water closets (*sixth revision*)
(Part 3) : 2024	Specific requirements of squatting pans (*sixth revision*)
(Part 4) : 2004	Specific requirements of wash basins (*fourth revision*)
(Part 5) : 1994	Specific requirements of laboratory sinks (*third revision*)
(Part 6) : 2021	Specific requirements of urinal and partition plates (*fifth revision*)
(Part 7) : 1995	Specific requirements of accessories for sanitary appliances (*third revision*)
(Part 8) : 2021	Specific requirements of close coupled and one-piece pedestal washdown and syphonic water closets (*sixth revision*)
(Part 9) : 2004	Specific requirements of pedestal type bidets (*fifth revision*)
(Part 14) : 2024	Specific requirements of integrated squatting pans (*second revision*)
(Part 15) : 2024	Specific requirements of universal water closets (*third revision*)

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IS No.	*Title*
(Part 16) : 2024	Specific requirements of washdown wall mounted water closets (*first revision*)
(Part 17) : 2001	Specific requirements of wall mounted bidets
IS 6411 : 1985	Specification for gel-coated glass fibre reinforced polyester resin bath tubs (*first revision*)
IS 7231 : 2021	Plastic flushing cisterns for water closets and urinals — Specification (*third revision*)
IS 11246 : 1992	Specification for glass fibre reinforced polyester resins (GRP) squatting pans (*first revision*)
IS 13983 : 1994	Specification for stainless steel sinks for domestic purposes

d) Valves and Water Fittings (Including Ferrules)

IS No.	*Title*
IS 778 : 1984	Specification for copper alloy gate, globe, and check valves for water works purposes (*fourth revision*)
IS 781 : 1984	Specification for cast copper alloy screw-down bib taps and stop valves for water services (*third revision*)
IS 1701 : 1960	Specification for mixing valves for ablutionary and domestic purposes
IS 1703 : 2000	Water fittings — Copper alloy float valves (horizontal plunger type) — Specification (*fourth revision*)
IS 1711 : 1984	Specification for self-closing taps for water supply purposes (*second revision*)
IS 1795 : 1982	Specification for pillar taps for water supply purposes (*second revision*)
IS 2692 : 1989	Specification for ferrules for water services (*second revision*)
IS 3004 : 1979	Specification for plug cocks for water supply purposes (*first revision*)
IS 3042 : 1965	Specification for single faced sluice gates (200 mm to1 200 mm size)
IS 3950 : 1979	Specification for surface boxes for sluice valves (*first revision*)
IS 4038 : 1986	Specification for foot valves for water works purposes (*second revision*)
IS 4346 : 1982	Specification for washers for use with fittings for water services (*first revision*)
IS 5312	Specification for swing check type reflux (non-return) valves:
(Part 1) : 2004	Single door pattern (*second revision*)
(Part 2) : 2013	Multi-door pattern (*first revision*)
IS 8931 : 2021	Copper alloy single taps combination tap assemblies stop valves and single lever mixers for water services — Specification (*second revision*)
IS 9338 : 2013	Cast iron/S.G. iron, cast steel screw-down stop valves for water works purposes — Specification (*second revision*)

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IS No.	*Title*
IS 9739 : 1981	Specification for pressure reducing valves for domestic water supply systems
IS 9758 : 1981	Specification for flush valves and fittings for water-closets and urinals
IS 9762 : 1994	Specification for polyethylene floats (spherical) for float valves (*first revision*)
IS 9763 : 2000	Plastic bib taps, pillar taps, angle valves, hot and cold water services — Specification (*second revision*)
IS 12234 : 2021	Plastic equilibrium float valves for cold water services — Specification *(first revision*)
IS 13049 : 1991	Specification for diaphragm type (plastic body) float operated valves for cold water services
IS 13114 : 1991	Specification for forged brass gate, globe and check valves for water works purposes
IS 13349 : 1992	Specification for single faced cast iron thimble mounted sluice gates
IS 14845 : 2000	Resilient seated cast iron air relief valves for water works purposes — Specification
IS 14846 : 2000	Sluice valves for water works purposes (50 mm to 1 200 mm) — Specification
IS 17585 : 2021	Beta (Outflow) valves for gravity type flushing cisterns — Specification

e) Water Meters

IS No.	*Title*
IS 779 : 1994	Specification for water meters (domestic type) (*sixth revision*)
IS 2373 : 1981	Specification for water meters (bulk type) (*third revision*)
IS 6784 : 1996	Method for performance testing of water meters (domestic type) (*second revision*)

21 STEEL AND ITS ALLOYS (*See* 23 FOR STRUCTURAL SECTIONS)

a) General

IS No.	*Title*
IS 1030 : 1998	Carbon steel castings for general engineering purposes (*fifth revision*)
IS 1136 : 2008	Preferred sizes for wrought metal products (*third revision*)
IS 1762 (Part 1) : 1974	Code for designation of steels: Part 1 Based on letter symbols (*first revision*)
IS 2049 : 1978	Colour code for the identification of wrought steel for general engineering purposes (*first revision*)
IS 2644 : 1994	High strength steel castings for general engineering and structural purposes — Specification (*fourth revision*)
IS 2708 : 1993	1.5 percent manganese steel castings for general engineering purposes — Specification (*third revision*)

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IS No.	*Title*
IS 7598 : 1990	Classification of steels (*first revision*)
IS 10461	Resistance to inter-granular corrosion of austenitic stainless steels — Method for determination:
(Part 1) : 2025	Corrosion test in nitric acid medium by measurement of loss in mass (Huey Test) (*second revision*)
(Part 2) : 1994	Corrosion test in a sulphuric acid/copper sulphate medium in the presence of copper turnings (Monypenny strauss test) (*first revision*)
IS/ISO 11951 : 2016	Cold-reduced tinmill products — Blackplate *(first revision)*
IS 12591 : 2018	Cold-reduced tinmill products — Electrolytic chromium/chromium oxide — Coated steel (*second revision*)
IS 14126 : 2019	Metallic coatings — Measurement of coating thickness — Coulometric method by anodic dissolution (*first revision*)
IS 16733 : 2018	Ferritic-pearlitic engineering steels for precipitation hardening from hot-working temperatures
IS 16854 : 2018	Single pack zinc rich coating for cold application — Specification
IS 17111 : 2025/ ISO 683-17 : 2023	Heat-Treatable steels, alloy steels and free-cutting steels — Ball and roller bearing steels (*first revision*)
IS 18435 (Part 11) : 2023	Corrosion of metals and alloys — Stress corrosion testing: Part 11 Testing the resistance of metals and alloys to hydrogen embrittlement and hydrogen-assisted cracking — Guidelines
IS 18436 : 2023	Metallic and other inorganic coatings — Post-coating treatments of iron or steel to reduce the risk of hydrogen embrittlement
IS 18437 : 2023/ ISO 15324 : 2000	Corrosion of metals and alloys — Evaluation of stress cracking by the drop evaporation test
IS 18438 : 2023/ ISO 18069 : 2015	Corrosion of metals and alloys — Method for determination of the uniform corrosion rate of stainless steels and nickel based alloys in liquids
IS 18463 : 2023/ ISO 9587 : 2007	Metallic and other inorganic coatings — Pretreatment of iron or steel to reduce the risk of hydrogen embrittlement

b) Structural Steel

IS No.	*Title*
IS 1161 : 2014	Steel tubes for structural purposes — Specification (*fifth revision*)
IS 2062	Structural steel
(Part 1) : 2025	Hot rolled medium and high tensile steel (*eight revision*)
(Part 2) : 2025	Hot rolled quenched and tempered steel plates, sheets, strips and wide flats
IS 2830 : 2012	Specification for carbon steel cast billet ingots, billets, blooms and slabs for re-rolling into steel for general structural purposes (*third revision*)

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IS No.	*Title*
IS 2831 : 2012	Specification for carbon steel billets ingots, blooms and slabs for re-rolling into structural steel (ordinary quality) (*fourth revision*)
IS 4923 : 2017	Hollow steel sections for structural use — Specification (*third revision*)
IS 8052 : 2006	Specification for steel ingots, billets and blooms for the production of springs, rivets and screws for general engineering applications (*second revision*)
IS 8952 : 1995	Steel ingots, blooms and billets for production of mild steel wire rods for general engineering purposes (*first revision*)
IS 11587 : 1986	Specification for structural weather resistant steels
IS 15103 : 2002	Fire resistant steel — Specification
IS 15911 : 2010	Structural steel (Ordinary quality) — Specification
IS 15962 : 2012	Structural steels for buildings and structures with improved seismic resistance
IS 16732 : 2019	Galvanized structural steel — Specification

c) Sheet and Strip

IS No.	*Title*
IS 277 : 2018	Galvanized steel strips and sheets plain and corrugated — Specification (*seventh revision*)
IS 412 : 1975	Specification for expanded metal steel sheets for general purposes (*second revision*)
IS 513	Cold reduced carbon steel sheet and strip:
(Part 1) : 2016	Cold forming and drawing purpose (*sixth revision*)
(Part 2) : 2016	High tensile and multi-phase steel (*sixth revision*)
IS 1079 : 2017	Hot rolled carbon steel sheet plate and strip — Specification (*seventh revision*)
IS 6911 : 2026	Stainless steel plate sheet and strip — Specification (*third revision*)
IS 7226 : 1974	Specification for cold rolled medium, high carbon and low alloy steel strip for general engineering purposes
IS 14246 : 2024	Continuously pre-painted galvanized steel sheets and strips — Specification (*second revision*)
IS 15961 : 2025	Hot-dip alumnium zinc alloy metallic coated steel strip and sheet — Specification (*first revision*)
IS 15965 : 2012	Pre-painted aluminium zinc alloy metallic coated steel strip & sheet (plain)
IS 18513 : 2023	Hot-dip zinc-aluminium-magnesium alloy coated steel sheets plates and strips — Specification
IS 18733 : 2024/ ISO 11531 : 2022	Metallic materials — Sheet and strip — Earing test

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IS No.	*Title*
IS 19210 : 2025	Pre-Painted Zinc-aluminium-magnesium alloy coated steel strip and sheet
IS 19474 : 2026	Hot-dip aluminium and aluminium-silicon coated steel sheet and strip — Specification
IS 19608 : 2026	Hot-rolled carbon and alloy steel plates, sheets and strips — Specification
IS 19626 : 2026	Electrolytic Zinc-nickel alloy coated steel sheet and strip — Specification

d) Bars, Rods, Wire and Wire Rods

IS No.	*Title*
IS 280 : 2006	Mild steel wire for general engineering purposes — Specification (*fourth revision*)
IS 1148 : 2009	Steel rivet bars medium and high tensile — For structural purposes — For structural purposes (*fourth revision*)
IS 1673 : 1984	Specification for mild steel wire cold heading quality (*second revision*)
IS 1812 : 1982	Specification for carbon steel wire for the manufacture of wood screw (*second revision*)
IS 1835 : 1976	Specification for round steel wire for ropes (*third revision*)
IS 1875 : 2025	Steel ingots, billets, blooms, slabs, and bars for forging — Specification (*sixth revision*)
IS 1921 : 2025	Solder wire — Solid and flux cored — Specification (*third revision*)
IS 2591 : 1982	Dimensions for hot rolled bars for threaded components (*second revision*)
IS 3150 : 1982	Specification for hexagonal wire netting for general purposes
IS 4826 : 2023	Hot-dip galvanized coatings on round steel wires requirements (*second revision*)
IS 6528 : 2024	Stainless steel wire — Specification (*second revision*)
IS 6603 : 2024	Stainless steel semi-finished products, bars, wire rods and bright bars — Specification (*second revision*)
IS 7887:1992	Specification for mild steel wire rods for general engineering purposes (*first revision*)
IS 7904 : 2018	High carbon steel wire rods — Specification (*second revision*)
IS 10631 : 1983	Stainless steel for welding electrode core wire
IS/ISO 16124 : 2015/ ISO 16124 : 2015	Steel wire rod — Dimensions and tolerances (*first revision*)

e) Plates and Studs

IS No.	*Title*
IS 1862 : 1975	Specification for studs (*second revision*)
IS 3502 : 2009	Specification for steel chequered plates (*third revision*)

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f) Tubes and Tubulars

IS No.	*Title*
IS 1161 : 2014	Specification for steel tubes for structural purposes (*fifth revision*)
IS 4923 : 2017	Hollow steel sections for structural use — Specification (*third revision*)
IS 17875 : 2022	Stainless steel seamless pipes and tubes for general services — Specification
IS 17876 : 2022	Stainless steel welded pipes and tubes for general services — Specification

g) Slotted Sections

IS No.	*Title*
IS 8081 : 1976	Specification for slotted sections

22 STONES

IS No.	*Title*
IS 1121	Methods of test for determination of strength properties of natural building stones:
(Part 1) : 2023	Compressive strength (*third revision*)
(Part 2) : 2023	Transverse strength (*third revision*)
(Part 3) : 2023	Indirect tensile strength (*third revision*)
(Part 4) : 2013	Shear strength (*second revision*)
(Part 5) : 2023	Flexural modulus of elasticity
IS 1122 : 2023	Determination of true specific gravity of natural building stones — Method of test (*second revision*)
IS 1123 : 1975	Method of identification of natural building stones (*first revision*)
IS 1124 : 1974	Method of test for determination of water absorption, apparent specific gravity and porosity of natural building stones (*first revision*)
IS 1125 : 2013	Method of test for determination of weathering of natural building stones (*second revision*)
IS 1126 : 2013	Method of test for determination of durability of natural building stones (*second revision*)
IS 1127 : 1970	Recommendations for dimensions and workmanship of natural building stones for masonry work (*first revision*)
IS 1128 : 1974	Specification for limestone (slab and tiles) (*first revision*)
IS 1129 : 1972	Recommendation for dressing of natural building stones (*first revision*)
IS 1130 : 1969	Specification for marble (blocks, slabs and tiles)

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IS No.	*Title*
IS 1706 : 2024	Determination of resistance to wear by abrasion of natural building stones — Method of test (*second revision*)
IS 3316 : 1974	Specification for structural granite (*first revision*)
IS 3620 : 1979	Specification for laterite stone block for masonry (*first revision*)
IS 3622 : 1977	Specification for sand stone (slabs and tiles) (*first revision*)
IS 4121 : 1967	Method of test for determination of water transmission rate by capillary action through natural building stones
IS 4122 : 1967	Method of test for surface softening of natural building stones by exposure to acidic atmospheres
IS 4348 : 2024	Determination of permeability of natural building stones — Methods of test (*second revision*)
IS 5218 : 1969	Method of test for toughness of natural building stones
IS 5640 : 1970	Method of test for determining the aggregate impact value of soft coarse aggregates
IS 6241 : 2024	Determination of stripping value of road aggregates — Method of test (*first revision*)
IS 6250 : 1981	Specification for roofing slate tiles (*first revision*)
IS 6579 : 1981	Specification for coarse aggregate for water bound macadam (*first revision*)
IS 7779	Schedule for properties and availability of stones for construction purposes:
(Part 1)	Gujarat State,
(Sec 1) : 1975	Availability of stones
(Sec 2) : 1975	Engineering properties of building stones
(Sec 3) : 1975	Engineering properties of stone aggregates
(Part 2)	Maharashtra state,
(Sec 1) : 1979	Availability of stones
(Sec 2) : 1979	Engineering properties of building stones
(Sec 3) : 1979	Engineering properties of stone aggregates
(Part 3)	Tamil Nadu state,
(Sec 2) : 1980	Engineering properties of building stones
(Sec 3) : 1980	Engineering properties of stone aggregates
(Part 4/Sec 1 to 3) : 1996	Karnataka state, Sections (1 to 3)
(Part 5)	Andhra Pradesh,
(Sec 1) : 1997	Availability of stones

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IS No.	*Title*
(Sec 2) : 1997	Engineering properties of building stones
(Sec 3):1997	Engineering properties of stone aggregates
IS 8759 : 2024	Maintenance and preservation of stone in buildings — Code of practice (*first revision*)
IS 9394 : 2024	Stone lintels — Specification (*first revision*)
IS 14223	Polished building stones specification:
(Part 1) : 2023	Granite and similar stones (*first revision*)
(Part 2) : 2025	Marble and similar stones

23 STRUCTURAL SECTIONS

a) Structural Shapes

IS No.	*Title*
IS 811 : 1987	Specification for cold formed light gauge structural steel sections (*second revision*)
IS 1173 : 2025	Hot rolled and slit steel tee bars — Dimensions and properties (*third revision*)
IS 1863 : 1979/ ISO 657-19	Rolled steel bulb flats (*first revision*)
IS 2314	Steel sheet piling section specification:
(Part 1) : 2023	Hot rolled sheet pile (*second revision*)
(Part 2) : 2023	Cold formed sheet pile (*second revision*)
IS 3443 : 1980	Specification for crane rail sections (*first revision*)
IS 3908 : 2025	Aluminium beam, channel and angle sections — Dimensions and properties (*second revision*)
IS 3964 : 1980	Specification for light rails (*first revision*)
IS 6445 : 1985	Specification for aluminium tee sections (*first revision*)

b) Dimensional Standards

IS No.	*Title*
IS 808 : 2021	Hot rolled steel beam column channel and angle sections — Dimensions and properties (*fourth revision*)
IS 1730 : 1989	Dimensions for steel plates, sheets strips and flats for general engineering purposes (*second revision*)
IS 1732 : 1989	Dimensions for round and square steel bars for structural and general engineering purposes (*second revision*)
IS 1852 : 1985	Rolling and cutting tolerances for hot rolled steel products (*fourth revision*)

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IS No.	*Title*
IS 2525 : 1982	Dimensions for wrought aluminium and aluminium alloy wire (*first revision*)
IS 2591 : 1982	Dimensions for hot rolled steel bars for threaded components (*second revision*)
IS 2673 : 2002	Dimensions for wrought aluminium and aluminium alloys, extruded round tube (*second revision*)
IS 2676 : 1981	Dimensions for wrought aluminium and aluminium alloys, sheet and strip (*first revision*)
IS 2677 : 1979	Dimensions for wrought aluminium and aluminium alloys, plates and hot rolled sheets (*first revision*)
IS 2678 : 1987	Dimensions and tolerances for wrought aluminium and aluminium alloys, drawn round tubes (*second revision*)
IS 3577 : 1992	Dimensions and tolerances for wrought aluminium and aluminium alloys rivet, bolt and screw stock (*first revision*)
IS 3954 : 1991	Hot rolled steel channel sections for general engineering purposes — Dimensions (*first revision*)
IS 3965 : 1981	Dimensions for wrought aluminium and aluminium alloys, bar, rod and section (*first revision*)
IS 6477 : 1983	Dimensions for wrought aluminium and aluminium alloys, extruded hollow sections (*first revision*)
IS 12779 : 1989	Rolling and cutting tolerances for hot rolled parallel flange beam and column sections
IS/ISO 16124 : 2015	Steel wire rod — Dimensions and tolerances (*first revision*)
IS/ISO 16160 : 2012	Hot-rolled steel sheet products dimensional and shape tolerances (*first revision*)
IS/ISO 16162 : 2012	Cold-rolled steel sheet products — Dimensional and shape tolerances (*first revision*)
IS/ISO 16163 : 2012	Continuously hot-dipped coated steel sheet products — Dimensional and shape tolerances (*first revision*)
IS 16998 : 2018	Hot-rolled steel plates — Tolerances on dimensions and shape

24 THERMAL INSULATION MATERIALS

IS No.	*Title*
IS 3144 : 2024	Mineral wool thermal insulation materials — Methods of test (*third revision*)
IS 3346 : 1980	Methods for the determination of thermal conductivity of thermal insulation materials (two slab, guarded hot-plate method (*first revision*)

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IS No.	*Title*
IS 3677 : 2026	Unbonded rock and slag wool for thermal insulation — Specification (*third revision*)
IS 4671 : 2018	Expanded polystyrene for thermal insulation purposes (*second revision*)
IS 5688 : 2018	Methods of test for preformed block-type and pipe-covering type thermal insulation (*second revision*)
IS 5724 : 2018	Thermal insulating cements — Methods of test (*first revision*)
IS 6598 : 2018	Cellular concrete for thermal insulation — Specification (*first revision*)
IS 7509 : 2024	Thermal insulating cements — Specification (*second revision*)
IS 8154 : 2024	Preformed calcium silicate insulation for temperatures up to 650 °C — Specification (*second revision*)
IS 8183 : 2024	Bonded mineral wool — Specification (*second revision*)
IS 9403 : 2018	Method of test for thermal conductance and transmittance of built-up Section by means of guarded hot box (*first revision*)
IS 9489 : 2018	Method of test for thermal conductivity of thermal insulation materials by means of heat flow meter (*first revision*)
IS 9490 : 2018	Method for determination of thermal conductivity of thermal insulation materials water calorimeter method (*first revision*)
IS 9742 : 2024	Sprayed mineral wool thermal insulation — Specification (*second revision*)
IS 9743 : 2020	Thermal insulation finishing cement — Specification (*second revision*)
IS 9842 : 2024	Preformed fibrous pipe insulation – Specification (*second revision*)
IS 11128 : 2018	Spray applied hydrated calcium silicate thermal insulation (*first revision*)
IS 11129 : 2018	Method of test for tumbling friability of preformed block-type thermal insulation (*first revision*)
IS 11239	Methods of test for rigid cellular thermal insulation materials:
(Part 1) : 2009/ ISO 1923 : 1981	Dimensions (*first revision*)
(Part 2) : 2019/ ISO 845 : 2006	Apparent density (*second revision*)
(Part 3) : 2009/ ISO 2796 : 1986	Dimensional stability (*first revision*)
(Part 4) : 2025/ ISO 1663 : 2023	Water vapour transmission rate (*second revision*)
(Part 5) : 2019/ ISO 4590 : 2016	Volume percent of open and closed cells (*second revision*)
(Part 6) : 1985	Heat distortion temperature
(Part 7) : 1985	Coefficient of linear thermal expansion at low temperatures

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IS No.	*Title*
(Part 8) : 2024	Flame height, time of burning and loss of mass (*first revision*)
(Part 9) : 1988	Water absorption
(Part 10) : 1985	Flexural strength
(Part 11) : 2024	Compressive strength (*first revision*)
(Part 12) : 1988	Horizontal burning characteristics
(Part 14) : 1992	Determination of flammability by oxygen index
IS 11307 : 2025	Specification for cellular glass block and pipe thermal insulating (*first revision*)
IS 11308 :2024	Hydraulic setting thermal insulating castables for temperatures up to 1 250 °C — Specification (*first revision*)
IS 12436 : 1988	Specification for preformed rigid polyurethane (PUR) and polyisocyanurate (PIR) foams for thermal insulation
IS 13204 : 2024	Specification for rigid phenolic foams for thermal insulation (*first revision*)
IS 13286 : 2025	Methods of test for surface spread of flame for thermal insulation materials (*first revision*)
IS 14656 : 2026	Ceramic fibre products — Methods of test (*first revision*)

25 THREADED FASTENERS, RIVETS AND NAILS

IS No.	*Title*
IS 207 : 1964	Specification for gate and shutter hooks and eyes (*revised*)
IS 451 : 1999	Specification for technical supply conditions for wood screws (*third revision*)
IS 554 : 1999	Specification for pipe threads where pressure-tide joints are made on the threads — Dimensions, tolerances and designation (*fourth revision*)
IS 723 : 2023	Steel countersunk head wire nails — Specification (*second revision*)
IS 724 : 2023	Mild steel and brass cup, ruler and square hooks and screw eyes — Specification (*second revision*)
IS 725 : 1961	Specification for copper wire nails (*revised*)
IS 730 : 2021	Specification for hook bolts for corrugated sheet roofing (*third revision*)
IS 1120 : 1975	Specification for coach screws (*first revision*)
IS 1363	Hexagon head bolts, screws and nuts of product grade C:
(Part 1) : 2023/ ISO 4016 : 2022	Hexagon head bolts (size range M5 to M64) (*sixth revision*)
(Part 2) : 2023/ ISO 4018 : 2022	Hexagon head screws (size range M5 to M64) (*sixth revision*)

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IS No.	*Title*
(Part 3) : 2018/ ISO 4034 : 2012	Style 1 Hexagon nuts (size range M5 to M64) (*fifth revision*)
IS 1364	Hexagon head bolts, screws and nuts of property grades A and B:
(Part 1) : 2023/ ISO 4014 : 2022	Hexagon head bolts (size range M1.6 to M64) (*sixth revision*)
(Part 2) : 2023/ ISO 4017 : 2022	Hexagon head screws (size range M1.6 to M64) (*sixth revision*)
(Part 3) : 2026/ ISO 4032 : 2023	Hexagon nuts, style 1 (size range M5 to M39) (*sixth revision*)
(Part 4) : 2025/ ISO 4035 : 2023	Hexagon nuts, style 0 (size range M1.6 to M64) (*fifth revision*)
(Part 5) : 2002/ ISO 4036 : 1999	Hexagon thin nuts — Product grade B (unchamfered) (size range M 1.6 to M10) (*fourth revision*)
(Part 6) : 2025/ ISO 4033 : 2023	Hexagon nuts, style 2 (Size Range M5 to M39) (*second revision*)
IS 1365 : 2022/ ISO 2009 : 2011	Slotted countersunk flat head screws — Product grade A (*fifth revision*)
IS 1366 : 2022/ ISO 1207 : 2011	Slotted cheese head screws — Product grade A (*fourth revision*)
IS 1367	Technical supply conditions for threaded steel fasteners:
(Part 1) : 2014/ ISO 8992 : 2005	General requirements for bolts, screws, studs and nuts (*fourth revision*)
(Part 2) : 2002/ ISO 4759-1 : 2000	Tolerances for fasteners — Bolts, screws, studs and nuts — Product grades a b and c (*third revision*)
(Part 3) : 2017/ ISO 898-1 : 2013	Mechanical properties of fasteners made of carbon steel and bolts, screws and studs (*fifth revision*)
(Part 5) : 2018 ISO 898-5 : 2012	Mechanical properties of fasteners made of carbon steel and alloy steel — Set screws and similar threaded fasteners with specified hardness classes-Coarse thread and fine pitch thread (*fourth revision*)
(Part 6) : 2025/ ISO 898-2:2022	Mechanical properties of fasteners made of carbon steel and alloy steel — Nuts with specified property classes (*fifth revision*)
(Part 7) : 1980	Mechanical properties and test methods for nuts without specified proof loads (*second revision*)
(Part 8) : 2020/ ISO 2320 : 2015	Prevailing torque type steel nuts functional properties (*fifth revision*)
(Part 9)	Surface discontinuities,
(Sec 1) : 1993/ ISO 6157-1 : 1988	Bolts screws and studs for general applications (*third revision*)

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IS No.	*Title*
(Sec 2) : 1993/ ISO 6167-3 : 1988	Bolts screws and studs for special applications (*third revision*)
(Part 10) : 2002/ ISO 6157-2 : 1995	Surface discontinuities — Nuts (*third revision*)
(Part 11) : 2024/ ISO 4042 : 2022	Electroplated coating systems (*fifth revision*)
(Part 12) : 1981	Phosphate coatings on threaded fasteners (*second Revision*)
(Part 13) : 2020/ ISO 10684 : 2004	Hot dip galvanized coatings on threaded fasteners (*third Revision*)
(Part 14)	Mechanical properties of corrosion-resistant stainless steel fasteners,
(Sec 1) : 2023/ ISO 3506-1 : 2020	Bolts screws and studs with specified grades and property classes (*fifth revision*)
(Sec 2) : 2023/ ISO 3506-2 : 2020	Nuts with specified grades and property classes (*fifth revision*)
(Sec 3) : 2018/ ISO 3506-3 : 2009	Set screws and similar fasteners not under tensile stress (*fourth revision*)
(Sec 4) : 2023/ ISO 3506-4 : 2009	Tapping screws
(Part 16) : 2002/ ISO 8991 : 1986	Designation system for fasteners (*third revision*)
(Part 17) : 2023/ ISO 3269 : 2019	Inspections, sampling and acceptance procedure (*fifth revision*)
IS 1367	Industrial fasteners — Threaded steel fasteners — Technical supply conditions:
(Part 18) : 1996	Packaging (*third revision*)
(Part 19) : 1997/ ISO 3800 : 1993	Axial load fatigue testing of bolts screws and studs
(Part 20) : 1996/ ISO 896-7 : 1992	Torsional test and minimum torques for bolts and screws with nominal diameters 1 mm to 10 mm
IS 1929 : 1982	Specification for hot forged steel rivets for hot closing (12 mm to 36 mm diameter) (*first revision*)
IS 2016 : 1967	Specification for plain washers (*first revision*)
IS 2155 : 1982	Specification for cold forged solid steel rivets for hot closing (6 mm to 16 mm diameter) (*first revision*)
IS 2269 : 2006	Hexagon socket head cap screws (*fifth revision*)
IS 2585 : 2006	Square head bolts, screws and square nuts of product Grade C — Specification (*second revision*)

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IS No.	*Title*
IS 2609 : 1972	Specification for coach bolts (*first revision*)
IS 2643 : 2005	Pipe threads where pressure-tight joints are not made on the threads — Dimensions, tolerances and designation (*third revision*)
IS 2687 : 1991	Specification for cap nuts (*second revision*)
IS 2769 : 1969	Sizes for squares and square holes for general engineering purposes (*first revision*)
IS 2907 : 1998	Specification for non-ferrous rivets (*first revision*)
IS 2998 : 1982	Specification for cold forged steel rivets for cold closing (1 mm to 16 mm diameter) (*first revision*)
IS 3063 : 1994	Specification for fasteners single coil rectangular section spring lock washers (*second revision*)
IS 3121 : 2023	Rigging screws and stretching screws — Specification (*second revision*)
IS 3468:1991	Specification for pipe nuts (*second revision*)
IS 3640 : 1982	Specification for hexagon fit bolts (*first revision*)
IS 3757 : 1985	Specification for high strength structural bolts (*second revision*)
IS 4206 : 2018/ ISO 888 : 2012	Dimensions for nominal lengths and thread lengths for bolts screws and studs *(second revision)*
IS 4762 : 2002	Fasteners — Worm drive hose clamps for general purposes — Specification (*second revision*)
IS 5370 : 1969	Specification for plain washers with outside diameter ≈ 3 × inside diameter
IS 5372 : 1975	Specification for taper washer for channels (ISMC) (*first revision*)
IS 5373 : 1969	Specification for square washers for wood fastenings
IS 5374 : 1975	Specification for taper washers for l-beam (ISMB) (*first revision*)
IS 5624 : 2021	Foundation bolts – Specification (*second revision*)
IS 6101 : 2018	Slotted pan head screws — Product Grade A (*third revision)*
IS 6113 : 1970	Specification for aluminium fasteners for building purposes
IS 6610 : 1972	Specification for heavy washers for steel structures
IS 6623 : 2004	High strength structural nuts — Specification (*second revision*)
IS 6639 : 1972	Specification for hexagon bolts for steel structures
IS 6649 : 1985	Specification for hardened and tempered washers for high strength structural bolts and nuts (*first revision*)
IS 6733 : 1972	Specification for wall and roofing nails
IS 6736 : 1972	Specification for slotted raised countersunk head wood screws

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IS No.	*Title*
IS 6739 : 1972	Specification for slotted round head wood screws
IS 6760 : 1972	Specification for slotted countersunk head wood screws
IS 6761 : 2023	Fasteners — Hexagon socket countersunk head screws with reduced loadability (*third revision*)
IS 7002 : 2018	Prevailing torque type hexagon regular nuts (with non-metallic insert) — Property classes 5,8 and 10 (*third revision*)
IS 7178 : 2018	Heat-Treated steel tapping screws — Mechanical properties (*fourth revision*)
IS 7483 : 2018/ ISO 7045 : 2011	Pan head screws with type H or type Z cross recess — Product grade A (*third revision*)
IS 7485	Countersunk flat head screws (common head style) with Type H or Type Z cross recess — Product grade A:
(Part 1) : 2018/ ISO 7046-1 : 2011	Steel screws of property class 4.8 (*third revision*)
(Part 2) : 2018/ ISO 7046-2 : 2011	Steel screws of property class 8.8, stainless steel screws and non- ferrous metal screws (*third revision*)
IS 7486 : 2018/ ISO 7047 : 2011	Raised countersunk head screws (common head style) with type H or type Z cross recess — Product grade A (*third revision*)
IS 8033 : 1976	Specification for washers with square hole for wood fastenings
IS 8412 : 1977	Specification for slotted countersunk head bolts for steel structures
IS 8536 : 2021/ ISO 225 : 2010	Fasteners — Bolts, screws, studs and nuts — Symbols and descriptions of dimensions (*second revision*)
IS 8822 : 1978	Specification for slotted mushroom head roofing bolts
IS 8869 : 1978	Specification for washers for corrugated sheet roofing
IS 8911 : 2018/ ISO 2010 : 2011	Slotted raised countersunk head screws — Product grade A (*second revision*)
IS 10102 : 1982	Specification for technical supply conditions for rivets
IS 10238 : 2001	Fasteners — Threaded steel fasteners — Step bolts for steel structures — Specification (*first revision*)
IS/ISO 10683 : 2018	Fasteners — Non electrolytically applied zinc flake coating systems
IS 12427 : 2001	Threaded steel fasteners — hexagon head transmission tower bolts — Specification (*first revision*)
IS 13722 : 2025/ ISO 8673 : 2023	Hexagon regular nuts (Style 1) with metric fine pitch thread — Product grades A and B — Specification (*third revision*)
IS 13723 : 2025/ ISO 8674 : 2023	Hexagon high nuts (Style 2) with metric fine pitch thread — Product grades A and B — Specification (*third revision*)
IS 13724 : 2025/ ISO 8675 : 2023	Hexagon thin nuts (Style 0) with metric fine pitch thread — Product grades A and B — Specification (*third revision*)

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IS No.	*Title*
IS 13725 : 2023/ ISO 8676 : 2022	Hexagon head screws with metric fine pitch thread –Product grades A and B (*third revision*)
IS 13726 : 2023/ ISO 8765 : 2022	Hexagon head bolts with metric fine pitch thread — Product grades A and B (*third revision*)
IS 14894 : 2023/ ISO 4015 : 2022	Fasteners-Hexagon head bolts of product grade B — Reduced shank (Shank diameter ≈ Pitch diameter) — Specification (*first revision*)
IS 15274 : 2021/ ISO 4759-3 : 2016	Tolerances for fasteners – Washers for bolts, screws and nuts — Product grades A, C and F (*first revision*)
IS 15581 : 2018	Hexagon nuts with flange, style 2 — Coarse thread (*first revision*)
IS 15582 : 2018	Hexagon bolts with flange – Small series — Product grade A with driving feature of product grade B (*first revision*)
IS 15628 : 2022/ ISO 4766 : 2011	Slotted set screws with flat point — Specification (*first revision*)
IS 17445 : 2020/ ISO 15330 : 1999	Fasteners — Preloading test for the detection of hydrogen embrittlement — Parallel bearing surface method
IS 17894 : 2022/ ISO 22081 : 2021	Geometrical product specifications (GPS) –Geometrical tolerancing — General geometrical specifications and general size specifications
IS 18471 : 2025	Fasteners — Drilling screws with tapping screw thread — Specification (*first revision*)
IS 18476 : 2023	Fasteners — Hexagon washer head drilling screws with tapping screw thread
IS 18480 : 2025	Self-tapping screws — Specification (*first revision*)
IS 18507 : 2026	Drywall screws — Specification (*first revision*)
IS 18508 : 2026	Chipboard screws — Specification (*first revision*)
IS 18509 : 2024	Cross recessed countersunk head wood screws — Specification
IS 18741 : 2024	Concrete nails — Specification
IS 19078 : 2025/ ISO 10666 : 1999	Drilling screws with tapping screw thread — Mechanical and functional properties
IS 19406 : 2026/ ISO 10587 : 2000	Metallic and other inorganic coatings — Test for residual embrittlement in both metallic coated and uncoated externally threaded articles and rods — Inclined wedge method

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26 TIMBER, BAMBOO AND OTHER LIGNOCELLULOSIC BUILDING MATERIALS

a) Timber and Bamboo

i) *Timber Classification*

IS No.	*Title*
IS 287 : 1993	Permissible moisture content for timber used for different purposes — Recommendations (*third revision*)
IS 399 : 1963	Classification of commercial timbers and their zonal distribution (*revised*)
IS 1150 : 2000	Trade names and abbreviated symbols *for timber species* (*third revision*)
IS 4970 : 1973	Key for identification of commercial timber (*first revision*)

ii) *Timber Conversion and Grading*

IS No.	*Title*
IS 190 : 1991	Specification for coniferous sawn timber (baulks and scantlings) (*fourth revision*)
IS 1326 : 2023	Non-Coniferous sawn timber baulks and scantling — Specification (*third revision*)
IS 1331 : 1971	Specification for cut sizes of timber (*second revision*)
IS 2377 : 1967	Tables for volumes of cut sizes of timber (*first revision*)
IS 3337 : 1978	Specification for ballies for general purposes (*first revision*)
IS 3386 : 1979	Specification for wooden fence posts (*first revision*)
IS 5966 : 2023	Non-coniferous timber in converted form for general purposes — Specification (*second revision*)
IS 14960 : 2001	Specification for preservative treated and seasoned sawn timber from rubberwood (*Hevea brasiliensis*)

iii) *Timber Testing*

IS No.	*Title*
IS 1708	Methods of testing small clear specimens of timber:
(Part 1) : 1986	Determination of moisture content (*second revision*)
(Part 2) : 1986	Determination of specific gravity (*second revision*)
(Part 3) : 1986	Determination of volumetric shrinkage (*second revision*)
(Part 4) : 1986	Determination of radial and tangential shrinkage and fibre saturation point (*second revision*)
(Part 5) : 1986	Determination of static bending strength (*second revision*)
(Part 6) : 1986	Determination of static bending strength under two point loading (*second revision*)

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IS No.	*Title*
(Part 7) : 1986	Determination of impact bending strength (*second revision*)
(Part 8) : 1986	Determination of compressive strength parallel to grain (*second revision*)
(Part 9) : 1986	Determination of compressive strength perpendicular to grain (*second revision*)
(Part 10) : 1986	Determination of hardness under static indentation (*second revision*)
(Part 11) : 1986	Determination of shear strength parallel to grain (*second revision*)
(Part 12) : 1986	Determination of tensile strength parallel to grain (*second revision*)
(Part 13) : 1986	Determination of tensile strength perpendicular to grain (*second revision*)
(Part 14) : 1986	Determination of cleavage strength parallel to grain (*second revision*)
(Part 15) : 1986	Determination of nail and screw holding power (*second revision*)
(Part 16) : 1986	Determination of brittleness by izod impact (*second revision*)
(Part 17) : 1986	Determination of brittleness by charpy impact (*second revision*)
(Part 18) : 1986	Determination of torsional strength (*second revision*)
IS 2408 : 1963	Methods of static tests of timbers in structural sizes
IS 2455 : 1990	Method of sampling of model trees and logs for timber testing and their conversion (*second revision*)
IS 2753	Methods for estimation of preservatives in treated timber and treating solutions:
(Part 1) : 1991	Determination of copper, arsenic, chromium, zinc, boron, creosote and fuel oil (*first revision*)
(Part 2) : 2014	Determination of copper in copper organic preservative salt (*second revision*)
IS 4907 : 2004	Method of testing timber connector joints (*first revision*)
IS 8292 : 1992	Methods for evaluation of working quality of timber under different wood operations — Method of test (*first revision*)
IS 8720 : 1978	Methods of sampling of timber scantlings from depots and their conversion for testing
IS 8745 : 1994	Methods of presentation of data of physical and mechanical properties of timber (*first revision*)
IS 10420 : 1982	Method of determination of sound absorption coefficient of timber by standing wave method
IS 10754 : 1983	Method of determination of thermal conductivity of timber
IS 11215 : 1991	Methods for determination of moisture content of timber and timber products (*first revision*)
IS 13621 : 1993	Method of test for determination of dielectric constant of wood under microwave frequencies

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iv) *Structural Timber and Test*

IS No.	*Title*
IS 2372 : 2021	Timber for cooling towers — Specification (*third revision*)
IS 3629 : 1986	Specification for structural timber in building (*first revision*)
IS 4891 : 1988	Specification for preferred cut sizes of structural timber (*first revision*)
IS 4924	Method of test for nail jointed timber trusses:
(Part 1) : 1968	Destructive test
(Part 2) : 1968	Proof test

v) *Logs*

IS No.	*Title*
IS 656 : 2023	Logs for plywood — Specification (*fourth revision*)
IS 3364	Method of measurement and evaluation of defects in timber:
(Part 1) : 1976	Logs (*first revision*)
(Part 2) : 1976	Converted timber (*first revision*)
IS 4895:1985	Specification for teak logs (*first revision*)
IS 5246:2000	Specification for coniferous logs (*first revision*)
IS 7308:1999	Specification for non-coniferous logs (*first revision*)

vi) *Bamboo*

IS No.	*Title*
IS 6874 : 2008	Method of tests for bamboos (*first revision*)
IS 8242 : 1976	Methods of tests for split bamboos
IS 17571 : 2021	Flattened bamboo board — Specification
IS 17572 : 2022	BWP grade bamboo mat-veneer composite board — Specification
IS 17573 : 2021	BWP grade bamboo mat board – Specification

(b) Reconstituted Products

i) *Plywood*

IS No.	*Title*
IS 303 : 2024	Plywood for general purposes — Specification (*fourth revision*)
IS 710 : 2024	Marine plywood — Specification (*third revision*)
IS 1328 : 2025	Veneered decorative plywood — Specification (*fourth revision*)

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IS No.	*Title*
IS 1734	Methods of test for plywood:
(Part 1) : 1983	Determination of density and moisture content (*second revision*)
(Part 2) : 1983	Determination of resistance of dry heat (*second revision*)
(Part 3) : 1983	Determination of fire resistance (*second revision*)
(Part 4) : 1983	Determination of glue shear strength (*second revision*)
(Part 5) : 1983	Test for adhesion of plies (*second revision*)
(Part 6) : 1983	Determination of water resistance (*second revision*)
(Part 7) : 1983	Mycological test (*second revision*)
(Part 8) : 1983	Determination of *p*H value (*second revision*)
(Part 9) : 1983	Determination of tensile strength (*second revision*)
(Part 10) : 1983	Determination of compressive strength (*second revision*)
(Part 11) : 1983	Determination of static bending strength (*second revision*)
(Part 12) : 1983	Determination of scarf joint strength (*second revision*)
(Part 13) : 1983	Determination of panel shear strength (*second revision*)
(Part 14) : 1983	Determination of plate shear strength (*second revision*)
(Part 15) : 1983	Central loading of plate test (*second revision*)
(Part 16) : 1983	Vibration of plywood plate test (*second revision*)
(Part 17) : 1983	Long time loading test of plywood strips (*second revision*)
(Part 18) : 1983	Impact resistance test on the surface of plywood (*second revision*)
(Part 19) : 1983	Part 19 Determination of nail and screw holding power (*second revision*)
(Part 20) : 1983	Part 20 Acidity and alkalinity resistance test (*second revision*)
IS 4990 : 2024	Plywood for concrete shuttering works — Specification *(fourth revision)*
IS 5509 : 2021	Fire retardant plywood – Specification *(third revision)*
IS 5539:1969	Specification for preservative treated plywood
IS 7316:1974	Specification for decorative plywood using plurality of veneers for decorative faces
IS 10701:2012	Structural plywood — Specification (*first revision*)
IS 13957:1994	Specification for metal faced plywood
IS 15791:2007	Museum plywood — Specification

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ii) *Blockboards, Particle Boards and Fibre Boards*

IS No.	*Title*
IS 1658 : 2006	Fibre hardboards — Specification (*third revision*)
IS 1659 : 2004	Block boards — Specification *(fourth revision)*
IS 2380	Methods of test for wood particle boards and boards from other lignocellulosic materials:
(Part 1) : 1977	Preparation and conditioning of test specimens (*first revision*)
(Part 2) : 1977	Accuracy of dimensions of boards (*first revision*)
(Part 3) : 1977	Determination of moisture content and density (*first revision*)
(Part 4) : 1977	Determination of static bending strength (*first revision*)
(Part 5) : 1977	Determination of tensile strength perpendicular to surface (*first revision*)
(Part 6) : 1977	Determination of tensile strength parallel to surface (*first revision*)
(Part 7) : 1977	Determination of compression-perpendicular to plane of the board (*first revision*)
(Part 8) : 1977	Compression parallel to surface test (*first revision*)
(Part 9) : 1977	Determination of resistance to shear in plane of the board (*first revision*)
(Part 10) : 1977	Falling hammer impact test (*first revision*)
(Part 11) : 1977	Surface hardness (*first revision*)
(Part 12) : 1977	Central loading of plate test (*first revision*)
(Part 13) : 1977	Long time loading bending test (*first revision*)
(Part 14) : 1977	Screw and nail withdrawal test (*first revision*)
(Part 15) : 1977	Lateral nail resistance (*first revision*)
(Part 16) : 1977	Determination of water absorption (*first revision*)
(Part 17) : 1977	Determination of swelling in water (*first revision*)
(Part 18) : 1977	Determination of mass and dimensional changes caused by moisture changes (*first revision*)
(Part 19) : 1977	Durability cyclic test for interior use (*first revision*)
(Part 20) : 1977	Accelerated weathering cyclic test for exterior use (*first revision*)
(Part 21) : 1977	Planeness test under uniform moisture content (*first revision*)
(Part 22) : 1981	Determination of surface glueability test
(Part 23) : 1981	Vibration test for particle boards
IS 3087 : 2025	Medium density particle boards of wood and other lignocellulosic materials for general purpose — Specification (*third revision*)

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IS No.	*Title*
IS 3097 : 2025	Veneered medium density particle boards from wood and other lignocellulosic material — Specification (*third revision*)
IS 3129 : 1985	Specification for low density particle boards (*first revision*)
IS 3308 : 1981	Specification for wood wool building slabs (*first revision*)
IS 3348 : 1965	Specification for fibre insulation boards
IS 3478 : 1966	Specification for high density wood particle boards
IS 12406 : 2025	Medium density fibre boards of wood and other lignocellulosic materials for general purpose — Specification (*third revision*)
IS 12823 : 2025	Prelaminated medium density particle boards from wood and other lignocellulosic material — Specification (*second revision*)
IS 13745 : 2020	Method for determination of formaldehyde content in wood-based panels by extraction method called perforator method (*first revision*)
IS 14276 : 2016	Cement bonded particle boards — Specification (*first revision*)
IS 14587 : 2025	Prelaminated medium density fibre boards of wood and other lignocellulosic materials — Specification (*second revision*)
IS 15786 : 2008	Prelaminated cement bonded particle boards — Specification
IS 19267 : 2025	Oriented strand boards — Specification

iii) *Wood-Based Laminates and Lumber*

IS No.	*Title*
IS 3513	Specification for resin treated compressed wood laminates (compregs):
(Part 3) : 1989	For General Purposes (*first revision*)
(Part 4) : 1966	Sampling and tests
IS 7638 : 1999	Methods of sampling for wood/lignocellulosic based panel products
IS 9307	Methods of tests for wood-based structural sandwich construction:
(Part 1) : 1979	Flexure test
(Part 2) : 1979	Edgewise compression test
(Part 3) : 1979	Flatwise compression test
(Part 4) : 1979	Shear test
(Part 5) : 1979	Flatwise tension test
(Part 6) : 1979	Flexure creep test
(Part 7) : 1979	Cantilever vibration test
(Part 8) : 1979	Weathering test

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IS No.	*Title*
IS 14315 : 1995	Specification for commercial veneers
IS 14616 : 1999	Specification for laminated veneer lumber
IS 16171 : 2014	Veneer laminated lumber – Specification

iv) *Bamboo and Coir Board Products*

IS No.	*Title*
IS 13958 : 1994	Specification for bamboo mat board for general purposes
IS 14588 : 1999	Specification for bamboo mat veneer composite for general purposes
IS 14842 : 2000	Specification for coir veneer board for general purposes
IS 15476 : 2004	Specification for bamboo mat corrugated sheets
IS 15491 : 2004	Specification for medium density coir boards for general purposes
IS 15877 : 2010	Coir faced block boards — Specification
IS 15878 : 2010	Coir hardboard for general purposes — Specification
IS 15972 : 2012	Bamboo-jute corrugated and semi-corrugated sheets — Specification

v) *Adhesives*

IS No.	*Title*
IS 848 : 2025	Synthetic resin adhesives for plywood (phenolic, aminoplastic and bio-materials based) — Specification (*third revision*)
IS 851 : 1978	Specification for synthetic resin adhesives for construction work (non-structural) in wood (*first revision*)
IS 852 : 1994	Animal glue for general wood-working purposes Specification (*second revision*)
IS 1508 : 1972	Specification for extenders for use in synthetic resin adhesives (urea-formaldehyde) for plywood (*first revision*)
IS 4835 : 1979	Specification for polyvinyl acetate dispersion-based adhesives for wood (*first revision*)
IS 9188 : 1979	Performance requirements for adhesive for structural laminated wood products for use under exterior exposure condition

27 UNIT WEIGHTS OF BUILDING MATERIALS

IS No.	*Title*
IS 875 (Part 1) : 1987	Code of practice for design loads (other than earthquake) for buildings and structures: Part 1 Dead loads — Unit weights of building material and stored materials (*second revision*)

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28 WATERPROOFING AND DAMP-PROOFING MATERIALS

IS No.	*Title*
IS 1322 : 1993	Specification for bitumen felts for waterproofing and damp-proofing (*fourth revision*)
IS 1580 : 1991	Specification for bituminous compound for waterproofing and caulking purposes (*second revision*)
IS 3037 : 1986	Specification for bitumen mastic for use in waterproofing of roofs (*first revision*)
IS 3384 : 1986	Specification for bitumen primer for use in waterproofing and damp-proofing (*first revision*)
IS 5871 : 1987	Specification for bitumen mastic for tanking and damp-proofing (*first revision*)
IS 7193 : 2013	Glass fibre base bitumen felts — Specification (*second revision*)
IS 12027 : 1987	Silicone-based water repellents — Specification
IS 12027 (Part 2) : 2025	Water repellents — Specification: Part 2 Silane based (*first revision*)
IS 13435	Acrylic based polymer waterproofing material — Method of tests:
(Part 1) : 2021	Determination of solid content (*first revision*)
(Part 2) : 1992	Determination of coarse particles
(Part 3) : 1992	Determination of capillary water take-up
(Part 4) : 1992	Determination of *p*H value
IS 13826	Bitumen based felts — Method of test:
(Part 1) : 1993	Breaking strength test
(Part 2) : 2022	Pliability test (*first revision*)
(Part 3) : 1993	Storage sticking test
(Part 4) : 1993	Pressure head test
(Part 5) : 1994	Heat resistance test
(Part 6) : 1993	Water absorption test
(Part 7) : 1993	Determination of binder content
IS 14695 : 1999	Glass fibre base coal tar pitch outer wrap — Specification
IS 16525 : 2017	Styrene butadiene styrene (SBS) modified bituminous waterproofing and damp-proofing membrane with polyester reinforcement — Specification
IS 16526 : 2017	Atactic polypropylene (APP) modified bituminous waterproofing and damp-proofing membrane with glass fibre reinforcement — Specification

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IS No.	*Title*
IS 16532 : 2017	APP modified bituminous waterproofing and damp-proofing membrane with polyester reinforcement — Specification
IS 16540 : 2017	Atactic polypropylene (APP) modified high molecular high density polyethylene (HMHDPE) bituminous waterproofing and damp-proofing membrane — Specification
IS 17856 : 2022	Vulcanized single layer rubber sheets for use as membrane — Specification

29 WELDING ELECTRODES AND WIRES

IS No.	*Title*
IS 814 : 2004	Covered electrodes for manual metal arc welding of carbon and carbon manganese steel — Specification (*sixth revision*)
IS 1278 : 1972	Specification for filler rods and wires for gas welding (*second revision*)
IS 1395 : 1982	Specification for low and medium alloy steel covered electrodes for manual metal arc welding (*third revision*)
IS 2879 : 1998	Mild steel for metal arc welding electrodes (*third revision*)
IS 4972	Resistance spot welding:
(Part 1) : 2022/ ISO 8430-1 : 2016	Electrode holders — Taper fixing 1 : 10 (*first revision*)
(Part 2) : 2022/ ISO 8430-2 : 2016	Electrode holders — Morse taper fixing (*first revision*)
(Part 3) : 2022/ ISO 8430-3 : 2016	Electrode holders parallel shank fixing for end thrust *(first revision)*
(Part 4) : 2022/ ISO 5183 – 1 : 1998	Electrode adaptors, male taper 1 : 10 — Conical fixing, taper 1 : 10 (*first revision*)
(Part 5) : 2022/ ISO 5183-2 : 2000	Electrode adaptors, male taper 1 : 10 — Parallel shank fixing for end-thrust electrodes (*first revision*)
(Part 6) : 2022/ ISO 5184 : 1979	Straight resistance spot welding electrodes (*first revision*)
(Part 7) : 2022/ ISO 1089 : 2023	Electrode taper fits for spot welding equipment — Dimensions (*second revision*)
(Part 8) : 2022/ ISO 5182 : 2016	Materials for electrodes and ancillary equipment (*first revision*)
IS 5206 : 1983	Covered electrodes for manual arc welding of stainless steel and other similar high alloy steels (*first revision*)
IS 5511 : 1991	Specification for covered electrodes for manual metal arc welding of cast iron (*first revision*)
IS 5856 : 2022/ ISO 14343 : 2017	Welding consumables — Wire electrodes, strip electrodes, wires and rods for arc welding of stainless and heat resisting steels — Classification (*third revision*)

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IS No.	*Title*
IS 5897 : 2026	Welding consumables — Wire electrodes, wires and rods for welding of aluminium and aluminium alloys — Specification (*second revision*)
IS 5898 : 1970	Specification for copper and copper alloy bare solid welding rods and electrodes
IS 6419 : 1996	Specification for welding rods and bare electrodes for gas shielded arc welding of structural steel (*first revision*)
IS 6560 : 2017/ ISO 21952 : 2012	Welding consumables — Wire electrodes, wires, rods and deposits for gas shielded arc welding of creep-resisting steels — Classification (*second revision*)
IS 8363 : 1976	Specification for bare wire electrodes for electroslag welding of steels
IS 10631 : 1983	Stainless steel for welding electrode core wire
IS 13007 : 2018	Arc welding and cutting — Non consumable tungsten electrodes — Classification (*first revision*)
IS 15977 : 2013	Classification and acceptance tests for bare solid wire electrodes and wire flux combination for submerged arc welding of structural steel — Specification
IS 18641 : 2024/ ISO 544 : 2017	Welding consumables — Technical delivery conditions for filler materials and fluxes — Type of product, dimensions, tolerances and markings
IS 18734 : 2024/ ISO 6847 : 2020	Welding consumables — Deposition of a weld metal pad for chemical analysis
IS 18738	Welding consumables — Methods of test:
(Part 1) : 2024/ ISO 15792-1 : 2020	Preparation of all-weld metal test pieces and specimens in steel, nickel and nickel alloys
(Part 2) : 2024/ ISO 15792-2 : 2020	Preparation of single-run and two-run technique test pieces and specimens in steel
(Part 3) : 2024/ ISO 15792-3 : 2011	Classification testing of positional capacity and root penetration of welding consumables in a fillet weld
IS 18783 : 2024/ ISO 14175 : 2008	Welding consumables — Gases and gas mixtures for fusion welding and allied processes
IS 8666 : 1977	Specification for copper and copper alloy covered electrodes for manual metal arc welding

30 WIRE ROPES AND WIRE PRODUCTS (Including wire for fencing)

IS No.	*Title*
IS 278 : 2009	Galvanized steel barbed wire for fencing — Specification (*fourth revision*)
IS 1755 : 2018	Metallic materials — Wire-wrapping test (*second revision*)
IS 2140 : 2022	Specification for stranded galvanized steel wire for fencing (*second revision*)
IS 2266 : 2024	Steel wire ropes for general engineering purpose — Specification (*sixth revision*)

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IS No.	*Title*
IS 2315 : 2023	Thimbles for wire ropes — Specification (*second revision*)
IS 2365 : 2024/ ISO 4344 : 2022	Steel wire ropes for lifts — Minimum requirements (*third revision*)
IS/ISO 2408 : 2017	Steel wire ropes — Requirements
IS 2485 : 2022	Drop forged sockets for wire ropes for general engineering purposes — Specification (*second revision*)
IS 2721 : 2025	Galvanized polyvinyl chloride (PVC), polyolefin and other polymer-coated steel chain link fence fabric — Specification (*third revision*)
IS 6594 : 2024	Technical supply conditions for steel wire ropes and strands (*fourth revision*)
IS 13902	Sockets for wire ropes for general purposes:
(Part 1) : 2022	General characteristics and conditions of acceptance (*first revision*)
(Part 2) : 2022	Special requirements for sockets produced by forging or machined from the solid (*first revision*)
IS 16013 : 2018	Welded wire gabions (metallic-coated or metallic-coated with PVC coating) — Specification (*first revision*)
IS 16014 : 2024	Mechanically woven, Double-Twisted, hexagonal wire mesh gabions, revet mattresses, rock fall netting and other products for civil engineering purposes (galvanized steel wire or galvanized steel wire with polymer coating) — Specification (*second revision*)
IS/ISO 17745 : 2016	Steel wire ring net panels — Definitions and specifications
IS/ISO 17746 : 2016	Steel wire rope net panels and rolls — Definitions and specifications
IS 17929	Fibre ropes for offshore station keeping:
(Part 1) : 2022/ ISO 18692-1 : 2018	General specification
(Part 2) : 2022/ ISO 18692-2 : 2019	Polyester
(Part 3) : 2022/ ISO 18692-3 : 2020	High modulus polyethylene (HMPE)
(Part 4) : 2024/ ISO 18692-4 : 2023	Polyarylate
(Part 5) : 2024/ ISO 18692-5 : 2024	Aramid
IS 18759 : 2024/ ISO 10325 : 2018	Fibre ropes — High modulus polyethylene — 8-Strand braided ropes, 12-Strand braided ropes and covered ropes
IS 19363 : 2025	Metallic or polymer coated — Welded wire mesh fencing — Specification

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 1 LOADS AND LOAD EFFECTS

BUREAU OF INDIAN STANDARDS

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National Building Code Sectional Committee, CED 46

FOREWORD

This NBCS (Part C/Section 1) covers the various loads, forces and effects which are to be taken into account for structural design of buildings. The various loads that are covered under this Section are dead load, imposed load, wind load, seismic force, snow load, special loads and load combinations.

This Section was first published in 1970 and subsequently revised in 1983, 2005 and 2016 as a chapter of the National Building Code of India. The first revision of this Section was modified in 1987 through Amendment No. 2 to the 1983 version to bring this Section in line with the latest revised loading code. Thereafter, in view of the revision of an important Indian Standard related to earthquake resistant design of structures, that is IS 1893, a need arose to revise this part of NBC. Therefore, the second revision of 2005 was formulated to consider the revised standard, IS 1893 (Part 1) : 2002 'Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings (*fifth revision*)' and to incorporate latest information on additional loads, forces and effects and the details regarding multi-hazard risk in various districts of India.

The significant changes incorporated in the second revision of 2005 included: the revision of the seismic zone map providing only four zones, instead of five, with erstwhile Zone I being merged into Zone II; changing of the values of seismic zone factors to reflect more realistic values of effective peak ground acceleration considering maximum considered earthquake (MCE) and service life of structure in each seismic zone; specifying of response spectra for three types of founding strata, namely rock and hard soil, medium soil and soft soil; the revision of empirical expression for estimating the fundamental natural period T_a of multi-storeyed buildings with regular moment resisting frames; adoption of the procedure of first calculating the actual force that may be experienced by the structure during the probable maximum earthquake, if it were to remain elastic, also bringing in the concept of response reduction due to ductile deformation and associated energy dissipation by introducing the 'response reduction factor' in place of the earlier performance factor; specifying a lower bound for the design base shear of buildings, based on empirical estimate of the fundamental natural period T_a; deletion of the soil-foundation system factor and instead, introduction of a clause to restrict the use of foundations vulnerable to differential settlements in severe seismic zones; revision of torsional eccentricity values upwards in view of serious damages observed in buildings with irregular plans; revision of modal combination rule in dynamic analysis of buildings; inclusion of a new clause on multi-hazard risk in various districts of India and a list of districts identified as multi-hazard prone districts; incorporation of latest amendments issued to IS 875 introduction of a clause on vibration in buildings for general guidance; and inclusion of reference to the Indian Standards on landslide control and design of retaining walls which were formulated after the first revision of the Section.

The significant changes incorporated in the third revision of 2016 included: enhanced provisions for parapets and balustrades, incorporating concentrated horizontal loads and adjustments in horizontal load per unit length; new areas of application, such as appurtenances fixed to structures; aerodynamic roughness heights for different terrain categories were explicitly addressed, aiding in the determination of turbulence intensity and wind speed profiles; a new modification factor k_4 was introduced for cyclonic regions; simplified empirical expressions for variations in hourly mean wind speed and turbulence intensity across different terrains were suggested, along with provisions for directionality, area averaging and correlation of pressures for design wind pressure; wind-induced interference for both tall and low-rise buildings was accounted for, with recommendations for wind tunnel or CFD studies for final designs of critical structures; updates to the wind speed map of India were included; and new provisions for design loads for helipads, emergency vehicles and collisions with structural elements in parking areas. For earthquake effects, the design spectra was defined for natural periods up to 6 s, applicable to all buildings regardless of construction material; temporary structures and buildings with flat slabs were brought under the scope of the standard (though with some conditions), with guidelines for addressing structural irregularities and estimating the in-plane stiffness of masonry infill walls; additional revisions including simplified torsional provisions, methods for computing natural periods in various structures, new provisions for evaluating liquefaction potential and blast loads; and inputs were also derived from the Heliport Manual, 1995 of the International Civil Aviation Organization for the design of roof top helipad, etc.

In the current revision, this publication has been renamed as National Building Construction Standards (NBCS).

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In this revision of this Part C/Section 1, load provisions have been updated in alignment with the existing IS 875 series and IS 4991. The IS 875 framework continues and the changes may be seen as its evolution. The present deterministic approach with safety margins will gradually be replaced by a unified approach of load classification and quantification. Loads will be grouped as natural or man-made and further as force-based or displacement-based. For natural hazards, return period–based probabilistic treatment will apply, while man-made loads will be based on statistical measures such as mean and standard deviation. This philosophy may, in due course, be reflected in future revisions of various Indian Standards on loading, following which their safety factors may also be recalibrated.

Further, considering the recent developments and R&D efforts nationally and internationally, the following significant modifications and inclusions have been made:

1) The provision relating to structural design basis report (SDBR) is introduced.

Dead Loads:

2) Unit weights of construction materials and stored materials have been included in Annex A.
3) Provisions relating to dead load for gardening and landscaping have been introduced.

Imposed Loads:

4) Imposed loads for dwelling units planned and executed in accordance with IS 8888 : 2020 'Requirements of low income housing for Urban areas guide (*second revision*)' have been brought at par with other residential buildings.
5) Imposed loads for datacenters have been included.
6) Imposed loads for roof top solar panels (with or without frames and with anchor blocks) and roof top HVAC equipment, have been included.
7) Imposed load for garage floors with and without stack parking have been updated.
8) Imposed load for refuge area and disaster shelter have been provided.

Wind Loads:

9) Provision relating to testing of building fabrics against wind borne debris has been included.
10) Table of basic wind speed for various cities of India has been updated.

Snow Loads:

11) A new term 'characteristic ground snow load' has been introduced and defined.
12) Characteristic ground snow load maps for union territory of Jammu and Kashmir, union territory of Ladakh and states of Himachal Pradesh, Uttarakhand and Sikkim have been introduced, resulting from studies conducted by Defence Geoinformatics Research Establishment (DRDO), Chandigarh.
13) Equation for calculation of characteristic ground snow load has been suggested in Annex H.
14) Clause relating to partial loading due to melting, sliding, snow redistribution and snow removal has been included.
15) Clause relating to ponding instability has been included.

Earthquake loads:

16) The Indian Standard IS 1893, on which the clause of earthquake load is largely based, is under revision at the time of publication of this NBCS. Major changes have been envisaged in the revision of IS 1893 which include apart from others, the introduction of probabilistic seismic hazard map of India having updated seismic zones. In the absence of availability of finalized version of revised IS 1893 at the time of revision of this Section, in this revision of the NBCS, the provisions of design as per existing IS 1893 (Part 1) : 2016 'Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings' have been continued through app ropriate reference to the same.

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Blast Loads:

17) Detailed provisions relating to design for blast loads have been included in Annex L. This annex provides prescriptive requirements for estimation of blast loads and simplified methods for structural analysis.

18) General requirements for blast-protection and categorization of structures have been provided.

19) Threat assessment requirements for determination of design explosive charge mass and scaled distance have been given.

20) Blast load estimation methods due to above-ground explosion of high-explosive under three different detonation scenarios: surface burst, free-air burst and air burst have been specified.

21) Evaluation of blast loads effects on structures of regular geometry and shape has been specified.

22) Prescriptive simplified single degree of freedom (SDOF) analysis procedure for peak response evaluation of structural elements based on flexural mode has been provided.

23) Guidelines for the application of multi-degree of freedom (MDOF) and detailed finite element analysis have been provided.

The provisions outlined in this Chapter are intended to assist structural designers in making informed engineering decisions when dealing with diverse loading scenarios, including special loads and unique load combinations that may arise in practical applications. It is expected that this Chapter will enable designers to adopt a rational and scientific approach in assessing structural loads and their combinations, thereby enhancing the resilience of structures against various external forces and ensuring their long-term performance. The Chapter serves as a fundamental reference for structural engineers, ensuring that the design approach aligns with safety requirements, reliability principles and advancements in engineering practices. It integrates knowledge from past experiences, research developments and international best practices while considering the specific conditions prevalent in India.

Structures are subjected to various loads, environmental effects and material degradation over time, which can impact their performance and safety. Hence, monitoring of structures has been gaining attention. Monitoring of structures is seen now along three directions, namely health monitoring, performance monitoring and safety monitoring. Structural health monitoring (SHM) involves monitoring of the condition of the materials of the structure, structural performance monitoring (SPM) involves monitoring of the response of the structure (without any damage induced in it) to daily loads the structure, such as vibrations of the structure under wind loads and structural safety monitoring involves monitoring the response of the structure when subjected to loads that damage the structure, such as, strong ground shaking of the structure during an earthquake, or air pressure induced under blast. In all cases, the engineer needs to assess the structure to identify actions needed towards repairing, correcting, protecting or retrofitting the structure, as the case be.

Monitoring of structures is a crucial process that enables the continuous or periodic assessment of structural integrity, ensuring early detection of potential damage and preventing catastrophic failures. It provides real-time insights into the behaviour of buildings and infrastructure under different conditions by utilizing advanced sensors, data acquisition systems and analytical models. The need for monitoring the structures arises from the increasing complexity of modern structures, aging infrastructure and the demand for enhanced safety and serviceability. It helps in identifying structural anomalies, degradation patterns and potential failure modes before they become critical. The system of monitoring of structures should be adopted to validate analytical models and evaluate techniques to ensure accurate assessment and decision-making. Implementing monitoring systems in buildings and structures where necessary enhances durability, reduces maintenance costs and supports proactive intervention strategies, ultimately improving structural resilience and public safety.

There has been a growing awareness among the consultants, academicians, researchers and practice engineers for design and construction of wind sensitive structures. In order to augment the available limited good quality meteorological wind data and structural response data, it is necessary to conduct full scale measurements in the field. Thus, as emphasized in the previous revision, all individuals and organizations responsible for putting-up of tall structures are encouraged to provide instrumentation in their existing and new structures at different elevations (at least at two levels) to continuously measure and monitor wind data. The instruments are required to collect data on wind direction, wind speed and structural response of the structure due to wind (with the help of accelerometer, strain gauges, etc). Such instrumentation in tall structures shall not in any way affect or alter the functional behaviour of such structures. The data so collected shall be very valuable in evolving more accurate wind loading of structures.

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The information contained in this Section is largely based on the following Indian Standards:

IS 875	Code of practice for design loads (other than earthquake) for buildings and structures:
Part 1 : 2026	Dead loads-unit weights of building material and stored materials (*third revision*) (*under preparation*)
Part 2 : 1987	Imposed loads (*second revision*)
Part 3 : 2015	Wind loads (*third revision*)
Part 4 : 2021	Snow loads (*third revision*)
Part 5 : 1987	Special loads and load combinations (*second revision*)
IS 4991 (Part 1) : 2024	Criteria for blast resistant design of structures: Part 1 Above-ground explosions (*first revision*)

This Section shall be read together with Sections 2 to 8 of Part C 'Structural Design' of this NBCS.

A special publication, SP 64 (S&T) : 2001 'Explanatory Handbook on Indian Standard Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures IS 875 (Part 3) : 1987' is also available. This publication gives detailed background information on the provisions for wind loads and the use of these provisions for arriving at the wind loads on buildings and structures while evaluating their structural safety. But, in view of revision of IS 875 (Part 3) in 2015, applicability of this revised standard and this Section of the NBCS shall be taken as final in case of conflict of provisions with SP 64.

Reference may also be made to the Vulnerability Atlas of India, 2011 and Landslide Hazard Zonation Map of India, 2003, published by Building Materials and Technology Promotion Council, Ministry of Urban Development and Poverty Alleviation, Government of India. The Vulnerability Atlas contains information pertaining to each State and Union Territory of India, on (a) seismic hazard map, (b) cyclone and wind map, (c) flood prone area map and (d) housing stock vulnerability table for each district indicating for each house type the level of risk to which it could be subjected. The Atlas can be used to identify areas in each district of the country which are prone to high risk from more than one hazard. The information will be useful in establishing the need of developing housing designs to resist the combination of such hazards.

All standards, whether given herein above or cross-referred to in the main text of this Section, are subject to revision. The parties to agreement based on this Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

As the regulation of land development and buildings is a State subject and is dealt by the States and local bodies such as urban local bodies within their jurisdiction through building regulatory documents like building regulations, building byelaws and fire regulations, this document is voluntary in nature and is non-binding. The implementation depends on adoption by concerned parties or stipulations in a contract or by appropriate adoption by the concerned authorities.

For the purpose of deciding whether a particular requirement of this section is complied with, the final value observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 2022 'Rules for rounding of numerical values (*second revision*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this section.

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Important Explanatory Note for Users of the NBCS

In any Part/Section of this NBCS, where reference is made to **'good practice'** in relation to design, constructional procedures or other related information and where reference is made to **"accepted standard"** in relation to material specification, testing, or other related information, the Indian Standards listed at the end of the Part/Section shall be used as a guide to the interpretation.

At the time of publication, the editions indicated in the standards were valid. All standards are subject to revision and parties to agreements based on any Part/ Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

In the list of standards given at the end of a Part/Section, the number appearing within parentheses in the first column indicates the number of the reference of the standard in the Part/Section. For example:

a) Good practices [C-1(1)] refers to the Indian Standard(s) give at serial number (1) of the list of standards given at the end of this Part/Section 1, that is, 8888 : 2020 'Requirements of low income housing for urban areas — Guide (second revision)'

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 1 LOADS AND LOAD EFFECTS

1 SCOPE

This NBCS (Part C/Section 1) covers basic design loads to be assumed in the design of buildings. The loads specified in this section are minimum loads to be considered for design purposes in the design of buildings.

2 DEAD LOAD

2.1 Assessment of Dead Load

The dead load in a building shall comprise the weight of all walls, partitions, floors and roofs and shall include the weights of all other permanent constructions (including built-in partitions, finishes, cladding and other similarly incorporated architectural and structural items, weight of fixed service equipment) in the building.

2.2 The list of dead loads of various construction materials and stored materials are given in Annex A.

2.3 Gardening and Landscaping

Elements on roof gardening or landscaping, including soil, plants and drainage layers, along with other features like walkways, fences and walls, are fixed in place and thus classified as dead loads. While the weight of walkways, fences and walls remains constant. The weight of soil and drainage layers, which support plant growth, can vary significantly due to their ability to absorb and retain water. For load calculations, when the weight adds to other structural loads, the dead load should be based on fully saturated soil and drainage layers to account for maximum weight. If the weight is used to counteract uplift forces, calculations should assume the dry weight of soil and drainage layers. Vegetative or landscaped roof areas may retain even more water than fully saturated soil, either from rainfall or irrigation. This additional water retention should be included in the design to accurately assess load variations.

In the design calculation of overturning, uplifting and sliding effects, the beneficial effects of soil shall not be considered. When the soil is used as dead weight and locked permanently, the same may be considered.

3 IMPOSED LOAD

3.1 This clause covers imposed loads to be assumed in the design of buildings. The imposed loads specified herein are minimum loads which should be taken into consideration for the purpose of structural safety of buildings.

NOTE — This Section does not cover detailed provisions for loads incidental to construction and special cases of vibration, such as moving machinery, heavy acceleration from cranes, hoists and the like. Such loads shall be dealt with individually in each case.

3.2 Terminology

3.2.1 For the purpose of imposed loads specified herein, the following definitions shall apply:

3.2.1.1 *Assembly buildings* — These shall include any building or part of a building where groups of people congregate or gather for amusement, recreation, social, religious, patriotic, civil, travel and similar purposes; for example, theatres, motion picture houses, assembly halls, conference/meeting halls, city halls, marriage halls, town halls, auditoria, exhibition halls, museums, skating rinks, gymnasiums, restaurants (also used as assembly halls), place of worship, dance halls, club rooms, passenger stations and terminals of air, surface and other public transportation services, recreation piers and stadia, etc.

3.2.1.2 *Business buildings* — These shall include any building or part of a building, which is used for transaction of business (other than that covered by mercantile buildings); for keeping of accounts and records for similar purposes; offices, banks, professional establishments, court houses and libraries shall be classified in this group so far as principal function of these is transaction of public business and the keeping of books and records.

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NOTE — The office buildings are the buildings primarily to be used as an office or for office purposes; 'office purposes' include the purpose of administration, clerical work, handling money, telephone and telegraph operating and operating computers, calculating machines, 'clerical work' includes writing, book-keeping, sorting papers, typing, filing, duplicating, drawing of matter for publication and the editorial preparation of matter for publication, etc.

3.2.1.3 *Dwellings* — These shall include any building or part occupied by members of single/multi-family units with independent cooking facilities. These shall also include apartment houses (flats).

3.2.1.4 *Educational buildings* — These shall include any building used for school, college or day-care purposes involving assembly for instruction, education or recreation and which is not covered by assembly buildings.

3.2.1.5 *Imposed load* — The load assumed to be produced by the intended use or occupancy of a building including the weight of movable partitions, distributed and concentrated loads, loads due to impact and vibration and dust loads but excluding wind, seismic, snow and other loads due to temperature changes, creep, shrinkage, differential settlement, etc.

3.2.1.6 *Industrial buildings* — These shall include any building or a part of a building or structure, in which products or materials of various kinds and properties are fabricated, assembled, manufactured or processed, for example, assembly plants, industrial laboratories, dry cleaning plants, power plants, generating units, pumping stations, fumigation chambers, laundries, buildings or structures in gas plants, refineries, dairies and saw-mills, etc.

3.2.1.7 *Institutional buildings* — These shall include any building or a part thereof, which is used for purposes such as medical or other treatment in case of persons suffering from physical and mental illness, disease or infirmity; care of infants, convalescents or aged persons and for penal or correctional detention in which the liberty of the inmates is restricted. Institutional buildings ordinarily provide sleeping accommodation for the occupants. It includes hospitals, sanatoria, custodial institutions or penal institutions like jails, prisons and reformatories.

3.2.1.8 *Occupancy or use group* — The principal occupancy for which a building or part of a building is used or intended to be used; for the purpose of classification of a building according to occupancy, an occupancy shall be deemed to include subsidiary occupancies which are contingent upon it.

3.2.1.9 *Mercantile buildings* — These shall include any building or a part of a building which is used as shops, stores, market for display and sale of merchandize either wholesale or retail. Office, storage and service and facilities incidental to the sale of merchandize and located in the same building shall be included under this group.

3.2.1.10 *Residential buildings* — These shall include any building in which sleeping accommodation is provided for normal residential purposes with or without cooking or dining or both facilities (except buildings under institutional buildings). It includes lodging or rooming houses, one or two-family private dwellings, dormitories, apartment houses (flats) and hotels.

3.2.1.11 *Storage buildings* — These shall include any building or part of a building used primarily for the storage or sheltering of goods, wares or merchandize, like warehouses, cold storages, freight depots, transit sheds, store houses, garages, hangers, truck terminals, grain elevators, barns and stables.

3.3 Imposed Loads on Floors Due to Use and Occupancy

3.3.1 *Imposed Loads*

The imposed loads to be assumed in the design of buildings shall be the greatest loads that probably will be produced by the intended use or occupancy, but shall not be less than the equivalent minimum loads specified in Table 1 subject to any reductions permitted in **3.3.2**.

Floors shall be investigated for both the uniformly distributed load (UDL) and the corresponding concentrated load specified in Table 1 and designed for the most adverse effects but they shall not be considered to act simultaneously. The concentrated loads specified in Table 1 may be assumed to act over an area of 0.3 m × 0.3 m. However, the concentrated loads need not be considered where the floors are capable of effective lateral distribution of this load.

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All other structural elements shall be investigated for the effects of uniformly distributed loads on the floors specified in Table 1.

NOTES

1 Where, in Table 1, no values are given for concentrated load, it may be assumed that the tabulated distributed load is adequate for design purposes.

2 The loads specified in Table 1 are equivalent uniformly distributed loads on the plan area and provide for normal effects of impact and acceleration. They do not take into consideration special concentrated loads and other loads.

3 Where the use of an area or floor is not provided in Table 1, the imposed load due to the use and occupancy of such an area shall be determined from the analysis of loads resulting from:

a) weight of the probable assembly of persons;

b) weight of the probable accumulation of equipment and furnishing;

c) weight of the probable storage materials; and

d) impact factor, if any.

4 While selecting a particular loading, the possible change in use or occupancy of the building should be kept in view. Designers should not necessarily select in every case the lower loading appropriate to the first occupancy. In doing this they might introduce considerable restrictions in the use of the building at a later date and thereby reduce its utility.

5 The loads specified herein, which are based on estimations, may be considered as the characteristic loads for the purpose of limit state method of design till such time statistical data are established based on load surveys to be conducted in the country.

6 When an existing building is altered by an extension in height or area, all existing structural parts affected by the addition shall be strengthened where necessary and all new structural parts shall be designed to meet the requirements for building hereafter erected.

7 The loads specified in the Section does not include loads incidental to construction. Therefore, close supervision during construction is essential to ensure that overloading of the building due to loads by way of stacking of building materials or use of equipment (for example, cranes and trucks) during construction or loads which may be induced by floor to floor propping in multi-storeyed construction, does not occur. However, if construction loads were of short duration, permissible increase in stresses in the case of working stress method or permissible decrease in load factors in limit state method, as applicable to relevant design codes, may be allowed for.

8 The loads in Table 1 are grouped together as applicable to buildings having separate principal occupancy or use. For a building with multiple occupancies, the loads appropriate to the occupancy with comparable use shall be chosen from other occupancies.

9 Regarding loading on lift machine rooms including storage space used for repairing lift machines, designers should go by the recommendations of lift manufacturers for the present. Regarding loading due to false ceiling, the same should be considered as imposed loads on the roof/floor to which it is fixed.

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Table 1 Imposed Floor Loads for Different Occupancies

(*Clauses* 3.3.1, 3.3.1.1 *and* 3.4.1)

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m^2	Concentrated Load kN
(1)	(2)	(3)	(4)
i)	*Residential Buildings*		
	a) Dwelling houses:		
	1) All rooms and kitchens	2.0	1.8
	2) Toilets and bath rooms	2.0	–
	3) Corridors, passages, staircases including fire escapes and store rooms	3.0	4.5
	4) Balconies	3.0	1.5 per metre run concentrated at the outer edge
	b) Dwelling units planned and executed in accordance with the accepted standard [C-1(1)] only:		
	1) Habitable rooms, kitchens and toilets and bath rooms	2.0	1.8
	2) Corridors, passages and staircases including fire escapes	3.0	4.5
	3) Balconies	3.0	1.5 per metre run concentrated at the outer edge
	c) Hotels, hostels, boarding houses, lodging houses, dormitories and residential clubs:		
	1) Living rooms, bed rooms and dormitories	2.0	1.8
	2) Kitchen and laundries	3.0	4.5

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Table 1 (*Continued*)

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m^2	Concentrated Load kN
(1)	(2)	(3)	(4)
	3) Billiards room and public lounges	3.0	2.7
	4) Store rooms	5.0	4.5
	5) Dining rooms, cafeterias and restaurants	4.0	2.7
	6) Office rooms	2.5	2.7
	7) Rooms for indoor games	3.0	1.8
	8) Baths and toilets	2.0	–
	9) Corridors, passages staircases including fire escapes and lobbies – as per the floor serviced (excluding stores and the like) but not less than	3.0	4.5
	10) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
	d) Boiler rooms and plant rooms – to be calculated but not less than	5.0	6.7
	e) Garages:		
	1) Garage floors (including parking area and repair workshops) for passenger cars and vehicles not exceeding 2.5 tonne gross weight, including access ways and ramps – to be calculated but not less than	2.5	9.0

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Table 1 (*Continued*)

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m^2	Concentrated Load kN
(1)	(2)	(3)	(4)
	i) Without stack	5.0	9.0
	ii) With 2 level stack		
	2) Garage floors for vehicles not exceeding 4.0 tonne gross weight (including access ways and ramps) – to be calculated but not less than	5.0	9.0
	f) Roof top solar panels (with or without frames and with anchor blocks)	1.75	1.75
	g) Roof top HVAC loading	9.0	9.0
ii)	*Educational Buildings*		
	a) Class rooms and lecture rooms (not used for assembly purposes)	3.0	2.7
	b) Dining rooms, cafeterias and restaurants	3.0[1)]	2.7
	c) Offices, lounges and staff rooms	2.5	2.7
	d) Dormitories	2.0	2.7
	e) Projection rooms (with audio-visual facilities)	5.0	–
	f) Kitchens	3.0	4.5
	g) Toilets and bath rooms	2.0	–
	h) Store rooms	5.0	4.5
	j) Libraries and archives:		
	1) Stack room/stack area	6.0 kN/m^2 for a minimum height of 2.2 m + 2.0 kN/m^2 per metre height beyond 2.2 m	4.5
	2) Reading rooms (without separate storage)	4.0	4.5
	3) Reading rooms (with separate storage)	3.0	4.5
	k) Boiler rooms and plant rooms – to be	4.0	4.5

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Table 1 (*Continued*)

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m^2	Concentrated Load kN
(1)	(2)	(3)	(4)
	calculated but not less than		
	m) Corridors, passages, lobbies, staircases including fire escapes – as per the floor serviced (without accounting for storage and projection rooms) but not less than	4.0	4.5
	n) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
iii)	*Institutional Buildings*		
	a) Bed rooms, wards, dressing rooms, dormitories and lounges	2.0	1.8
	b) Kitchens, laundries and laboratories	3.0	4.5
	c) Dining rooms, cafeterias and restaurants	3.0[1)]	2.7
	d) Toilets and bathrooms	2.0	–
	e) X-ray rooms, operating rooms and general storage areas – to be calculated but not less than	3.0	4.5
	f) Office rooms and O.P.D. rooms	2.5	2.7
	g) Corridors, passages, lobbies, staircases including fire escapes – as per the floor serviced (without accounting for storage and projection rooms) but not less than	4.0	4.5
	j) Boiler rooms and plant rooms – to be	5.0	4.5

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Table 1 (*Continued*)

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m^2	Concentrated Load kN
(1)	(2)	(3)	(4)
	calculated but not less than		
	k) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
iv)	*Assembly Buildings*		
	a) Assembly areas :		
	1) With fixed seats[2)]	4.0	–
	2) Without fixed seats	5.0	3.6
	c) Restaurants (subject to assembly), museums and art galleries and gymnasia	4.0	4.5
	d) Projection rooms (with audio-visual facilities)	5.0	–
	e) Stages	5.0	4.5
	f) Office rooms, kitchens and laundries	3.0	4.5
	g) Dressing rooms	2.0	1.8
	h) Lounges and billiards rooms	2.0	2.7
	j) Toilets and bathrooms	2.0	–
	k) Corridors, passages and staircases including fire escapes	4.0	4.5
	m) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
	n) Boiler rooms and plant rooms including weight of machinery	7.5	4.5
	p) Corridors, passages, subject to loads greater than from crowds, such as wheeled vehicles, trolleys and the like corridors, staircases and passages in grandstands	5.0	4.5

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Table 1 (*Continued*)

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m^2	Concentrated Load kN
(1)	(2)	(3)	(4)
v)	*Business and Office Buildings* (*see also* 3.2.1)		
	a) Rooms for general use with separate storage	2.5	2.7
	b) Rooms without separate storage	4.0	4.5
	c) Banking halls	3.0	2.7
	d) Business computing machine rooms (with fixed computers or similar	3.0	4.0
	e) equipment)		
	f) UPS, server and electrical panel	5.0	4.5
	g) Records/files store rooms and storage space	5.0	4.5
	h) Vaults and strong rooms – to be calculated but not less than	5.0	4.5
	j) Cafeterias and dining rooms	3.0[1)]	2.7
	k) Kitchens	3.0	2.7
	m) Corridors, passages, lobbies, staircases including fire escapes – as per the floor serviced (excluding stores) but not less than	4.0	4.5
	n) Bath and toilets rooms	2.0	–
	p) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
	q) Stationary stores	4.0 for each metre of storage height	9.0
	r) Boiler rooms and plant rooms – to be calculated but not less than	5.0	6.7
	s) Libraries	*See* Sl No. (ii)	

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Table 1 (*Continued*)

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m^2	Concentrated Load kN
(1)	(2)	(3)	(4)
vi)	*Mercantile Buildings*		
	a) Retail shops	4.0	3.6
	b) Wholesale shops – to be calculated but not less than	6.0	4.5
	c) Office rooms	2.5	2.7
	d) Dining rooms, restaurants and cafeterias	3.0[1)]	2.7
	e) Toilets	2.0	–
	f) Kitchens and laundries	3.0	4.5
	g) Boiler rooms and plant rooms – to be calculated but not less than	5.0	6.7
	h) Corridors, passages, stair-cases including fire escapes and lobbies	4.0	4.5
	j) Corridors, passages, staircases subject to loads greater than from crowds, such as wheeled vehicles, trolleys and the like	5.0	4.5
	k) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
vii)	*Industrial Buildings*[3)]		
	a) Work areas without machinery/ equipment	2.5	4.5
	a) Work areas with machinery/equipment[4)] (To be calculated but not less than)		
	1) Light duty	5.0	4.5
	2) Medium duty	7.0	4.5
	3) Heavy duty	10.0	4.5

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Table 1 (*Continued*)

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m^2	Concentrated Load kN
(1)	(2)	(3)	(4)
	b) Boiler rooms and plant rooms – to be calculated but not less than	5.0	6.7
	c) Cafeterias and dining rooms	3.0[1)]	2.7
	d) Corridors, passages, stair cases including fire escapes	4.0	4.5
	e) Corridors, passages, lobbies, staircases subject to machine loads and wheeled vehicles – to be calculated but not less than	5.0	4.5
	f) Kitchens	3.0	4.5
	g) Toilets and bathrooms	2.0	–
viii)	*Storage Buildings*[4)]		
	a) Storage rooms (other than cold storage) and warehouses – to be calculated based on the bulk density of materials stored but not less than	2.4 kN/m^2 per metre of storage height with a minimum of 7.5 kN/m^2	7.0
	b) Cold storage – to be calculated but not less than	5.0 kN/m^2 per metre of storage height with a minimum of 15 kN/m^2	9.0
	c) Corridors, passages, staircases including fire escapes – as per the floor serviced but not less than	4.0	4.5
	d) Corridors, passages subject to loads greater than from crowds, such as wheeled vehicles, trolleys and the like	5.0	4.5
	e) Boiler rooms and plant rooms	7.5	4.5
ix)	Refuge area and disaster shelter	3.0	-

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Table 1 (*Concluded*)

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL) kN/m^2	Concentrated Load kN
(1)	(2)	(3)	(4)
x)	Datacentres		
	a) Data hall Load (With or without base isolation)		
	1) Tier 1	7.2	
	2) Tier 2	8.4	
	3) Tier 3	12	
	4) Tier 4	12	
	b) Data hall floor Hanging load (With or without base isolation)		
	1) Tier 1	1.2	
	2) Tier 2	1.2	
	3) Tier 3	2.4	
	4) Tier 4	2.4	

[1)] Where unrestricted assembly of persons is anticipated, the value of UDL should be increased to 4.0 kN/m^2.

[2)] With fixed seats implies that the removal of the seating and the use of the space for other purposes is improbable. The maximum likely load in this case is, therefore, closely controlled.

[3)] The loading in industrial buildings (workshops and factories) varies considerably and so three loadings under the terms 'light', 'medium' and 'heavy' are introduced in order to allow for more economical designs but the terms have no special meaning in themselves other than the imposed load for which the relevant floor is designed. It is, however important particularly in the case of heavy weight loads, to assess the actual loads to ensure that they are not in excess of 10 kN/m^2; in case where they are in excess, the design shall be based on the actual loadings.

[4)] For various mechanical handling equipment which are used to transport goods, as in warehouses, workshops, store rooms, etc, the actual load coming from the use of such equipment shall be ascertained and design should cater to such loads.

3.3.1.1 *Load application*

The uniformly distributed loads specified in Table 1 shall be applied as static loads over the entire floor area under consideration or a portion of the floor area, whichever arrangement produces critical effects on the structural elements as provided in respective design codes.

In the design of floors, the concentrated loads are considered to be applied in the positions which produce the maximum stresses and where deflection is the main criterion in the positions which produce the maximum deflections. Concentrated load, when used for the calculation of bending and shear, are assumed to act at a point. When used for the calculation of local effects such as crushing or punching, they are assumed to act over an actual area of application of 0.3 m × 0.3 m.

3.3.1.2 *Loads due to light partitions*

In office and other buildings, where actual loads due to light partitions cannot be assessed at the time of planning the floors and the supporting structural members shall be designed to carry, in addition to other loads, uniformly distributed loads per square metre of not less than 33.33 percent of weight per metre run of finished partitions, subject to a minimum of 1 kN/m^2, provided total weight of partition walls per m^2 of the wall area does not exceed 1.5 kN/m^2 and the total weight per metre length is not greater than 4.0 kN.

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3.3.2 *Reduction in Imposed Loads on Floors*

3.3.2.1 *For members supporting floors*

Except as provided for in 3.3.2.1(a), the following reductions in assumed total imposed loads on the floors may be made in designing columns, load bearing walls, piers, their supports and foundations:

a) No reduction shall be made for any plant or machinery which is specifically allowed for, or for buildings for storage purposes, warehouses and garages. But, for other buildings, where the floor is designed for an imposed floor load of 5.0 kN/m^2 or more, the reductions shown in 3.3.2.1 may be taken provided that the loading assumed is not less than it would have been, if all the floors had been designed for 5.0 kN/m^2 with no reductions; and

NOTE — In case, if the reduced load in the lower floor is lesser than the reduced load in the upper floor, then the reduced load of the upper floor will be adopted.

b) An example is given in Annex B illustrating the reduction of imposed loads in a multi-storeyed building in the design of column members.

Number of Floors (*Including the Roof*) *to be Carried by Member Under Consideration*	*Reduction in Total Distributed Imposed Load on All Floors to be Carried by the Member Under Consideration* Percent
1	0
2	10
3	20
4	30
5 to 10	40
Over 10	50

3.3.2.2 *For beams in each floor level*

Where a single span of beam, girder or truss supports not less than 50 m^2 of floor at one general level, the imposed floor load may be reduced in the design of the beams, girders or trusses by 5 percent for each 50 m^2 area supported subject to a maximum reduction of 25 percent. But, no reduction shall be made in any of the following types of loads:

a) Any superimposed moving load;

b) Any actual load due to machinery or similar concentrated loads;

c) The additional load in respect of partition walls; and

d) Any impact or vibration.

NOTE — The above reduction does not apply to beams, girders or trusses supporting roof loads.

3.3.3 *Posting of Floor Capacities*

Where a floor or part of a floor of a building has been designed to sustain a uniformly distributed load exceeding 3.0 kN/m^2 and in assembly, business, mercantile, industrial or storage buildings, a permanent notice in the form shown below indicating the actual uniformly distributed and/or concentrated loadings for which the floor has been structurally designed shall be posted in a conspicuous place in a position adjacent to such floor or on such part of a floor.

NOTES

1 The lettering of such notice shall be embossed or cast suitably on a tablet whose least dimension shall not be less than 0.25 m and located not less than 1.5 m above floor level with lettering of a minimum size of 25 mm.

2 If a concentrated load or a bulk load has to occupy a definite position on the floor, the same could also be indicated in the table.

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DESIGNED IMPOSED FLOOR LOADING

Distributed……..kN/m^2

Concentrated…….kN

Label Indicating Designed Imposed Floor Loading

3.4 Imposed Loads on Roofs

3.4.1 *Imposed Loads on Various Types of Roofs*

On flat roofs, sloping roofs and curved roofs, the imposed loads due to use and occupancy of the buildings and the geometry of the types of roofs shall be as given in Table 2.

Table 2 Imposed Loads on Various Types of Roofs

(*Clauses* 3.4.1 *and* 3.4.3)

Sl No.	Type of Roof	Imposed Load Measured on Plan Area	Minimum Imposed Load Measured on Plan
(1)	(2)	(3)	(4)
i)	Flat, sloping or curved roof with slopes up to and including 10 degrees		
	a) Access provided	1.5 kN/m^2	3.75 kN uniformly distributed over any span of one metre width of the roof slab and 9 kN uniformly distributed over the span of any beam or truss or wall
	b) Access not provided except for maintenance	0.75 kN/m^2	1.9 kN uniformly distributed over any span of one metre width of the roof slab and 4.5 kN uniformly distributed over the span of any beam or truss or wall
ii)	Sloping roof with slope greater than 10 degrees	For roof membrane sheets or purlins – 0.75 kN/m^2 less 0.02 kN/m^2 for every degree increase in slope over 10 degrees	Subject to a minimum of 0.4 kN/m^2
iii)	Curved roof with slope of line obtained by joining springing point to the crown with the horizontal, greater than 10 degrees	$(0.75 - 0.52\ \alpha^2)$ kN/m^2 where $\alpha = h/l$	Subject to a minimum of 0.4 kN/m^2

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Table 2 (*Concluded*)

Sl No.	Type of Roof	Imposed Load Measured on Plan Area	Minimum Imposed Load Measured on Plan
(1)	(2)	(3)	(4)
		h = height of the highest point of the structure measured from its springing; and l = chord width of the roof if singly curved and shorter of the two sides if doubly curved. Alternatively, where structural analysis can be carried out for curved roofs of all slopes in a simple manner applying the laws of statistics, the curved roofs shall be divided into minimum 6 equal segments and for each segment imposed load shall be calculated appropriate to the slope of the chord of each segment as given in (i) and (ii).	

NOTES

1 The loads given above do not include loads due to snow, rain, dust collection, etc. The roof shall be designed for imposed loads given above or for snow/rain load, whichever is greater.

2 For special types of roofs with highly permeable and absorbent material, the contingency of roof material increasing in weight due to absorption of moisture shall be provided for.

Roofs of buildings used for promenade or incidental to assembly purposes shall be designed for the appropriate imposed floor loads given in Table 1 for the occupancy.

3.4.2 *Concentrated Load on Roof Coverings*

To provide for loads incidental to maintenance, unless otherwise specified by the Engineer-in-Charge, all roof coverings (other than glass or transparent sheets made of fibre glass) shall be capable of carrying an incidental load of 0.90 kN concentrated on an area of 1 250 mm^2 so placed as to produce maximum stresses in the covering. The intensity of the concentrated load may be reduced with the approval of the Engineer-in-Charge, where it is ensured that the roof coverings would not be traversed without suitable aids. In any case, the roof coverings shall be capable of carrying the loads in accordance with **3.4.1**, **3.4.3** and **3.4.4** and wind load.

3.4.3 *Loads Due to Rain*

On surfaces whose positioning, shape and drainage system are such as to make accumulation of rain water possible, loads due to such accumulation of water and the imposed loads for the roof as given in Table 2 shall be considered separately and the more critical of the two shall be adopted in the design.

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3.4.4 *Dust Loads*

In areas prone to settlement of dust on roofs (example, steel plants, cement plants), provision for dust load equivalent to probable thickness of accumulation of dust may be made.

3.4.5 *Loads on Members Supporting Roof Coverings*

Every member of the supporting structure which is directly supporting the roof covering(s) shall be designed to carry the more severe of the following loads except as provided in **3.4.5.1**:

a) The load transmitted to the members from the roof covering(s) in accordance with **3.4.1**, **3.4.3** and **3.4.4**; and

b) An incidental concentrated load of 0.90 kN concentrated over a length of 125 mm placed at the most critical positions on the member.

NOTE — Where it is ensured that the roofs would be traversed only with the aid of planks and ladders capable of distributing the loads on them to two or more supporting members, the intensity of concentrated load indicated in **3.4.5(b)** may be reduced to 0.5 kN with the approval of the Engineer-in-Charge.

3.4.5.1 In case of sloping roofs with slope greater than 10°, members supporting the roof purlins, such as trusses, beams, girders, etc, may be designed for two-thirds of the imposed load on purlin or roofing sheets.

3.5 Imposed Horizontal Loads on Parapets, Balustrades and Other Appurtenances Fixed to Structure and on Grandstands

3.5.1 *Parapets, Parapet Walls, Balustrades and other Appurtenances Fixed to the Structure such as Grab Bars, Fixed Ladders and Guardrails in Stilt Parking*

Parapets, parapet walls, balustrades and other appurtenances fixed to the structure such as grab bars, fixed ladders and guardrails in stilt parking, together with the members which give them structural support, shall be designed for the minimum loads given in Table 3. These are expressed as horizontal forces acting at handrail or coping level. These loads shall be considered to act vertically also but not simultaneously with the horizontal forces. The values given in Table 3 are minimum values and where values for actual loadings are available, they shall be used instead.

Table 3 Horizontal Loads on Parapets, Parapet Walls and Balustrades
(*Clause* 3.5.1)

Sl No.	Usage Area	Intensity of Horizontal Load kN/m Run	Concentrated Load (For each 3 m Length) kN
(1)	(2)	(3)	(4)
i)	Light access stairs, gangways and like, not more than 600 mm wide	0.25	—
ii)	Light access stairs, gangways and like, more than 600 mm wide; stairways, landings, balconies and parapet walls (private and part of dwellings)	0.35	—
iii)	a) All other stairways, landings and balconies and all parapets and handrails to roofs [except those subject to overcrowding covered under (iv)]	0.75	0.89
	b) Panel fillers	0.20	0.22

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Table 3 (*Concluded*)

Sl No.	Usage Area	Intensity of Horizontal Load kN/m Run	Concentrated Load (For each 3 m Length) kN
(1)	(2)	(3)	(4)
iv)	Parapets and balustrades:		
	a) In places of assembly, such as restaurants and bars, retail and public areas, not likely to be overcrowded	1.5	1.5
	b) In places of assembly, such as retail areas, theatres, cinemas, bars, auditoria, shopping malls, discothèques, places of worship, likely to be overcrowded	3.0	1.5
v)	Grab bars	—	1.11
vi)	Fixed ladders	—	1.33
vii)	Guardrail systems and handrail assemblies	0.73	0.89

NOTE — In the case of guard parapets on a floor of multi-storeyed car park or crash barriers provided in certain buildings for fire escape, the value of imposed horizontal load (together with impact load) may be determined. For (vii), this load need not be considered for (a) one- and two- family dwellings and (b) factory, industrial and storage occupancies, in areas that serve occupant load not greater than 22 kN.

3.5.2 *Grandstands and the Like*

Grandstands, stadia, assembly platforms, reviewing stands and the like shall be designed to resist a horizontal force applied to seats of 0.35 kN per linear metre along the line of seats and 0.15 kN per linear metre perpendicular to the line of the seats. These loadings need not be applied simultaneously. Platforms without seats shall be designed to resist a minimum horizontal force of 0.25 kN/m^2 of plan area.

3.6 Loading Effects Due to Impact and Vibration

The crane loads to be considered under imposed loads shall include the vertical loads, eccentricity effects induced by vertical loads, impact factors, lateral and longitudinal braking forces acting across and along the crane rails, respectively.

3.6.1 *Impact Allowance for Lifts, Hoists and Machinery*

The imposed loads specified in 3.3.1 shall be assumed to include adequate allowance for ordinary impact conditions. However, for structures carrying loads which induce impact or vibration, as far as possible, calculations shall be made for increase in the imposed load due to impact or vibration. In the absence of sufficient data for such calculation, the increase in the imposed loads shall be as follows:

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Structures	*Impact Allowance* Percent *Min*
a) For frames supporting lifts and hoists	100
b) For foundations, footings and piers supporting lifts and hoisting apparatus	40
c) For supporting structures and foundations for light machinery, shaft or motor units	20
d) For supporting structures and foundations for reciprocating machinery or power units	50

3.6.2 *Concentrated Imposed Loads with Impact and Vibration*

3.6.2.1 Concentrated imposed loads with impact and vibration which may be due to installed machinery shall be considered and provided for in the design. The impact factor shall not be less than 20 percent which is the amount allowable for light machinery.

3.6.2.2 Provision shall also be made for carrying any concentrated equipment loads while the equipment is being installed or moved for servicing and repairing.

3.6.3 *Impact Allowance for Crane Girders*

For crane gantry girders and supporting columns, the impact allowances (given below) shall be deemed to cover all forces set up by vibration, shock from slipping of slings, kinetic action of acceleration and retardation and impact of wheel loads.

Type of Load	*Additional Load*
a) Vertical loads for electric overhead cranes	25 percent of maximum static loads for crane girders for all class of cranes 25 percent for columns supporting Class III and Class IV cranes 10 percent for columns supporting Class I and Class II cranes No additional load for design of foundations
b) Vertical loads for hand operated cranes	10 percent of maximum wheel loads for crane girders only
c) Horizontal forces transverse to rails:	
1) For electric overhead cranes with trolley having rigid mast for suspension of lifted weight (such as soaker crane, stripper crane, etc)	10 percent of weight of crab and the weight lifted by the cranes, acting on any one crane track rail, acting in either direction and equally distributed amongst all the wheels on one side of rail track For frame analysis, this force, calculated as above, shall be applied on one side of the frame at a time in either direction
2) For all other electric overhead cranes and hand operated cranes	5 percent of weight of crab and the weight lifted by the cranes, acting on any one crane track rail, acting in either direction and equally distributed amongst the wheels on one side of rail track For the frame analysis, the force, calculated as above, shall be applied on one side of the frame at a time in either direction

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	Type of Load	*Additional Load*
d)	Horizontal traction forces along the rails for overhead cranes, either electrically operated or hand operated	5 percent of all static wheel loads

Forces specified in (c) and (d) shall be considered as acting at the rail level and being appropriately transmitted to the supporting system. Gantry girders and their vertical supports shall be designed on the assumption that either of the horizontal forces in (c) and (d) may act at the same time as the vertical load.

NOTE — *See* as per good practice [C-1(2)] for classification (Class I to Class IV) of cranes.

3.6.3.1 *Overloading factors in crane supporting structures*

For all ladle cranes and charging cranes where there is possibility of overloading from production considerations, an overloading factor of 10 percent of the maximum wheel loading shall be taken.

3.6.4 *Crane Load Combinations*

In the absence of any specific indications, the load combinations shall be as indicated below.

3.6.4.1 *Vertical loads*

In an aisle, where more than one crane is in operation or has provision for more than one crane in future, the following load combinations shall be taken for vertical loading:

a) two adjacent cranes working in tandem with full load and with overloading according to **3.6.3.1**; and

b) for long span gantries, where more than one crane can come in the span, the girder shall be designed for one crane fully loaded with overloading according to **3.6.3.1** plus as many loaded cranes as can be accommodated on the span but without taking into account overloading according to **3.6.3**(**a**) to give the maximum effect.

3.6.4.2 *Lateral surge*

For design of columns and foundations, supporting crane girders, the following crane combinations shall be considered:

a) *For single bay frames* — Effect of one crane in the bay giving the worst effect shall be considered for calculation of surge force; and

b) *For multi-bay frames* — Effect of two cranes working, one each in any of two bays in the cross section to give the worst effect shall be considered for calculation of surge force.

3.6.4.3 *Tractive force*

3.6.4.3.1 Where one crane is in operation with no provision for future crane, tractive force from only one crane shall be taken.

3.6.4.3.2 Where more than one crane is in operation or there is provision for future crane, tractive force from two cranes giving maximum effect shall be considered.

NOTE — Lateral surge force and longitudinal tractive force acting across and along the crane rail respectively shall not be assumed to act simultaneously. However, if there is only one crane in the bay, the lateral and longitudinal forces may act together simultaneously with vertical loads.

3.7 Rooftop Helipad

Elevated helipad structures shall be designed as per the latest guidelines of Directorate General of Civil Aviation and International Civil Aviation Organization's Heliport Manual.

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3.7.1 *Structural Design*

Elevated helipads may be designed for a specific helicopter type though greater operational flexibility will be obtained from a classification system of design. The final approach and take-off area (FATO) should be designed for the largest or heaviest type of helicopter that is anticipated to use the helipad and account taken of other types of loading such as personnel, freight, snow, refueling equipment, etc. For the purpose of design, it is to be assumed that the helicopter will land on two main wheels, irrespective of the actual number of wheels in the undercarriage, or on two skids, if they are fitted. The loads imposed on the structure should be taken as point loads at the wheel centres as shown below:

Sl No.	*Helicopter Category*	*Maximum Take-off Mass*		*Point Load for Each Wheel*	*Under-Carriage Wheel Centres*	*Super-imposed Load* S_{Ha}	*Super-imposed Load* S_{Hb}
		kg	kN	kN	M	kN/m^2	kN/m^2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	1	Up to 2300	Up to 22.6	12.0	1.75	0.5	1.5
ii)	2	2 301 to 5000	22.6 to 49.2	25.0	2.0	0.5	2.0
iii)	3	5001 to 9000	49.2 to 88.5	45.0	2.5	0.5	2.5
iv)	4	9001 to 13500	88.5 to 133.0	67.0	3.0	0.5	3.0
v)	5	13501 to 19500	133.0 to 192.0	96.0	3.5	0.5	3.0
vi)	6	19501 to 27000	192.0 to 266.0	133.0	4.5	0.5	3.0

3.7.2 The FATO should be designed for the worse condition derived from consideration of the following two cases.

3.7.3 *Case A — Helicopter on Landing*

When designing a FATO on an elevated helipad and in order to cover the bending and shear stresses that result from a helicopter touching down, the following should be taken into account:

a) *Dynamic load due to impact on touchdown* — This should cover the normal touchdown, with a rate of descent of 1.8 m/s, which equates to the serviceability limit state. The impact load is then equal to 1.55 times the maximum take-off mass of the helicopter;

The emergency touchdown should also be covered at a rate of descent of 3.6m/s (12 ft/s), which equates to the ultimate limit state. The partial safety factor in this case should be taken as 1.66. Therefore,

Ultimate design load = 1.66 × service load

= (1.66 × 1.5) maximum take-off mass

= 2.5 maximum take-off mass

The sympathetic response factor discussed at **3.7.3(b)** below should be applied.

b) *Sympathetic response on the FATO* — The dynamic load should be increased by a structural response factor dependent upon the natural frequency of the platform slab when considering the design of supporting beams and columns. This increase in loading will usually apply only to slabs with one or more freely supported edges. It is recommended that the average structural response factor (R) of 1.3 should be used in determining the ultimate design load;

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c) *Over-all superimposed load on the FATO* (S_{Ha}) — To allow for snow load, personnel, freight and equipment loads, etc, in addition to wheel loads, an allowance of 0.5 KN/m^2 should be included in the design;

d) *Lateral load on the platform supports* — The supports of the platform should be designed to resist a horizontal point load equivalent to 0.5 maximum take-off mass of the helicopter, together with the wind loading [*see* (*f*)], applied in the direction which will provide the greater bending moments;

e) *Dead load of structural members* — The partial safety factor to be used for the dead load should be taken as 1.4;

f) *Wind loading* — Shall be as per 4; and

g) *Punching shear* — Check for the punching shear of an undercarriage wheel or skid using the ultimate design load with a contact area of 64.5 × 10^3 mm².

3.7.4 *Case B — Helicopter at Rest*

When designing a FATO on an elevated helipad and in order to cover the bending and shear stresses from a helicopter at rest, the following should be taken into account:

a) *Dead load of the helicopter* — Each structural element must be designed to carry the point load, in accordance with the table under 3.7.1, from the two main wheels or skids applied simultaneously in any position on the FATO so as to produce the worst effect from both bending and shear;

b) *Over-all superimposed load* (S_{Hb}) — In addition to wheel loads, an allowance for over-all superimposed load given in the table under 3.7.1, over the area of the FATO, should be included in the design; and

c) *Dead load on structural members and wind loading* — The same factors should be included in the design for these items as given for Case A.

NOTE — The above design loads for helicopters at rest are summarized below:

Design Load for Helicopter on Landing — Case A

i) *Superimposed loads*	
a) Helicopter	2.5 L_H distributed as two point loads at the wheel centres for the helicopter category given in the informal table under 3.7.1. Average value for R = 1.3.
b) Lateral load	1.6 $\frac{L_H}{2}$ applied horizontally in any direction.
c) Over-all superimposed load	Load at platform level together with the maximum wind loading. 1.4 S_{Ha} over the whole area of the platform (S_{Ha} given in the informal table under 3.7.1).
ii) Dead load	1.4G
iii) Wind loading	1.4W
iv) Punching shear check	2.5 $L_H R$ load over tyre skid contact area of 64.5 × 10^3 mm^2.

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Design Load For Helicopter at Rest — Case B

i) *Superimposed loads*	
a) Helicopter	1.6 L_H distributed as two point loads at the wheel centres for the helicopter category given in the informal table under 3.7.1.
b) Over-all superimposed load (Personnel, freight, etc)	1.6 S_{Hb} over the whole area of the platform. S_{Hb} given in the informal table under 3.7.1.
ii) Shear check	Check as appropriate

Symbol	*Meaning*
L_H	Maximum take-off mass of helicopter
G	Dead load of structure
W	Wind loading
R	Structural response factor
S_{Ha}	Superimposed load — Case A
S_{Hb}	Superimposed load — Case B

Load Type	*Partial Load Factors*
Dynamic load (ultimate design load)	2.5
Imposed load	1.6
Dead load	1.4
Wind loading	1.4

3.7.5 Normally, the upper load limit of the selected helicopter category should be used for design purposes; however, to avoid over-design, the upper limit in any band may be exceeded by up to 10 percent if the maximum take-off mass of a helicopter falls just into the next higher category.

3.8 Fire Tenders and Emergency Vehicles

Where a structure or portions of a structure are accessed and loaded by fire department access vehicles and other similar emergency vehicles, the structure shall be designed for the greater of the following loads:

a) the actual operational loads, including outrigger reactions and contact areas of the vehicles as stipulated and provided by the building official;

b) the contact area of wheel dimensions to be used considering 45° distribution at the reinforcement of slab considering the bear coat on the slab and any soil filling.

or

in absence of any data, the bare slab loading may be considered as 15 kN/m^2 to 20 kN/m^2.

c) when the fire tender load is combined with gravity loads and the same can be considered as accidental load with the partial load factor of 1.05; and

d) fire tender load need not be combined with lateral loads.

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4 WIND LOAD

4.1 General

4.1.1 This clause gives wind forces and their effects (static and dynamic) that should be taken into account when designing buildings, structures and components thereof.

4.1.2 Wind speeds vary randomly both in time and space and hence assessment of wind loads and response predictions are very important in the design of several buildings and structures. A large majority of structures met with in practice do not however, suffer wind induced oscillations and generally do not require to be examined for the dynamic effects of wind. For such normal, short and heavy structures, estimation of loads using static wind analysis has proved to be satisfactory. The details of this method involving important wind characteristics such as the basic wind speeds, terrain categories, modification factors, wind pressure and force coefficients, etc, are given in **4.4** and **4.5**.

4.1.3 Nevertheless, there are various types of structures or their components such as some tall buildings which require investigation of wind induced oscillations. The influence of dynamic velocity fluctuations on the along wind loads (drag loads) for these structures shall be determined using Gust Factor Method, included in **4.8**. A method for calculation of across wind response of tall buildings is included in **4.8.3**.

4.1.4 This Section also applies to buildings or other structures during erection/construction and the same shall be considered carefully during various stages of erection/construction. In locations where the strongest winds and icing may occur simultaneously, loads on structural members, cables and ropes shall be calculated by assuming an ice covering based on climatic and local experience.

4.1.5 Wind is air in motion relative to the surface of the earth. The primary cause of wind is traced to earth's rotation and differences in terrestrial radiation. The radiation effects are primarily responsible for convection either upwards or downwards. The wind generally blows horizontal to the ground at high wind speeds. Since vertical components of atmospheric motion are relatively small, the term ‘wind’ denotes almost exclusively the horizontal wind; vertical winds are always identified as such. The wind speeds are assessed with the aid of anemometers or anemographs which are installed at meteorological observatories at heights generally varying from 10 m to 30 m above ground.

4.1.6 Very strong winds (more than 80 km/h) are generally associated with cyclonic storms, thunderstorms, dust storms or vigorous monsoons. A feature of the cyclonic storms over the Indian area is that they rapidly weaken after crossing the coasts and move as depressions/lows inland. The influence of a severe storm after striking the coast does not; in general exceed about 60 km, though sometimes, it may extend even up to 120 km. Very short duration hurricanes of very high wind speeds called Kal Baisaki or Norwesters occur fairly frequently during summer months over North East India.

4.1.7 The wind speeds recorded at any locality are extremely variable and in addition to steady wind at any time, there are effects of gusts which may last for a few seconds. These gusts cause increase in air pressure but their effect on stability of the building may not be so important; often, gusts affect only part of the building and the increased local pressures may be more than balanced by a momentary reduction in the pressure elsewhere. Because of the inertia of the building, short period gusts may not cause any appreciable increase in stress in main components of the building although the walls, roof sheeting and individual cladding units (glass panels) and their supporting members such as purlins, sheeting rails and glazing bars may be more seriously affected. Gusts can also be extremely important for design of structures with high slenderness ratios.

4.1.8 The liability of a building to high wind pressures depends not only upon the geographical location and proximity of other obstructions to air flow but also upon the characteristics of the structure itself.

4.1.9 The effect of wind on the structure as a whole is determined by the combined action of external and internal pressures acting upon it. In all cases, the calculated wind loads act normal to the surface to which they apply.

4.1.10 The stability calculations as a whole shall be done considering the combined effect, as well as separate effects of imposed loads and wind loads on vertical surfaces, roofs and other part of the building above general roof level.

4.1.11 Buildings shall also be designed with due attention to the effects of wind on the comfort of people inside

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and outside the buildings.

4.1.12 Wind borne debris present in cyclone or high speed wind has been causing a significant amount of damage to building envelopes. The actual in-service performance of fenestration assemblies and impact protective systems in areas prone to severe high speed wind depends on many factors. The main factors that affect the actual loading on building surfaces during a severe windstorm include varying wind direction, duration of the wind event, height above ground, building shape, terrain and surrounding structures. The resistance of fenestration or protective systems assemblies to wind loading after impact depends upon the product design, installation, load magnitude, duration and repetition. The method for determination of performance relating to safety of external building envelope like glazed curtain walls, exterior doors, exterior windows, impact protective systems (shutters, screens), skylight, wall cladding system, etc, impacted by wind borne debris, for buildings and other structures located in geographic regions that are prone to cyclone or extreme wind events shall be in accordance with [C-1(3)].

NOTES

1 This Section does not apply to buildings or structures with unconventional shapes, unusual locations and abnormal environmental conditions that have not been covered in this NBCS. Special investigations are necessary in such cases to establish wind loads and their effects. Wind tunnel studies may also be required in such situations.

2 In the case of tall structures with unsymmetrical geometry, the designs may have to be checked for torsional effects due to wind pressure.

3 The provisions given in good practice [C-1(4)] may be suitably referred by the design engineer/planner for reference.

4.2 Notations

The notations to be followed unless otherwise specified in relevant clauses under wind loads are given in **C-1**.

4.3 Terminology

4.3.1 For the purpose of wind loads, the following definitions shall apply.

4.3.1.1 *Angle of attack* — An angle between the direction of wind and a reference axis of the structure.

4.3.1.2 *Breadth* — It means horizontal dimension of the building measured normal to the direction of wind.

4.3.1.3 *Depth* — It means the horizontal dimension of the building measured in the direction of the wind.

NOTE — Breadth and depth are dimensions measured in relation to the direction of wind, whereas length and width are dimensions related to the plan.

4.3.1.4 *Developed height* — It is the height of upward penetration of the velocity profile in a new terrain. At large fetch lengths, such penetration reaches the gradient height, above which the wind speed may be taken to be constant. At lesser fetch lengths, a velocity profile of a smaller height but similar to that of the fully developed profile of that terrain category has to be taken, with the additional provision that the velocity at the top of this shorter profile equal to that of the un-penetrated earlier velocity profile at that height.

4.3.1.5 *Effective frontal area* — The projected area of the structure normal to the direction of wind.

4.3.1.6 *Element of surface area* — The area of surface over which the pressure coefficient is taken to be constant.

4.3.1.7 *Force coefficient* — A non-dimensional coefficient such that the total wind force on a body is the product of the force coefficient, the dynamic pressure of the incident design wind speed and the reference area over which the force is required.

NOTE — When the force is in the direction of the incident wind, the non-dimensional coefficient will be called as ‘drag coefficient’. When the force is perpendicular to the direction of incident wind, the non-dimensional coefficient will be called as ‘lift coefficient’.

4.3.1.8 *Ground roughness* — The nature of the earth’s surface as influenced by small scale obstructions such as trees and buildings (as distinct from topography) is called ground roughness.

4.3.1.9 *Gust* — A positive or negative departure of wind speed from its mean value, lasting for not more than, say, 2 min over a specified interval of time.

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4.3.1.10 *Peak gust* — A peak gust or peak gust speed is the wind speed associated with the maximum amplitude.

4.3.1.11 *Fetch length* — It is the distance measured along the wind from a boundary at which a change in the type of terrain occurs. When the changes in terrain types are encountered (such as, the boundary of a town or city, forest, etc), the wind profile changes in character but such changes are gradual and start at ground level, spreading or penetrating upwards with increasing fetch length.

4.3.1.12 *Gradient height* — It is the height above the mean ground level at which the gradient wind blows as a result of balance among pressure gradient force, Coriolis force and centrifugal force. For the purpose of this Section, the gradient height is taken as the height above the mean ground level, above which the variation of wind speed with height need not be considered.

4.3.1.13 *Tall building* — A building with a height more than or equal to 50 m or having a height to smaller lateral dimension more than 6.

4.3.1.14 *Low rise building* — A building having its height less than 20 m.

4.3.1.15 *Mean ground level* — The mean ground level is the average horizontal plane of the area enclosed by the boundaries of the structure.

4.3.1.16 *Pressure coefficient* — It is the ratio of the difference between the pressure acting at a point on the surface and the static pressure of the incident wind to the design wind pressure, where the static and design wind pressures are determined at the height of the point considered after taking into account the geographical location, terrain conditions and shielding effect. The pressure coefficient is also equal to $[1 - (V_p/V_z)^2]$, where V_p is the actual wind speed at any point on the structure at a height corresponding to that of V_z.

NOTE — Positive sign of the pressure coefficient indicates pressure acting towards the surface and negative sign indicates pressure acting away from the surface.

4.3.1.17 *Return period* — It is the number of years, reciprocal of which gives the probability of extreme wind exceeding a given wind speed in anyone year.

4.3.1.18 *Shielding effect* — Shielding effect or shielding refers to the condition where wind has to pass along some structure(s) or structural element(s) located on the upstream wind side, before meeting the structure or structural element under consideration. A factor called ‘shielding factor’ is used to account for such effects in estimating the force on the shielded structures.

4.3.1.19 *Suction* — It means pressure less than the atmospheric (static) pressure and is taken to act away from the surface.

4.3.1.20 *Solidity ratio* — It is equal to the effective area (projected area of all the individual elements) of a frame normal to the wind direction divided by the area enclosed by the boundary of the frame normal to the wind direction.

NOTE — Solidity ratio is to be calculated for individual frames.

4.3.1.21 *Terrain category* — It means the characteristics of the surface irregularities of an area which arise from natural or constructed features. The categories are numbered in increasing order of roughness.

4.3.1.22 *Topography* — The nature of the earth's surface as influenced by the hill and valley configurations.

4.3.1.23 *Velocity profile* — The variation of the horizontal component of the atmospheric wind speed at different heights above the mean ground level is termed as velocity profile.

4.4 Wind Speed

4.4.1 *Nature of Wind in Atmosphere*

In general, wind speed in the atmospheric boundary layer increases with height from zero at ground level to maximum at a height called the gradient height. There is usually a slight change in direction (Ekman effect) but this is ignored in this Section. The variation with height depends primarily on the terrain conditions. However, the wind speed at any height never remains constant and it has been found convenient to resolve its instantaneous

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magnitude into an average or mean value and a fluctuating component around this average value. The average value depends on the average time employed in analyzing the meteorological data and this averaging time varies from few seconds to several minutes. The magnitude of fluctuating component of the wind speed, which is called gust, depends on the averaging time. In general, smaller the averaging interval more is the magnitude of the gust speed.

4.4.2 *Basic Wind Speed*

Figure 1 gives basic wind speed map of India, as applicable to 10 m height above mean ground level for different zones of the country. Basic wind speed is based on peak gust velocity averaged over a short time interval of about 3 s and corresponds to mean heights above ground level in an open terrain (Category 2). Basic wind speeds presented in Fig. 1 have been worked out for a 50 year return period. Basic wind speed for some important cities/towns is also given in Annex D.

4.4.3 *Design Wind Speed* (V_z)

The basic wind speed (V_b) for any site shall be obtained from Fig. 1 and shall be modified to include the following effects to get design wind speed, V_z at any height z, for the chosen structure:

a) Risk level;

b) Terrain roughness and height of structure;

c) Local topography; and

d) Importance factor for the cyclonic region.

It can be mathematically expressed as follows:

$$V_z = V_b\, k_1\, k_2\, k_3\, k_4$$

where

V_z = design, in m/s, wind speed at height z;

k_1 = probability factor (risk coefficient) (*see* **4.4.3.1**);

k_2 = terrain roughness and height factor (*see* **4.4.3.2**);

k_3 = topography factor (*see* **4.4.3.3**); and

k_4 = importance factor for the cyclonic region (*see* **4.4.3.4**).

NOTE — Wind speed may be taken as constant up to a height of 10 m. However, pressures for buildings less than 10 m high may be reduced by 20 percent for evaluating stability and design of the framing.

4.4.3.1 *Risk coefficient* (k_1 factor)

Figure 1 gives basic wind speeds for terrain Category 2 as applicable at 10 m above ground level based on 50 years mean return period. The suggested life period to be assumed in design and the corresponding k_1 factors for different class of structures for the purpose of design are given in Table 4. In the design of buildings and structures, a regional basic wind speed having a mean return period of 50 years shall be used except as specified in the Note of Table 4.

4.4.3.2 *Terrain, height factor* (k_2 factor)

4.4.3.2.1 *Terrain*

Selection of terrain categories shall be made with due regard to the effect of obstructions which constitute the ground surface roughness. The terrain category used in the design of a structure may vary depending on the direction of wind under consideration. Wherever sufficient meteorological information is available about the nature of wind direction, the orientation of any building or structure may be suitably planned.

Terrain in which a specific structure stands shall be assessed as being one of the following terrain categories:

a) *Category* 1 — Exposed open terrain with few or no obstructions and in which the average height of any

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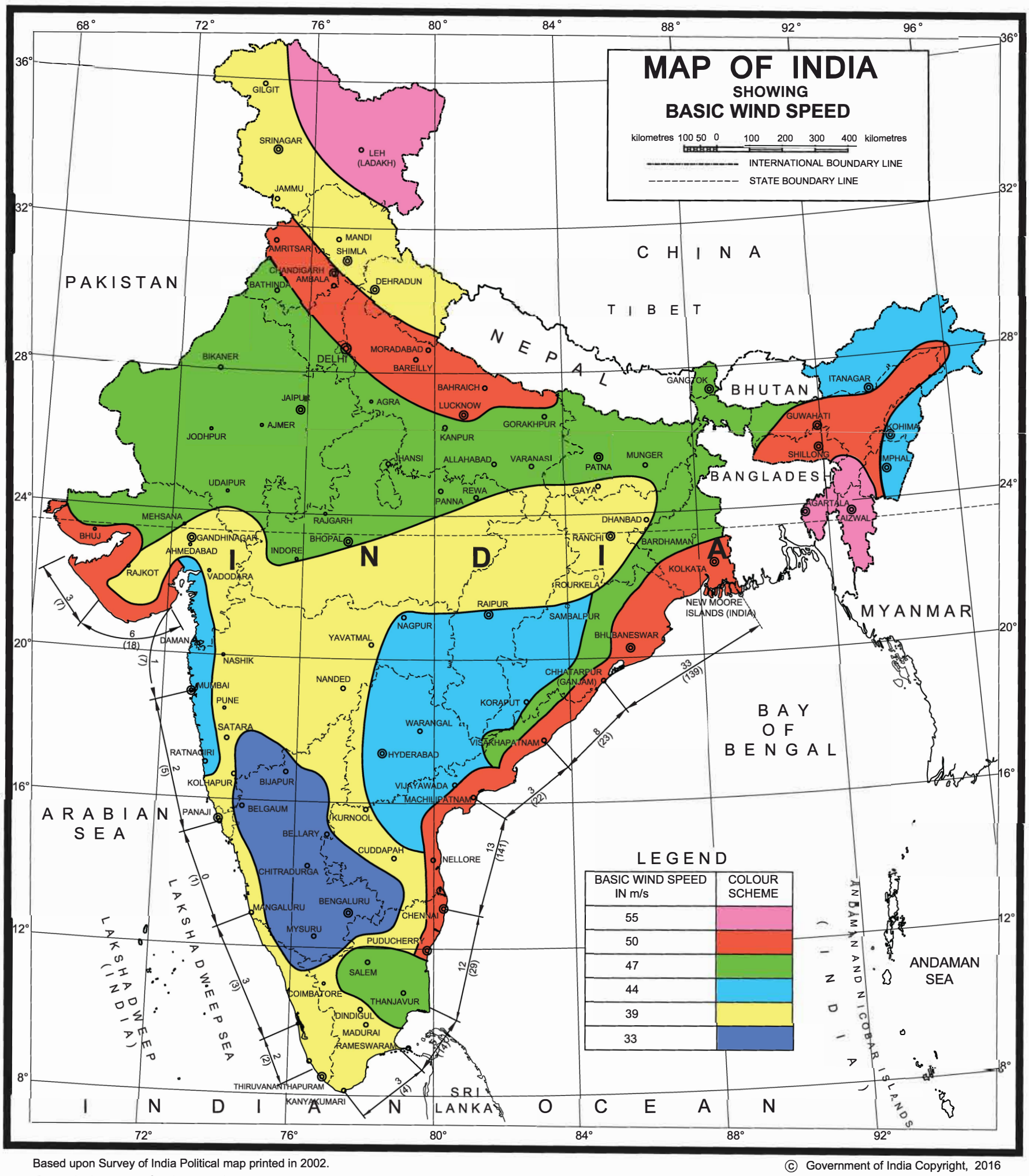

Based upon Survey of India Political map printed in 2002.

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The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate baseline.

The interstate boundaries between Arunachal Pradesh, Assam and Meghalaya shown on this map are as interpreted from the North-Eastern Areas (Reorganization) Act, 1971, but have yet to be verified.

The external boundaries and coastlines of India agree with the Record/Master Copy certified by Survey of India.

The responsibility for the correctness of internal details rest with the publisher.

NOTES

1 The occurrence of a tornado is possible in virtually any part of India. They are particularly more severe in the northern parts of India. The recorded number of these tornados is too small to assign any frequency. The devastation caused by a tornado is due to exceptionally high winds about its periphery, and the sudden reduction in atmospheric pressure at its centre, resulting in an explosive outward pressure on the elements of the structure. the regional basic wind speeds do not include any specific allowance for tornados. It is not the usual practice to allow for the effect of tornados unless special requirements are called for as in the case of important structures such as, nuclear power reactors and satellite communication towers.

2 The total number of cyclonic storms that have struck different sections of east and west coasts are included in Fig. 1, based on available records for the period from 1877 to 1982. The figures above the lines (between the stations) indicate the total number of severe cyclonic storms with or without a core of hurricane winds (speeds above 87 km/h) and the figures in the brackets below the lines indicate the total number of cyclonic storms. Their effect on land is already reflected in the basic wind speeds specified in Fig. 1. These have been included only as additional information.

FIG. 1 BASIC WIND SPEED IN m/s (BASED ON 50-YEARS RETURN PERIOD)

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object surrounding the structure is less than 1.5 m. The equivalent aerodynamic roughness height, ($z_{0,1}$) for this terrain is 0.002 m. Typically this category represents open sea-coasts and flat plains without trees;

b) *Category* 2 — Open terrain with well scattered obstructions having heights generally between 1.5 m and 10 m. The equivalent aerodynamic roughness height, ($z_{0,2}$) for this terrain is 0.02 m. This is the criterion for measurement of regional basic wind speeds and represents airfields, open park lands and undeveloped sparsely built-up outskirts of towns and suburbs. Open land adjacent to sea coast may also be classified as Category 2 due to roughness of large sea waves at high winds;

c) *Category* 3 — Terrain with numerous closely spaced obstructions having the size of buildings/structures up to 10 m in height with or without a few isolated tall structures. The equivalent aerodynamic roughness height, ($z_{0,3}$) for this terrain is 0.2 m. This category represents well wooded areas and shrubs, towns and industrial areas full or partially developed. It is likely that the, next higher category than this will not exist in most design situations and that selection of a more severe category will be deliberate; and

d) *Category* 4 — Terrain with numerous large high closely spaced obstructions. The equivalent aerodynamic roughness height, ($z_{0,4}$) for this terrain is 2.0 m. This category represents large city centres, generally with obstructions above 25 m and well developed industrial complexes.

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Table 4 Risk Coefficients for Different Classes of Structures in Different Wind Speed Zones

(*Clause* 4.4.3.1)

Sl No.	Class of Structure	Mean Probable Design Life of Structure in Years	k_1 Factor for Basic Wind Speed (in m/s) of					
			33	39	44	47	50	55
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	All general buildings and structures and ground based free standing tower up to 50 m	50	1.0	1.0	1.0	1.0	1.0	1.0
ii)	Temporary sheds, structures such as those used during construction operations (for example forL-work and false work), structures during construction stages and boundary walls	5	0.82	0.76	0.73	0.71	0.70	0.67
iii)	Buildings and structures presenting a low degree of hazard to life and property in the event of failure, such as isolated towers in wooded areas, farm buildings other than residential buildings	25	0.94	0.92	0.91	0.90	0.90	0.89
iv)	Important buildings and structures, such as hospitals, communication buildings/towers and power plant structures	100	1.05	1.06	1.07	1.07	1.08	1.08

NOTE — The factor k_1 is based on statistical concepts which take into account the degree of reliability required and period of time in years during which there will be exposure to wind, that is, life of the structure. Whatever wind speed is adopted for design purposes, there is always a probability (however small) that it may be exceeded in a storm of exceptional violence; more the period of years over which there is exposure to the wind, more is the probability. Larger return periods ranging from 100 to 1 000 years (implying lower risk level) in association with larger periods of exposure may have to be selected for exceptionally important structures, such as, nuclear power reactors and satellite communication towers. Equation given below may be used in such cases to estimate k_1 factors for different periods of exposure and chosen probability of exceedance (risk level). The probability level of 0.63 is normally considered sufficient for design of buildings and structures against wind effects and the values of k_1 corresponding to this risk level are given above.

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Table 4 (*Concluded*)

$$k_1 = \frac{X_{N,P}}{X_{50,0.63}} = \frac{A - B\left[l_n \left\{ -\frac{1}{N} l_n (1 - P_N) \right\} \right]}{A + 4B}$$

where

N = mean probable design life of structure in years;

P_N = risk level in N consecutive years (probability that the design wind speed is exceeded;

$X_{N,P}$ = at least once in N successive years), nominal value = 0.63; and

$X_{50,0.63}$ = extreme wind speed for given values of N and P_N; .

A and *B* have the following values for different basic wind speed zones:

Zone m/s	*A** m/s	*B** m/s
33	23.1 (83.2)	2.6 (9.2)
39	23.3 (84.0)	3.9 (14.0)
44	24.4 (88.0)	5.0 (18.0)
47	24.4 (88.0)	5.7 (20.5)
50	24.7 (88.8)	6.3 (22.8)
55	25.2 (90.8)	7.6 (27.3)

* Values of *A* and *B* in 'km/h' are given within bracket

4.4.3.2.2 *Variation of wind speed with height in different terrains* (k_2)

Table 5 gives multiplying factors (k_2) by which the basic wind speed given in Fig. 1 shall be multiplied to obtain the wind speed at different heights, in each terrain category.

Table 5 Factors to Obtain Design Wind Speed Variation with Height in Different Terrains

(*Clause* 4.4.3.2.2)

Sl No.	Height (z), m	Terrain and Height Multiplier (k_2)			
		Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
(1)	(2)	(3)	(4)	(5)	(6)
i)	10	1.05	1.00	0.91	0.80
ii)	15	1.09	1.05	0.97	0.80
iii)	20	1.12	1.07	1.01	0.80
iv)	30	1.15	1.12	1.06	0.97
v)	50	1.20	1.17	1.12	1.10
vi)	100	1.26	1.24	1.20	1.20
vii)	150	1.30	1.28	1.24	1.24
viii)	200	1.32	1.30	1.27	1.27
ix)	250	1.34	1.32	1.29	1.28
x)	300	1.35	1.34	1.31	1.30

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Table 5 (*Concluded*)

Sl No.	Height (z), m	Terrain and Height Multiplier (k_2)			
		Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
(1)	(2)	(3)	(4)	(5)	(6)
xi)	350	1.35	1.35	1.32	1.31
xii)	400	1.35	1.35	1.34	1.32
xiii)	450	1.35	1.35	1.35	1.33
xiv)	500	1.35	1.35	1.35	1.34

NOTE — For intermediate values of height z in a given terrain category, use linear interpolation.

4.4.3.2.3 *Terrain categories in relation to the direction of wind*

The terrain category used in the design of a structure may vary depending on the direction of wind under consideration. Where sufficient meteorological information is available, the basic wind speed may be varied for specific wind direction.

4.4.3.2.4 *Changes in terrain categories*

The velocity profile for a given terrain category does not develop to full height immediately with the commencement of that terrain category but develop gradually to height (h_x) which increases with the fetch or upwind distance (x).

a) *Fetch and developed height relationship* — The relation between the developed height (h_x) and the fetch (x) for wind-flow over each of the four terrain categories may be taken as given in Table 6; and

b) For structures of heights more than the developed height (h_x) in Table 6, the velocity profile may be determined in accordance with the following:

   1) The less or least rough terrain; or

   2) The method described in Annex E.

Table 6 Fetch and Developed Height Relationship

(*Clause* 4.4.3.2.4)

Sl No.	Fetch (x) km	Developed Height (h_x) m			
		Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
(1)	(2)	(3)	(4)	(5)	(6)
i)	0.2	12	20	35	60
ii)	0.5	20	30	35	95
iii)	1	25	45	80	130
iv)	2	35	65	110	190
v)	5	60	100	170	300
vi)	10	80	140	250	450

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Table 6 (*Concluded*)

Sl No.	Fetch (x) km	Developed Height (h_x) m			
		Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
(1)	(2)	(3)	(4)	(5)	(6)
vii)	20	120	200	350	500
viii)	50	180	300	400	500

4.4.3.3 *Topography* (k_3 *factor*)

The basic wind speed V_b given in Fig. 1 takes into account the general level of site above sea level. This does not allow for local topographic features such as hills, valleys, cliffs, escarpments, or ridges which can significantly affect wind speed in their vicinity. The effect of topography is to accelerate wind near the summits of hills or crests of cliffs, escarpments or ridges and decelerate the wind in valleys or near the foot of cliffs, steep escarpments, or ridges.

The effect of topography shall be significant at a site when the upwind slope (θ) is more than about 3° and below that, the value of k_3 may be taken to be equal to 1.0. The value of k_3 is confined in the range of 1.0 to 1.36 for slopes more than 3°. A method of evaluating the value of k_3 for values more than 1.0 is given in Annex F. It may be noted that the value of k_3 varies with height above ground level, with a maximum near the ground and reducing to 1.0 at higher levels.

4.4.3.4 *Importance factor for cyclonic region* (k_4 *factor*)

The east coast of India is relatively more vulnerable for occurrences of severe cyclones. On the west coast, Gujarat is vulnerable for severe cyclones. Studies of wind speed and damage to buildings and structures point to the fact that the speeds given in the basic wind speed map are often exceeded during the cyclones. The effect of cyclonic storms is largely felt in a belt of approximately 60 km width at the coast. In order to ensure better safety of structures in this region (60 km wide on the east coast as well as on the Gujarat coast), the values of k_4 are stipulated as applicable according to the importance of the structure:

	k_4
Structures of post-cyclone importance for emergency services (such as cyclone shelters, hospitals, schools, communication towers, etc)	1.30
Industrial structures	1.15
All other structures	1.00

NOTE — For guidelines related to estimation of cyclonic factor (k4) as given in good practice [C-1(5)] may be referred.

4.4.4 *Hourly Mean Wind Speed*

The hourly mean wind speed at height z, for different terrains can be obtained as:

$$\overline{V}_{z,H} = \overline{k}_{2,i} V_b$$

where

$\overline{k}_{2,i}$ = hourly mean wind speed factor for terrain category 'i'.

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$$= 0.1\,423\left[\ln\left(\frac{z}{z_{0,i}}\right)\right]\left(z_{0,i}\right)^{0.0706}$$

The design hourly mean wind speed at height z can be obtained as:

$$\overline{V}_{z,d} = \overline{V}_{z,H} k_1 k_3 k_4 \quad = V_b k_1 \overline{k}_{2,i} k_3 k_4$$

4.4.5 *Turbulence Intensity*

The turbulence intensity variations with height for different terrains can be obtained using the relations given below:

a) *Terrain category* 1

$$I_{z,1} = 0.3507 - 0.0535 \log_{10}\left(\frac{z}{z_{0,1}}\right)$$

b) *Terrain category* 2

$$I_{z,2} = I_{z,1} + \frac{1}{7}\left(I_{z,4} - I_{z,1}\right)$$

c) *Terrain category* 3

$$I_{z,3} = I_{z,1} + \frac{3}{7}\left(I_{z,4} - I_{z,1}\right)$$

d) *Terrain category* 4

$$I_{z,4} = 0.466 - 0.1358 \log_{10}\left(\frac{z}{z_{0,4}}\right)$$

4.4.6 *Off-shore Wind Velocity*

Cyclonic storms form far away from the sea coast and gradually reduce in speed as they approach the sea coast. Cyclonic storms generally extend up to about 60 km inland after striking the coast. Their effect on land is already reflected in basic wind speeds specified in Fig. 1. The influence of wind speed off the coast up to a distance of about 200 km may be taken as 1.15 times the value on the nearest coast in the absence of any definite wind data. The factor 1.15 shall be used in addition to k_4.

4.5 Wind Pressures and Forces on Buildings/Structures

4.5.1 *General*

The wind load on a building/structure shall be calculated for:

a) the building/structure as a whole;

b) individual structural elements as roofs and walls; and

c) individual cladding units including glazing and their fixings.

4.5.2 *Design Wind Pressure*

The wind pressure at any height above mean ground level shall be obtained by the following relationship between wind pressure and wind speed:

$$p_z = 0.6 V_z^2$$

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where

p_z = wind pressure at height z, in N/m^2; and

V_z = design wind speed at height z, in m/s.

The design wind pressure p_d can be obtained as:

$$p_d = K_d K_a K_c p_z$$

where

K_d = wind directionality factor;

K_a = area averaging factor; and

K_c = combination factor [*see* **4.5.3.3.13**].

The value of p_d, however shall not be taken as less than 0.70 p_z.

NOTES

1 The coefficient 0.6 (in SI units) in the above formula depends on a number of factors and mainly on the atmospheric pressure and air temperature. The value chosen corresponds to the average Indian atmospheric conditions.

2 K_d should be taken as 1.0 when considering local pressure coefficients.

4.5.2.1 *Wind directionality factor, K_d*

Considering the randomness in the directionality of wind and recognizing the fact that pressure or force coefficients are determined for specific wind directions, it is specified that for buildings, solid signs, open signs, lattice frameworks and trussed towers (triangular, square, rectangular) a factor of 0.90 may be used on the design wind pressure. For circular or near-circular forms this factor may be taken as 1.0.

For the cyclone affected regions also the factor K_d shall be taken as 1.0.

4.5.2.2 *Area averaging factor, K_a*

Pressure coefficients given in **4.5.3** are a result of averaging the measured pressure values over a given area. As the area becomes larger, the correlation of measured values decrease and *vice-versa*. The decrease in pressures due to larger areas may be taken into account as given in Table 7.

Table 7 Area Averaging Factor (K_a)

(*Clause* 4.5.2.2)

Sl No.	Tributary Area (A) m^2	Area Averaging Factor (K_a)[1)]
(1)	(2)	(3)
i)	≤ 10	1.0
ii)	25	0.9
iii)	≥ 100	0.8

[1)] Linear interpolation for intermediate values of tributary area, A is permitted.

4.5.2.2.1 *Tributary area*

a) *Overall structure* — For evaluating loads on frames the tributary area shall be taken as the centre to centre distances between frames multiplied by the individual panel dimension in the other direction together with overall pressure coefficients;

b) *Individual elements* — For beam type elements, purlins, etc, the tributary area shall be taken as effective

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span multiplied by spacing. The effective span is the actual span for mid span and cantilever load effects; and half the sum of adjacent spans for support moments and reactions;

c) For plate type elements, the area of individual plates between supports is taken as the tributary area; and

d) For glass cladding, individual pane area of glass is the tributary area.

4.5.3 *Pressure Coefficients*

The pressure coefficients are always given for a particular surface or part of the surface of a building/structure. The wind load acting normal to a surface is obtained by multiplying the area of that surface or its appropriate portion by the pressure coefficient (C_p) and the design wind pressure at the height of the surface from the ground. The average values of these pressure coefficients for some building shapes are given in 4.5.3.2 and 4.5.3.3.

Average values of pressure coefficients are given for critical wind directions in one or more quadrants. In order to determine the maximum wind load on the building/structure, the total load should be calculated for each of the critical directions shown from all quadrants. Where considerable variation of pressure occurs over a surface, it has been sub-divided and mean pressure coefficients given for each of its several parts.

In addition, areas of high local suction (negative pressure concentration) frequently occurring near the edges of walls and roofs are separately shown. Coefficients for the local effects should only be used for calculation of forces on these local areas affecting roof sheeting, glass panels and individual cladding units including their fixtures. They should not be used for calculating force on entire structural elements such as roof, walls or structure as a whole.

NOTES

1 The pressure coefficients given in different tables have been obtained mainly from measurements on models in wind tunnels and the great majority of data available has been obtained in conditions of relatively smooth flow. Where sufficient field data exists as in the case of rectangular buildings, values have been obtained to allow for turbulent flow.

2 In recent years, wall glazing and cladding design has been a source of major concern. Although of less consequence than the collapse of main structures, damage to glass can be hazardous and cause considerable financial losses.

3 For pressure coefficients for structures not covered herein, reference may be made to specialist literature on the subject or advice may be sought from specialists in the subject.

4.5.3.1 *Wind load on individual members*

When calculating the wind load on individual structural elements such as roofs and walls and individual cladding units and their fittings, it is essential to take account of the pressure difference between opposite faces of such elements or units. For clad structures, it is, therefore, necessary to know the internal pressure as well as the external pressure. Then the wind load, F (in N), acting in a direction normal to the individual structural element or cladding unit is:

$$F = (C_{pe} - C_{pi})\, A\, p_d$$

where

C_{pe} = external pressure coefficient;

C_{pi} = internal pressure coefficient;

A = surface area of structural element or cladding unit, in m^2; and

p_d = design wind pressure, in N/ m^2.

NOTES

1 If the surface design pressure varies with height, the surface areas of the structural element may be sub-divided so that the specified pressures are taken over appropriate areas.

2 Positive wind load indicates the force acting towards the structural element and negative away from it.

4.5.3.2 *Internal pressure coefficients*

Internal air pressure in a building depends upon the degree of permeability of cladding to the flow of air. The internal air pressure may be positive or negative depending on the direction of flow of air in relation to openings

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in the buildings.

a) In the case of buildings where the claddings permit the flow of air with openings not more than about 5 percent of the wall area but where there are no large openings, it is necessary to consider the possibility of the internal pressure being positive or negative. Two design conditions shall be examined, one with an internal pressure coefficient of + 0.2 and another with an internal pressure coefficient of - 0.2.

   The internal pressure coefficient is algebraically added to the external pressure coefficient and the analysis which indicates greater distress of the member shall be adopted. In most situations a simple inspection of the sign of external pressure will at once indicate the proper sign of the internal pressure coefficient to be taken for design.

   NOTE — The term normal permeability relates to the flow of air commonly afforded by claddings not only through open windows and doors, but also through the slits round the closed windows and doors and through chimneys, ventilators and through the joints between roof coverings, the total open area being less than 5 percent of area of the walls having the openings.

b) *Buildings/structures with medium and large openings* — Buildings/structures with medium and large openings may also exhibit either positive or negative internal pressure depending upon the direction of wind. Buildings/structures with medium openings between about 5 percent to 20 percent of wall area shall be examined for an internal pressure coefficient of + 0.5 and later with an internal pressure coefficient of - 0.5 and the analysis which produces greater distress of the member shall be adopted. Buildings/structures with large openings, that is, openings larger than 20 percent of the wall area shall be examined once with an internal pressure coefficient of + 0.7 and again with an internal pressure coefficient of - 0.7 and the analysis which produces greater distress of the member shall be adopted.

Buildings/structures with one open side or opening exceeding 20 percent of wall area may be assumed to be subjected to internal positive pressure or suction similar to those of buildings with large openings. A few examples of buildings with one side openings are shown in Fig. 2 indicating values of internal pressure coefficients with respect to the direction of wind.

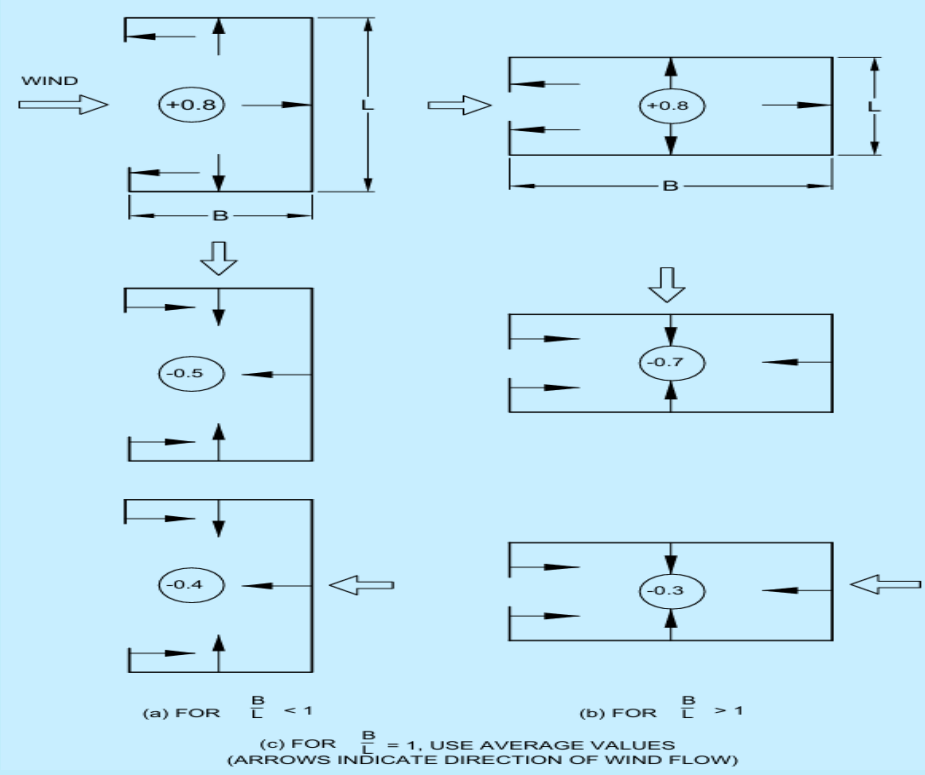

FIG. 2 VALUES OF INTERNAL PRESSURE COEFFICIENTS WITH RESPECT TO THE DIRECTION OF WIND FOR BUILDINGS WITH ONE SIDE OPENINGS

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4.5.3.3 *External pressure coefficients*

4.5.3.3.1 *Walls*

The average external pressure coefficient for the walls of clad buildings of rectangular plan shall be as given in Table 8. In addition, local pressure concentration coefficients are also given.

4.5.3.3.2 *Pitched, hipped and monoslope roofs of clad buildings*

The average external pressure coefficients and pressure concentration coefficients for pitched roofs of rectangular clad building shall be as given in Table 9. Where no pressure concentration coefficients are given, the average coefficients shall apply. The pressure coefficients on the underside of any overhanging roof shall be taken in accordance with **4.5.3.3.5**.

NOTES

1 The pressure concentration shall be assumed to act outward (suction pressure) at the ridges, eaves, cornices and 90° corners of roofs.

2 The pressure concentration shall not be included with the net external pressure when computing overall loads.

3 For hipped roofs, pressure coefficients (including local values) may be taken on all the four slopes, as appropriate from Table 9 and be reduced by 20 percent for the hip slope.

For mono slope roofs of rectangular clad buildings, the average pressure coefficient and pressure concentration coefficient for mono slope (lean-to) roofs of rectangular clad buildings shall be as given in Table 10.

4.5.3.3.3 *Canopy roofs with* $(1/4 < h/w < 1$ *and* $1 < L/w < 3)$

The pressure coefficients are given in Table 11 and Table 12 separately for mono pitch and double pitch canopy roofs such as open-air parking garages, shelter areas, outdoor areas, railway platforms, stadiums and theatres. The coefficients take into account of the combined effect of the wind exerted on and under the roof for all wind directions; the resultant is to be taken normal to the canopy. Where the local coefficients overlap, the greater of the two given values should be taken. However, the effect of partial closures of one side and or both sides, such as those due to trains, buses and stored materials shall be foreseen and taken into account.

The solidity ratio, ϕ, is equal to the area of obstructions under the canopy divided by the gross area under the canopy, both areas normal to the wind direction. $\phi = 0$ represents a canopy with no obstructions underneath. $\phi = 1$ represents the canopy fully blocked with contents to the downwind eaves. Values of C_p for intermediate solidities may be linearly interpolated between these two extremes and apply upwind of the position of maximum blockage only. For downwind of the position of maximum blockage, the coefficients for $\phi = 0$ may be used.

In addition to the forces due to the pressures normal to the canopy, there will be horizontal loads on the canopy due to the wind pressure on any fascia and to friction over the surface of the canopy. For any wind direction, only the greater of these two forces need to be taken into account. Fascia loads should be calculated on the area of the surface facing the wind, using a force coefficient of 1.3. Frictional drag should be calculated using the coefficients given in **4.5.4.1**.

NOTE — Table 13 to Table 18 may be used to get internal and external pressure coefficients for pitches and troughed free roofs for some specific cases for which aspect ratios and roof slopes have been specified. However, while using Table 13 to Table 18 any significant departure from it should be investigated carefully. No increase shall be made for local effects except as indicated.

4.5.3.3.4 *Pitched and saw-tooth roofs of multi-span buildings*

For pitched and saw-tooth roofs of multi-span buildings, the external average pressure coefficients shall be as given in Table 19 and Table 20, respectively provided that all the spans shall be equal and the height to the eaves shall not exceed the span.

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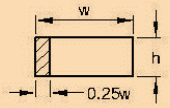
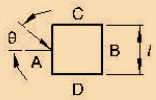
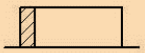
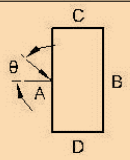
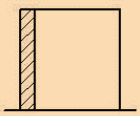
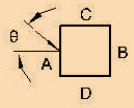
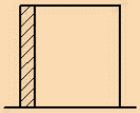
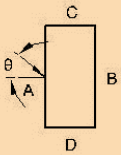
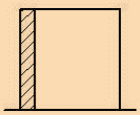
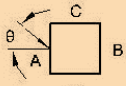
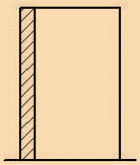
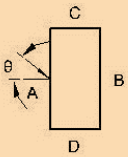
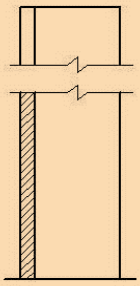
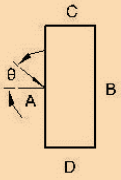
Table 8 External Pressure Coefficients (C_{pe}) for Walls of Rectangular Clad Buildings

(*Clause* 4.5.3.3.1)

BUILDING HEIGHT RATIO	BUILDING PLAN RATIO	ELEVATION	PLAN	WIND ANGLE Θ	C_{pe} FOR SURFACE A	B	C	D	LOCAL C_{pe}
$\frac{h}{w} \leq \frac{1}{2}$	$1 < \frac{l}{w} \leq \frac{3}{2}$			Degrees 0 90	+ 0.7 - 0.5	- 0.2 - 0.5	- 0.5 + 0.7	- 0.5 - 0.2	- 0.8
	$\frac{3}{2} < \frac{l}{w} < 4$			0 90	+ 0.7 - 0.5	- 0.25 - 0.5	- 0.6 + 0.7	- 0.6 - 0.1	- 1.0
$\frac{1}{2} < \frac{h}{w} \leq \frac{3}{2}$	$1 \leq \frac{l}{w} \leq \frac{3}{2}$			0 90	+ 0.7 - 0.6	- 0.25 - 0.6	- 0.6 + 0.7	- 0.6 - 0.25	- 1.1
	$\frac{3}{2} \leq \frac{l}{w} < 4$			0 90	+ 0.7 - 0.5	- 0.3 - 0.5	- 0.7 + 0.7	- 0.7 - 0.1	- 1.1
$\frac{3}{2} < \frac{h}{w} < 6$	$1 < \frac{l}{w} \leq \frac{3}{2}$			0 90	+ 0.8 - 0.8	- 0.25 - 0.8	- 0.8 + 0.8	- 0.8 - 0.25	- 1.2
	$\frac{3}{2} \leq \frac{l}{w} < 4$			0 90	+ 0.7 - 0.5	- 0.4 - 0.5	- 0.7 + 0.8	- 0.7 - 0.1	- 1.2
$\frac{h}{w} \geq 6$	$\frac{l}{w} = \frac{3}{2}$			0 90	+ 0.95 - 0.8	- 1.85 - 0.8	- 0.9 + 0.9	- 0.9 - 0.85	1.25
	$\frac{l}{w} = 1.0$			0 90	+ 0.95 - 0.7	- 1.25 - 0.7	- 0.7 + 0.95	- 0.7 - 1.25	1.25
	$\frac{l}{w} = 2$			0 90	+ 0.85 - 0.75	- 0.75 - 0.75	- 0.75 + 0.85	- 0.75 - 0.75	1.25

NOTE : h is the height to eaves or parapet, l is the greater horizontal dimension of a building and w is the lesser horizontal dimension of a building.

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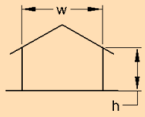
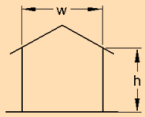
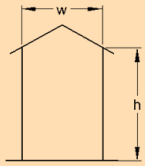
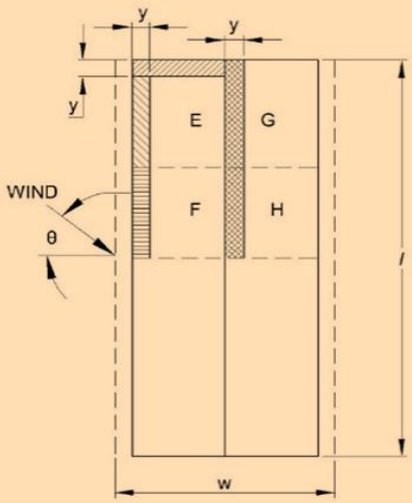
Table 9 External Pressure Coefficients (C_{pe}) for Pitched Roofs of Rectangular Clad Buildings

(*Clause* 4.5.3.3.2)

BUILDING HEIGHT RATIO	ROOF ANGLE α	WIND ANGLE θ 0°		WIND ANGLE θ 90°		LOCAL COEFFICIENTS			
		EF	*GH*	*EG*	*FH*				
	Degrees								
$\frac{h}{w} \leq \frac{1}{2}$ 	0	- 0.8	- 0.4	- 0.8	- 0.4	- 2.0	- 2.0	- 2.0	-----
	5	- 0.9	- 0.4	- 0.8	- 0.4	- 1.4	- 1.2	- 1.2	- 1.0
	10	- 1.2	- 0.4	- 0.8	- 0.6	- 1.4	- 1.4		- 1.2
	20	- 0.4	- 0.4	- 0.7	- 0.6	- 1.0			- 1.2
	30	0	- 0.4	- 0.7	- 0.6	- 0.8			- 1.1
	45	+ 0.3	- 0.5	- 0.7	- 0.6				- 1.1
	60	+ 0.7	- 0.6	- 0.7	- 0.6				- 1.1
$\frac{1}{2} \leq \frac{h}{w} \leq \frac{3}{2}$ 	0	- 0.8	- 0.6	- 1.0	- 0.6	- 2.0	- 2.0	- 2.0	-----
	5	- 0.9	- 0.6	- 0.9	- 0.6	- 2.0	- 2.0	- 1.5	- 1.0
	10	- 1.1	- 0.6	- 0.8	- 0.6	- 2.0	- 2.0	- 1.5	- 1.2
	20	- 0.7	- 0.5	- 0.8	- 0.6	- 1.5	- 1.5		- 1.0
	30	- 0.2	- 0.5	- 0.8	- 0.8	- 1.0			- 1.0
	45	+ 0.2	- 0.5	- 0.8	- 0.8				
	60	+ 0.6	- 0.5	- 0.8	- 0.8				
$\frac{3}{2} < \frac{h}{w} < 6$ 	0	- 0.7	- 0.6	- 0.9	- 0.7	- 2.0	- 2.0	- 2.0	-----
	5	- 0.7	- 0.6	- 0.8	- 0.8	- 2.0	- 2.0	- 1.5	- 1.0
	10	- 0.7	- 0.6	- 0.8	- 0.8	- 2.0	- 2.0	- 1.5	- 1.2
	20	- 0.8	- 0.6	- 0.8	- 0.8	- 1.5	- 1.5		- 1.2
	30	- 1.0	- 0.5	- 0.8	- 0.7	- 1.5			
	40	- 0.2	- 0.5	- 0.8	- 0.7	- 1.0			
	45	+ 0.2	- 0.5	- 0.8	- 0.7				
	60	+ 0.5	- 0.5	- 0.8	- 0.7				



KEY PLAN

$y = h$ or $0.15\ w$

Whichever is the lesser.

NOTE - 1 *h* is the height to eaves or parapet and *w* is the lesser horizontal dimension of a building.

NOTE - 2 Where no local coefficients are given , the overall coefficient apply.

NOTE - 3 For hipped roofs the local coefficient for the hip ridge may be conservatively taken as the the appropriate ridge value.

NOTE - 4 *w* and *l* are dimensions between the walls excluding overhangs.

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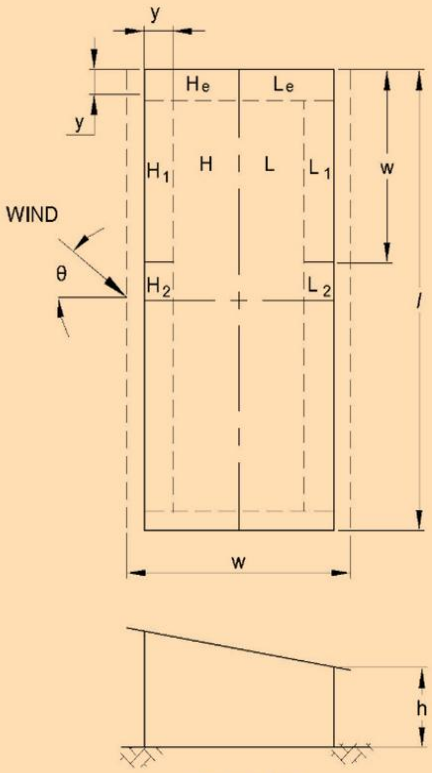
Table 10 External Pressure Coefficients (C_{pe}) for Monoslope Roofs for Rectangular Clad Buildings

$$\frac{h}{w} < 2$$

(*Clause* 4.5.3.3.2)



y = *h* or 0.15 *w*
Whichever is the lesser.

NOTE :- Area *H* and area *L* refer to the whole quadrant.

ROOF ANGLE α	WIND ANGLE θ									LOCAL COEFFICIENTS (C_{pe})					
	0°		45°		90°		135°		180°						
	H	*L*	*H*	*L*	*H* & *L* *	*H* & *L* **	*H*	*L*	*H*	*L*	H_1	H_2	L_1	L_2	H_e	L_e
Degrees																
5	- 1.0	- 0.5	- 1.0	- 0.9	- 1.0	- 0.5	- 0.9	- 1.0	- 0.5	- 1.0	- 2.0	- 1.5	- 2.0	- 1.5	- 2.0	- 2.0
10	- 1.0	- 0.5	- 1.0	- 0.8	- 1.0	- 0.5	- 0.8	- 1.0	- 0.4	- 1.0	- 2.0	- 1.5	- 2.0	- 1.5	- 2.0	- 2.0
15	- 0.9	- 0.5	- 1.0	- 0.7	- 1.0	- 0.5	- 0.6	- 1.0	- 0.3	- 1.0	- 1.8	- 0.9	- 1.8	- 1.4	- 2.0	- 2.0
20	- 0.8	- 0.5	- 1.0	- 0.6	- 0.9	- 0.5	- 0.5	- 1.0	- 0.2	- 1.0	- 1.8	- 0.8	- 1.8	- 1.4	- 2.0	- 2.0
25	- 0.7	- 0.5	- 1.0	- 0.6	- 0.8	- 0.5	- 0.3	- 0.9	- 0.1	- 0.9	- 1.8	- 0.7	- 0.9	- 0.9	- 2.0	- 2.0
30	- 0.5	- 0.5	- 1.0	- 0.6	- 0.8	- 0.5	- 0.1	- 0.6	0	- 0.6	- 1.8	- 0.5	- 0.5	- 0.5	- 2.0	- 2.0

* Applied to length *w* / 2 from wind-ward end.

** Applies to remainder

NOTE :- 1 *h* is the height of eaves at lower side, *l* is the greater horizontal dimensions of a building and *w* is the lesser horizontal dimension of a building.

NOTE :- 2 l and w are overall length and width including overhangs,

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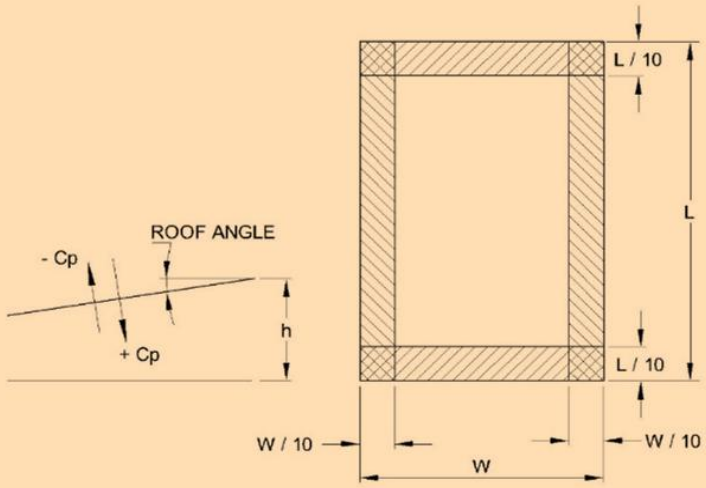
Table 11 Pressure Coefficients for Monoslope Free Roofs

(*Clause* 4.5.3.3.3)



ROOF ANGLE (Degrees) α	SOLIDITY RATIO Φ	MAXIMUM (LARGEST + ve) AND MINIMUM (LARGEST - ve) PRESSURE COEFFICIENTS			
		OVERALL COEFFICIENTS	LOCAL COEFFICIENTS		
			□	▨	▨
0	All values of Φ	+ 0.2	+ 0.5	+ 1.8	+ 1.1
5		+ 0.4	+ 0.8	+ 2.1	+ 1.3
10		+ 0.5	+ 1.2	+ 2.4	+ 1.6
15		+ 0.7	+ 1.4	+ 2.7	+ 1.8
20		+ 0.8	+ 1.7	+ 2.9	+ 2.1
25		+ 1.0	+ 2.0	+ 3.1	+ 2.3
30		+ 1.2	+ 2.2	+ 3.2	+ 2.4
0	Φ = 0	- 0.5	- 0.6	- 1.3	- 1.4
	Φ = 1	- 1.0	- 1.2	- 1.8	- 1.9
5	Φ = 0	- 0.7	- 1.1	- 1.7	- 1.8
	Φ = 1	- 1.1	- 1.6	- 2.2	- 2.3
10	Φ = 0	- 0.9	- 1.5	- 2.0	- 2.1
	Φ = 1	- 1.3	- 2.1	- 2.6	- 2.7
15	Φ = 0	- 1.1	- 1.8	- 2.4	- 2.5
	Φ = 1	- 1.4	- 2.3	- 2.9	- 3.0
20	Φ = 0	- 1.3	- 2.2	- 2.8	- 2.9
	Φ = 1	- 1.5	- 2.6	- 3.1	- 3.2
25	Φ = 0	- 1.6	- 2.6	- 3.2	- 3.2
	Φ = 1	- 1.7	- 2.8	- 3.5	- 3.5
30	Φ = 0	- 1.8	- 3.0	- 3.8	- 3.6
	Φ = 1	- 1.8	- 3.0	- 3.8	- 3.6

NOTE 1: For monopitch canopies the centre of pressure should be taken to act at 0.3 w from the windward edge.

NOTE 2: W and L are overall width and length including overhangs,

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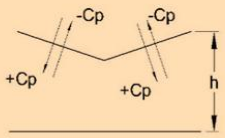
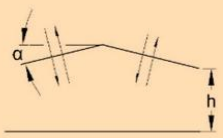
Table 12 Pressure Coefficients for Free Standing Double Sloped Roofs

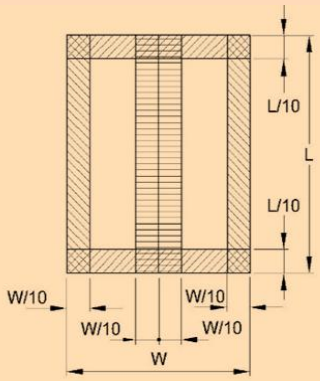
(*Clause* 4.5.3.3.3)



- ve ROOF ANGLE



+ ve ROOF ANGLE



KEY PLAN

ROOF ANGLE (Degrees) α	SOLIDITY RATIO Φ	MAXIMUM (LARGEST + ve) AND MINIMUM (LARGEST - ve) PRESSURE COEFFICIENTS				
		OVERALL COEFFICIENTS	LOCAL COEFFICIENTS			
			▭	▨	▨	▬
- 20	All values of Φ	+ 0.7	+ 0.8	+ 1.6	+ 0.6	+ 1.7
- 15		+ 0.5	+ 0.6	+ 1.5	+ 0.7	+ 1.4
- 10		+ 0.4	+ 0.6	+ 1.4	+ 0.8	+ 1.1
- 5		+ 0.3	+ 0.5	+ 1.5	+ 0.8	+ 0.8
+ 5		+ 0.3	+ 0.6	+ 1.8	+ 1.3	+ 0.4
+ 10		+ 0.4	+ 0.7	+ 1.8	+ 1.4	+ 0.4
+ 15		+ 0.4	+ 0.9	+ 1.9	+ 1.4	+ 0.4
+ 20		+ 0.6	+ 1.1	+ 1.9	+ 1.5	+ 0.4
+ 25		+ 0.7	+ 1.2	+ 1.9	+ 1.6	+ 0.5
+ 30		+ 0.9	+ 1.3	+ 1.9	+ 1.6	+ 0.7
- 20	Φ = 0	- 0.7	- 0.9	- 1.3	- 1.6	- 0.6
	Φ = 1	- 0.9	- 1.2	- 1.7	- 1.9	- 1.2
- 15	Φ = 0	- 0.6	- 0.8	- 1.3	- 1.6	- 0.6
	Φ = 1	- 0.8	- 1.1	- 1.7	- 1.9	- 1.2
- 10	Φ = 0	- 0.6	- 0.8	- 1.3	- 1.5	- 0.6
	Φ = 1	- 0.8	- 1.1	- 1.7	- 1.9	- 1.3
- 5	Φ = 0	- 0.5	- 0.7	- 1.3	- 1.6	- 0.6
	Φ = 1	- 0.8	- 1.5	- 1.7	- 1.9	- 1.4
+ 5	Φ = 0	- 0.6	- 0.6	- 1.4	- 1.4	- 1.1
	Φ = 1	- 0.9	- 1.3	- 1.8	- 1.8	- 2.1
+ 10	Φ = 0	- 0.7	- 0.7	- 1.5	- 1.4	- 1.4
	Φ = 1	- 1.1	- 1.4	- 2.0	- 1.8	- 2.4
+ 15	Φ = 0	- 0.8	- 0.9	- 1.7	- 1.4	- 1.8
	Φ = 1	- 1.2	- 1.5	- 2.2	- 1.9	- 2.8
+ 20	Φ = 0	- 0.9	- 1.2	- 1.8	- 1.4	- 2.0
	Φ = 1	- 1.3	- 1.7	- 2.3	- 1.9	- 3.0
+ 25	Φ = 0	- 1.0	- 1.4	- 1.9	- 1.4	- 2.0
	Φ = 1	- 1.4	- 1.9	- 2.4	- 2.1	- 3.0
+ 30	Φ = 0	- 1.0	- 1.4	- 1.9	- 1.4	- 2.0
	Φ = 1	- 1.4	- 2.1	- 2.6	- 2.2	- 3.0

Each slope of a duopitch canopy should be able to withstand forces using both the maximum and the minimum coefficients, and the whole canopy should be able to support forces using one slope at the maximum coefficient with the other slope at the minimum coefficient. For duoptich canopies the centre of pressure should be taken to act at the centre of each slope.

Note :- W and L are overall width and length including overhangs

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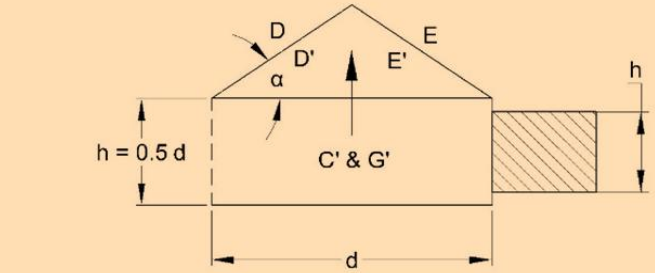
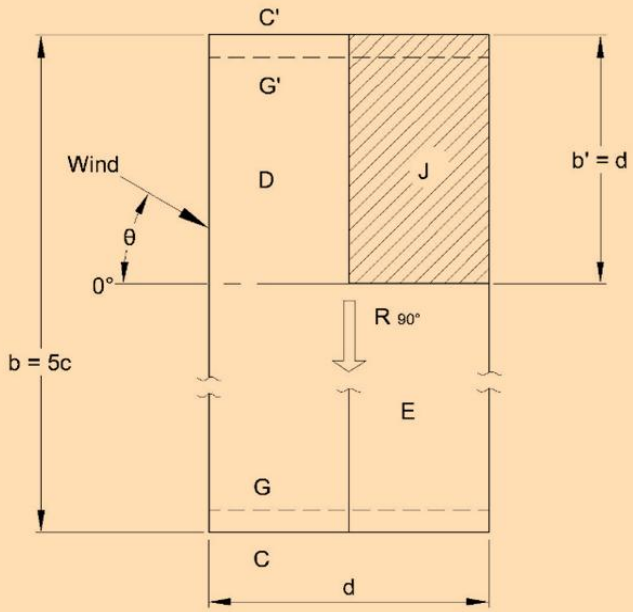
Table 13 Pressure Coefficients (Top and Bottom) for Pitched Roofs, Roof Slope,
(*Clause* 4.5.3.3.3)





Roof slope α = 30°
θ = 0°- 45°, *D, D', E, E'* full length.
θ = 90°, *D, D', E, E'* part length.
b', thereafter $C_p = 0$

θ	PRESSURE COEFFICIENTS, C_p							
	D	*D'*	*E*	*E'*	END SURFACES			
					C	*C'*	*G*	*G'*
0°	+ 0.6	- 1.0	- 0.5	- 0.9				
45°	+ 0.1	- 0.3	- 0.6	- 0.3				
90°	- 0.3	- 0.4	- 0.3	- 0.4	- 0.3	- 0.8	- 0.3	- 0.4
For all value of θ	For J : C_p Top = 1.0, C_p bottom = - 0.2 Tangentially acting friction : $R_{90°} = 0.05\, p_d\, bd$							

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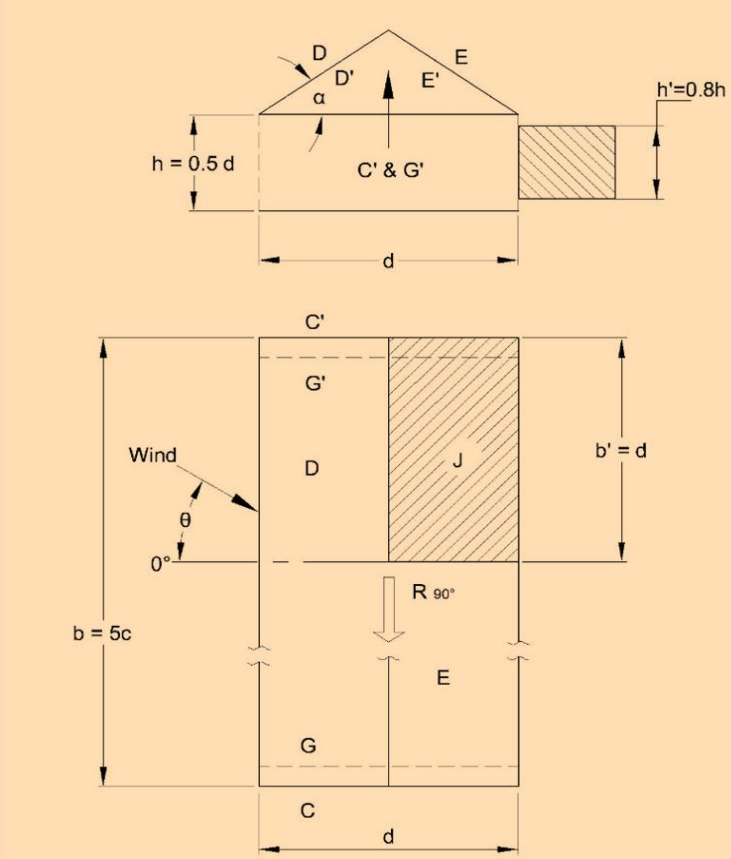
Table 14 Pressure Coefficients (Top and Bottom) for Pitched Free Roofs, $\alpha = 30^\circ$ With Effects of Train or Stored Materials

(*Clause* 4.5.3.3.3)



Roof slope $\alpha = 30°$
Effects of trains or stored materials.
$\theta = 0° - 45°$, or $135° - 180°$, *D, D', E, E'* full length.
$\theta = 90°$, *D, D', E, E'* part length.
b', thereafter $C_p = 0$

θ	PRESSURE COEFFICIENTS, C_p							
	D	*D'*	*E*	*E'*	END SURFACES			
					C	*C'*	*G*	*G'*
0°	+ 0.1	+ 0.8	- 0.7	+ 0.9				
45°	- 0.1	+ 0.5	- 0.8	+ 0.5				
90°	- 0.4	- 0.5	- 0.4	- 0.5	- 0.3	+ 0.8	+ 0.3	- 0.4
180°	- 0.3	- 0.6	+ 0.4	- 0.6				
For all value of θ	For J : C_p Top = -1.5, C_p bottom = 0.5 Tangentially acting friction : $R_{90°} = 0.05\, p_d\, bd$							

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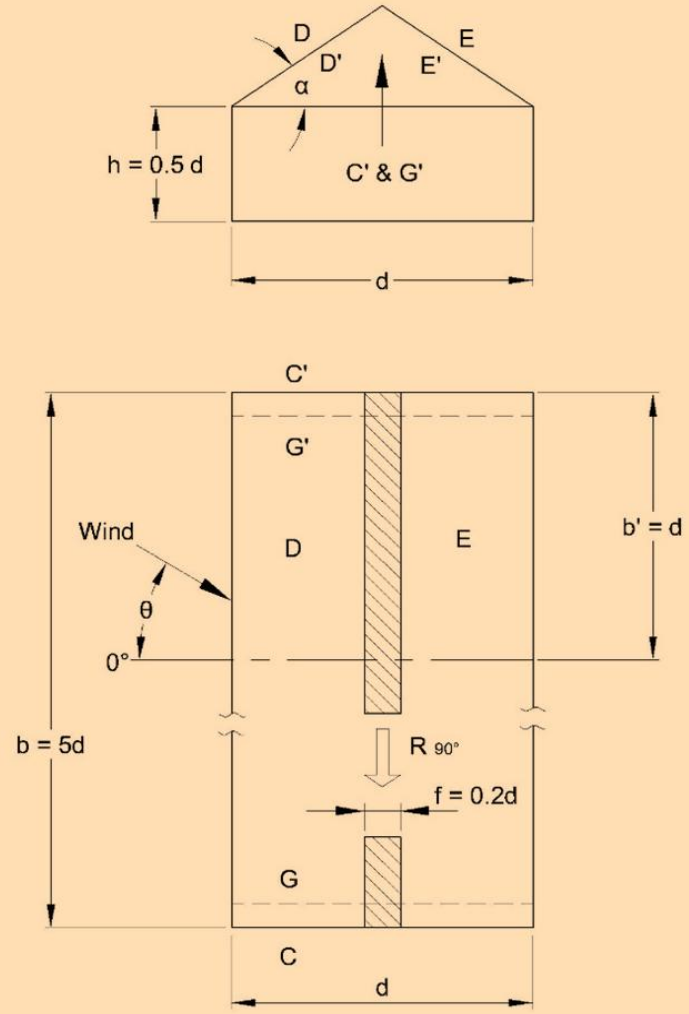
Table 15 Pressure Coefficients (Top and Bottom) for Pitched Free Roofs, $\alpha = 10^{\circ}$

(*Clause* 4.5.3.3.3)



Roof slope α = 10°

θ = 0°- 45°, *D*, *D'*, *E*, *E'* full length.

θ = 90°, *D*, *D'*, *E*, *E'* part length.

b', thereafter $C_p = 0$

θ	PRESSURE COEFFICIENTS, C_p							
	D	*D'*	*E*	*E'*	END SURFACES			
					C	*C'*	*G*	*G'*
0°	- 1.0	+ 0.3	- 0.5	+ 0.2				
45°	- 0.3	+ 0.1	- 0.3	+ 0.1				
90°	- 0.3	0	- 0.3	0	- 0.4	+ 0.8	+ 0.3	- 0.6
For all value of θ	For *f* : C_p Top = - 1.0, C_p bottom = 0.4 Tangentially acting friction : $R_{90^{\circ}} = 0.1\, p_d\, bd$							

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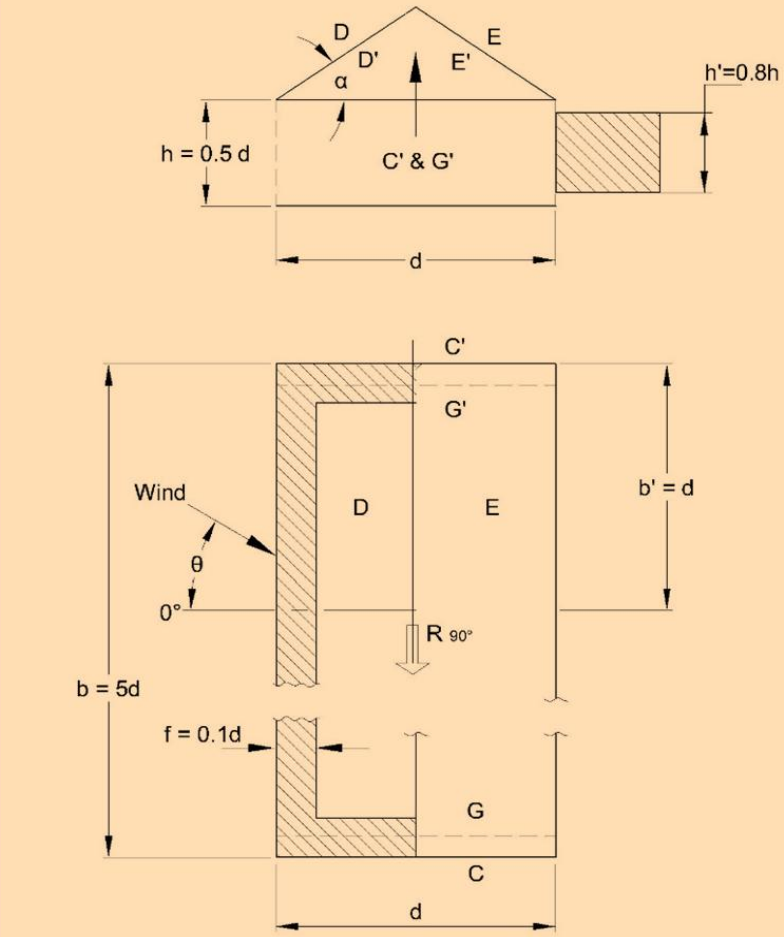
Table 16 Pressure Coefficients (Top and Bottom) for Pitched Free Roofs, $\alpha = 10°$ With Effects of Train or Stored Materials

(*Clause* 4.5.3.3.3)



Roof slope α = 10°
Effects of trains or stored materials.
θ = 0°- 45°, or 135° - 180°, *D, D', E, E'* full length.
θ = 90°, *D, D', E, E'* part length.
b', thereafter $C_p = 0$

θ	PRESSURE COEFFICIENTS, C_p							
	D	*D'*	*E*	*E'*	END SURFACES			
					C	*C'*	*G*	*G'*
0°	- 1.3	+ 0.8	- 0.6	0.7				
45°	- 0.5	+ 0.4	- 0.3	+ 0.3				
90°	- 0.3	0	- 0.3	0	- 0.4	+ 0.8	+ 0.3	- 0.6
180°	- 0.4	- 0.3	- 0.6	- 0.3				
For all value of θ	For *f* : C_p Top = -1.6, C_p bottom = - 0.9 Tangentially acting friction : $R_{90°} = 0.1\, p_d\, bd$							

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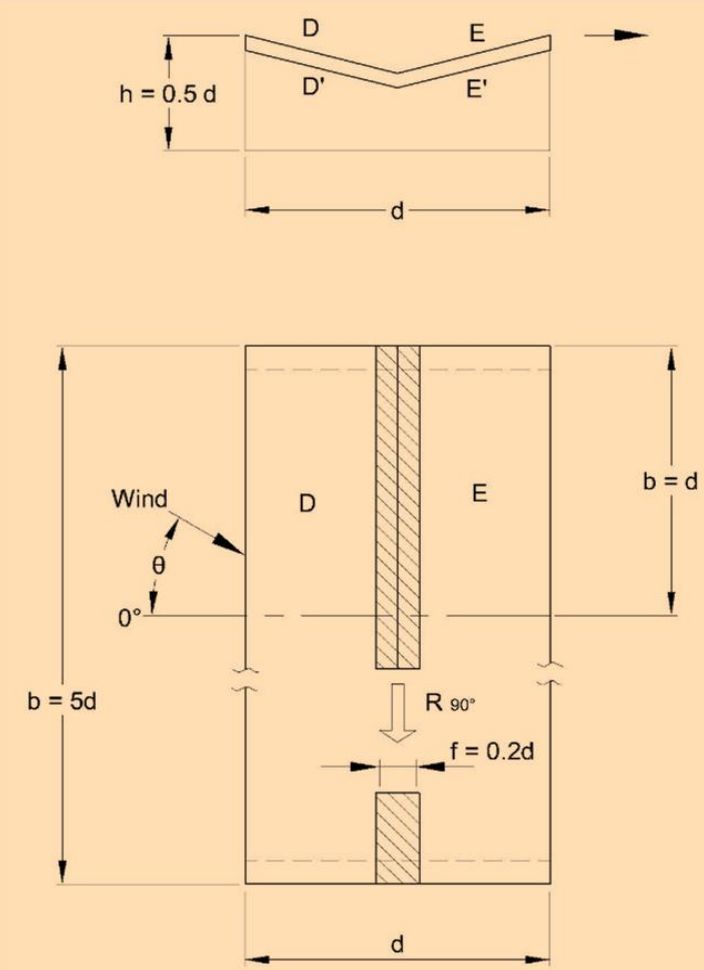
Table 17 Pressure Coefficients for Troughed Free Roofs, $\alpha = 10^{\circ}$

(*Clause* 4.5.3.3.3)



Roof slope $\alpha = 10^{\circ}$

$\theta = 0^{\circ}$- 45°, *D, D', E, E'* full length.

$\theta = 90^{\circ}$, *D, D', E, E'* part length.

b', thereafter $C_p = 0$

θ	PRESSURE COEFFICIENTS, C_p			
	D	*D'*	*E*	*E'*
0°	+ 0.3	- 0.7	+ 0.2	- 0.9
45°	0	- 0.2	+ 0.1	- 0.3
90°	- 0.1	0.1	- 0.1	+ 0.1
For all value of θ	For *f* : C_p Top = 0.4, C_p bottom = - 1.5 Tangentially acting friction : $R_{90^{\circ}} = 0.1\, p_d\, bd$			

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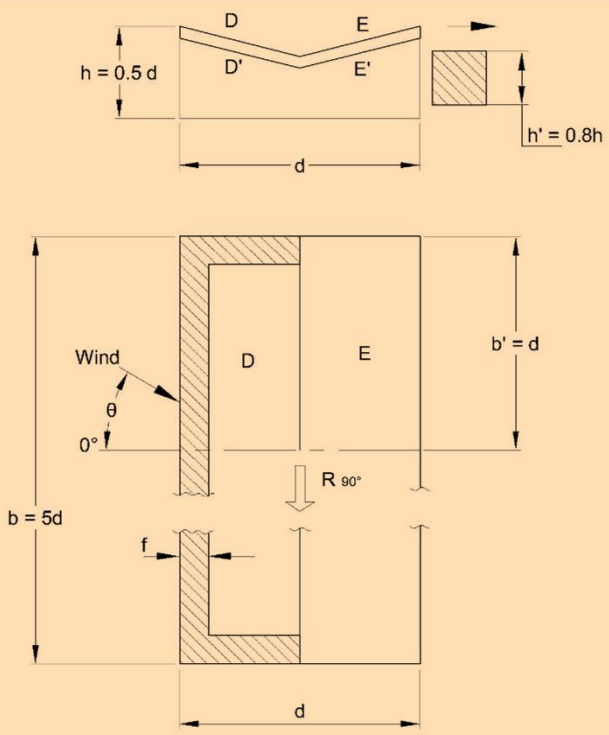
Table 18 Pressure Coefficients (Top and Bottom) for Troughed Free Roofs, $\alpha = 10^{\circ}$ With Effects of Train or Stored Material

(*Clause* 4.5.3.3.3)



Roof slope $\alpha = 10°$
Effects of trains or stored materials.
$\theta = 0°$- 45°, or 135° - 180°, *D, D', E, E'* full length.
$\theta = 90°$, *D, D', E, E'* part length.
b', thereafter $C_p = 0$

θ	PRESSURE COEFFICIENTS, C_p			
	D	*D'*	*E*	*E'*
0°	- 0.7	+ 0.8	- 0.6	+ 0.6
45°	- 0.4	+ 0.3	- 0.2	+ 0.2
90°	- 0.1	+ 0.1	- 0.1	+ 0.1
180°	- 0.4	- 0.2	- 0.6	- 0.3
For all value of θ	For *f* : C_p Top = -1.1, C_p bottom = 0.9 Tangentially acting friction : $R_{90°} = 0.1 p_d bd$			

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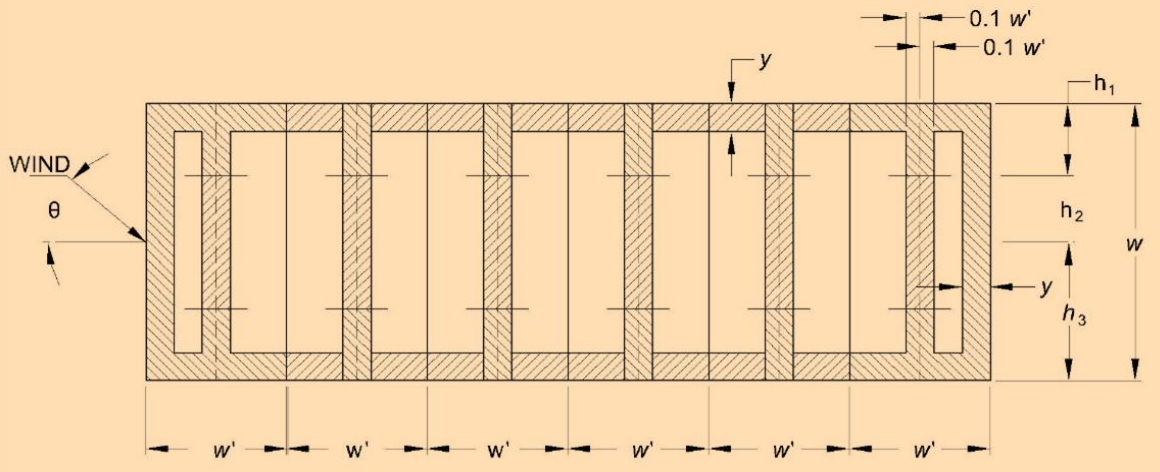
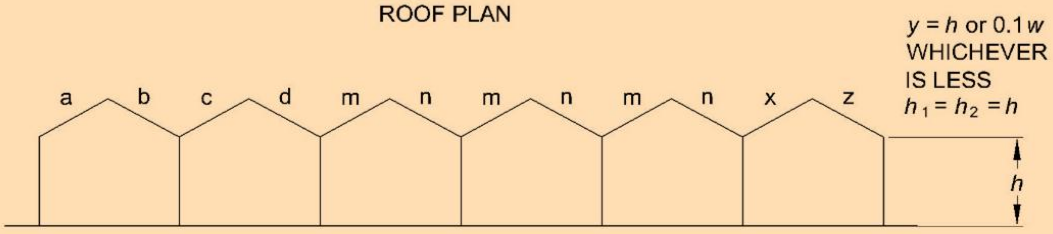
Table 19 External Pressure Coefficients (C_{pe}) for Pitched Roofs of Multi-span Buildings (All Spans Equal) with $h < w'$

(*Clause* 4.5.3.3.4)



ROOF PLAN



SECTION

ROOF ANGLE	WIND ANGLE	FIRST SPAN		FIRST INTERME - DIATE SPAN		OTHER INTERME - DIATE SPANS		END SPAN		LOCAL COEFFICIENT	
α Degrees	θ Degrees	a	b	c	d	m	n	x	z		
5	0	- 0.9	- 0.6	- 0.4	- 0.3	- 0.3	- 0.3	- 0.3	- 0.3		
10		- 1.1	- 0.6	- 0.4	- 0.3	- 0.3	- 0.3	- 0.3	- 0.4		
20		- 0.7	- 0.6	- 0.4	- 0.3	- 0.3	- 0.3	- 0.3	- 0.5	- 2.0	- 1.5
30		- 0.2	- 0.6	- 0.4	- 0.3	- 0.2	- 0.3	- 0.2	- 0.5		
45		+ 0.3	- 0.6	- 0.6	- 0.4	- 0.2	- 0.4	- 0.2	- 0.5		
DISTANCE											
ROOF ANGLE α DEGREES		WIND ANGLE θ DEGREES		h_1		h_2		h_3			
UP TO 45		90		- 0.8		- 0.6		- 0.2			

Frictional drag : When wind angle θ = 0°, horizontal forces due to frictional drag are allowed for in the above values, and

When wind angle θ = 90°, allow for frictional drag in accordance with **4.5.4.1**

NOTE — Evidence on these buildings is fragmentary and any departure from the cases given should be investigated separately.

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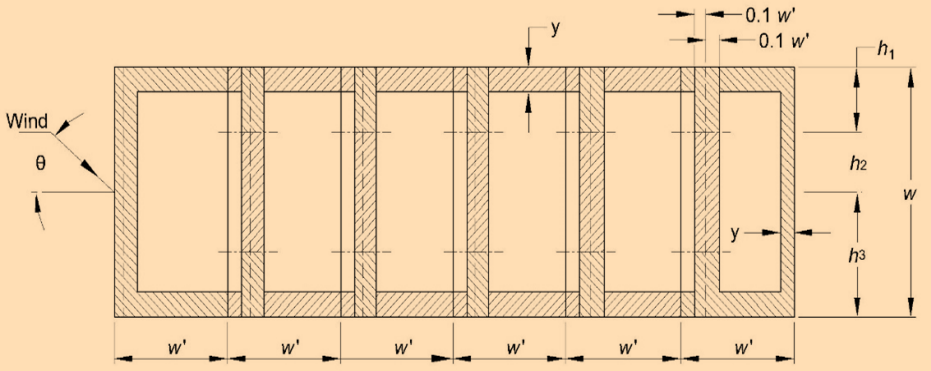
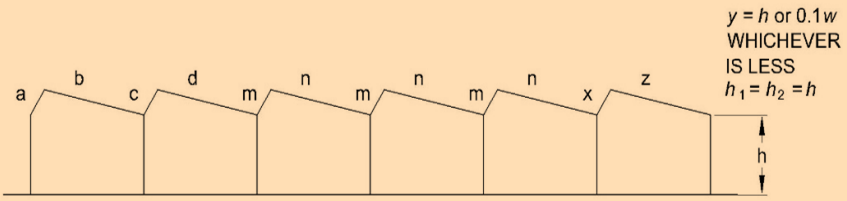
Table 20 External Pressure Coefficients (Cpe) for Saw Tooth Roofs of Multi-Span Buildings (All Spans Equal) with h < w'

(*Clause* 4.5.3.3.4)



ROOF PLAN



SECTION

WIND ANGLE	FIRST SPAN		FIRST INTERME - DIATE SPAN		OTHER INTERME - DIATE SPANS		END SPAN		LOCAL COEFFICIENT	
θ Degrees	a	b	c	d	m	n	x	z		
0	+ 0.6	- 0.7	- 0.7	- 0.4	- 0.3	- 0.2	- 0.1	- 0.3		
180	- 0.5	- 0.3	- 0.3	- 0.3	- 0.4	- 0.6	- 0.6	- 0.1	- 2.0	- 1.5

DISTANCE			
WIND ANGLE θ DEGREES	h_1	h_2	h_3
90	- 0.8	- 0.6	- 0.2
270	Similar to 90°, h_1, h_2, h_3, are needed to be reckoned from the windword edge in the same order		

Frictional drag : When wing angle θ = 0°, horizontal forces due to frictional drag are allowed for in the above values, and

When wind angle θ = 90°, allow for frictional drag in accordance with **4.5.4.1**

NOTE — Evidence on these buildings is fragmentary and any departure from the cases given should be investigated separately.

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4.5.3.3.5 *Pressure coefficients on overhangs from roofs*

The pressure coefficients on the top overhanging portion of the roofs shall be taken to be the same as that of the nearest top portion of the non-overhanging portion of the roofs. The pressure coefficients for the underside surface of the overhanging portions shall be taken as follows and shall be taken as positive, if the overhanging portion is on the windward side:

a) 1.25, if the overhanging slopes downwards;

b) 1.00, if the overhanging is horizontal; and

c) 0.75, if the overhanging slopes upwards.

For overhanging portions on sides other than windward side, the average pressure coefficients on adjoining walls may be used.

4.5.3.3.6 *Curved roofs*

For curved roofs the external pressure coefficients shall be as given in Table 21. Allowance for local effects shall be made in accordance with Table 9. Two values of C_2 have been given for elevated curved roofs. Both the load cases have to be analysed and critical load effects are to be considered in design.

4.5.3.3.7 *Cylindrical structures*

For the purpose of calculating the wind pressure distribution around a cylindrical structure of circular cross section, the value of external pressure coefficients given in Table 22 may be used, provided that the Reynolds number is more than 10 000. They may be used for wind blowing normal to the axis of cylinders having axis normal to the ground plane and cylinders having their axis parallel to the ground plane. '*h*' is height of a vertical cylinder or length of a horizontal cylinder. Where there is a free flow of air around both ends, *h* is to be taken as half the length when calculating *h*/*D* ratio.

In the calculation of resultant load on the periphery of the cylinder, the value of C_{pi} shall be taken into account. For open ended cylinders, C_{pi} shall be taken as follows:

a) - 0.8, where *h*/*D* is more than or equal to 0.3; and

b) - 0.5, where *h*/*D* is less than 0.3.

4.5.3.3.8 *Roofs and bottoms of cylindrical elevated structures*

The external pressure coefficients for roofs and bottoms of cylindrical elevated structures shall be as given in Table 23. Alternately, the pressure distribution given in Fig. 3 can be used together with the force coefficients given in Table 28 for the cylindrical portion.

4.5.3.3.9 *Combined roofs*

The average external pressure coefficients for combined roofs are shown in Table 24.

4.5.3.3.10 *Roofs with skylight*

The average external pressure coefficients for roofs with skylight are shown in Table 25.

4.5.3.3.11 *Grandstands*

The pressure coefficients on the roof (top and bottom) and rear wall of a typical grandstand roof which is open on three sides are given in Table 26. The pressure coefficients are valid for a particular ratio of dimensions as specified in Table 24 but may be used for deviations up to 20 percent. In general, the maximum wind load occurs when the wind is blowing into the open front of the stand, causing positive pressure under the roof and negative pressure on the roof.

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4.5.3.3.12 *Spheres*

The external pressure coefficients for spheres shall be as given in Table 27.

4.5.3.3.13 *Frames*

When taking wind loads on frames of clad buildings it is reasonable to assume that the pressures or suctions inside and outside the structure shall not be fully correlated. Therefore, when taking the combined effect of wind loads on the frame, a reduction factor of $K_c = 0.90$ may be used over the building envelope when roof is subjected to pressure and internal pressure is suction, or *vice-versa*.

4.5.4 *Force Coefficients*

The value of force coefficients (C_f) apply to a building or structure as a whole and when multiplied by the effective frontal area (A_e) of the building or structure and design wind pressure, p_d gives the total wind load (F) on that particular building or structure, expressed as:

$$F = C_f A_e p_d$$

where

F = force acting in a direction specified in the respective tables; and

C_f = force coefficient for the building.

NOTES

1 The value of the force coefficient differs for the wind acting on different faces of a building or structure. In order to determine the critical load, the total wind load should be calculated for each wind direction.

2 If surface design pressure varies with height, the surface area of the building/structure may be sub-divided so that specified pressures are taken over appropriate areas.

3 In tapered buildings/structures, the force coefficients shall be applied after sub-dividing the building/structure into suitable number of strips and the load on each strip calculated individually, taking the area of each strip as A_e.

4 For force coefficients for structures not covered herein, reference may be made to specialist literature on the subject or advice may be sought from specialist in the subject.

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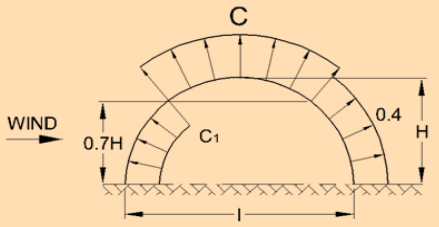
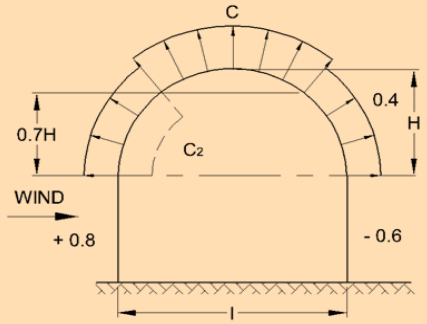
Table 21 External Pressure Coefficients (Cpe) for Curved Roofs

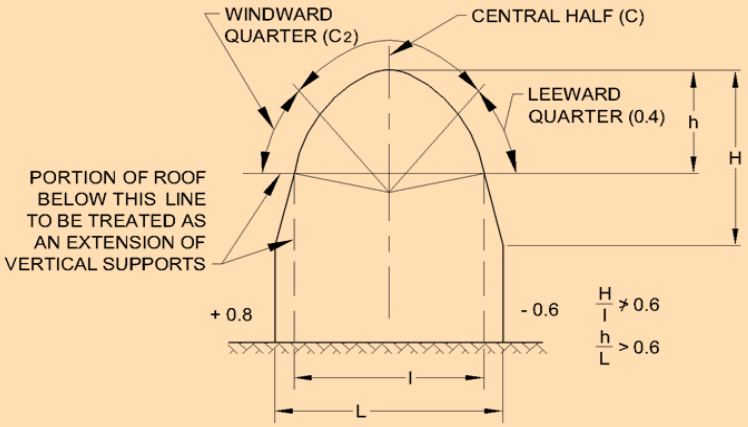
(*Clause* 4.5.3.3.6)



a) ROOF SPRINGING FROM GROUND LEVEL



b) ROOF ON ELEVATED STRUCTURE



c) DOUBLE CURVED ROOFS

VALUES OF C, C_1 and C_2

H/l	C	C_1	C_2	C_2
0.1	- 0.8	+ 0.1	- 0.8	+ 0.05
0.2	- 0.9	+ 0.3	- 0.7	+ 0.1
0.3	- 1.0	+ 0.4	- 0.3	+ 0.15
0.4	- 1.1	+ 0.6	+ 0.4	-
0.5	- 1.2	+ 0.7	+ 0.7	-

Note :- When the wind is blowing normal to gable ends, C_{pe} may be taken as equal to - 0.7 for the full width of the roof over a length of $l/2$ from the gable ends and - 0.5 for the remaining portion.

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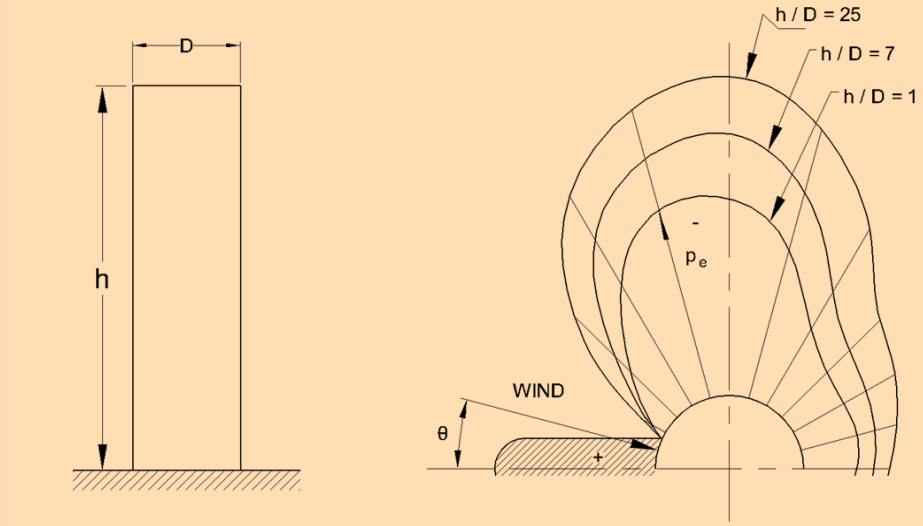
Table 22 External Pressure Coefficients (C_{pe}) Around Cylindrical Structures

(*Clause* 4.5.3.3.7)



POSITION OF PERIPHERY, θ IN DEGREES	PRESSURE COEFFICIENTS C_{pe}		
	h / *D* = 25	*h* / *D* = 7	*h* / *D* = 1
0	1.0	1.0	1.0
15	0.8	0.8	0.8
30	0.1	0.1	0.1
45	- 0.9	- 0.8	- 0.7
60	- 1.9	- 1.7	- 1.2
75	- 2.5	- 2.2	- 1.6
90	- 2.6	- 2.2	- 1.7
105	- 1.9	- 1.7	- 1.2
120	- 0.9	- 0.8	- 0.7
135	- 0.7	- 0.6	- 0.5
150	- 0.6	- 0.5	- 0.4
165	- 0.6	- 0.5	- 0.4
180	- 0.6	- 0.5	- 0.4

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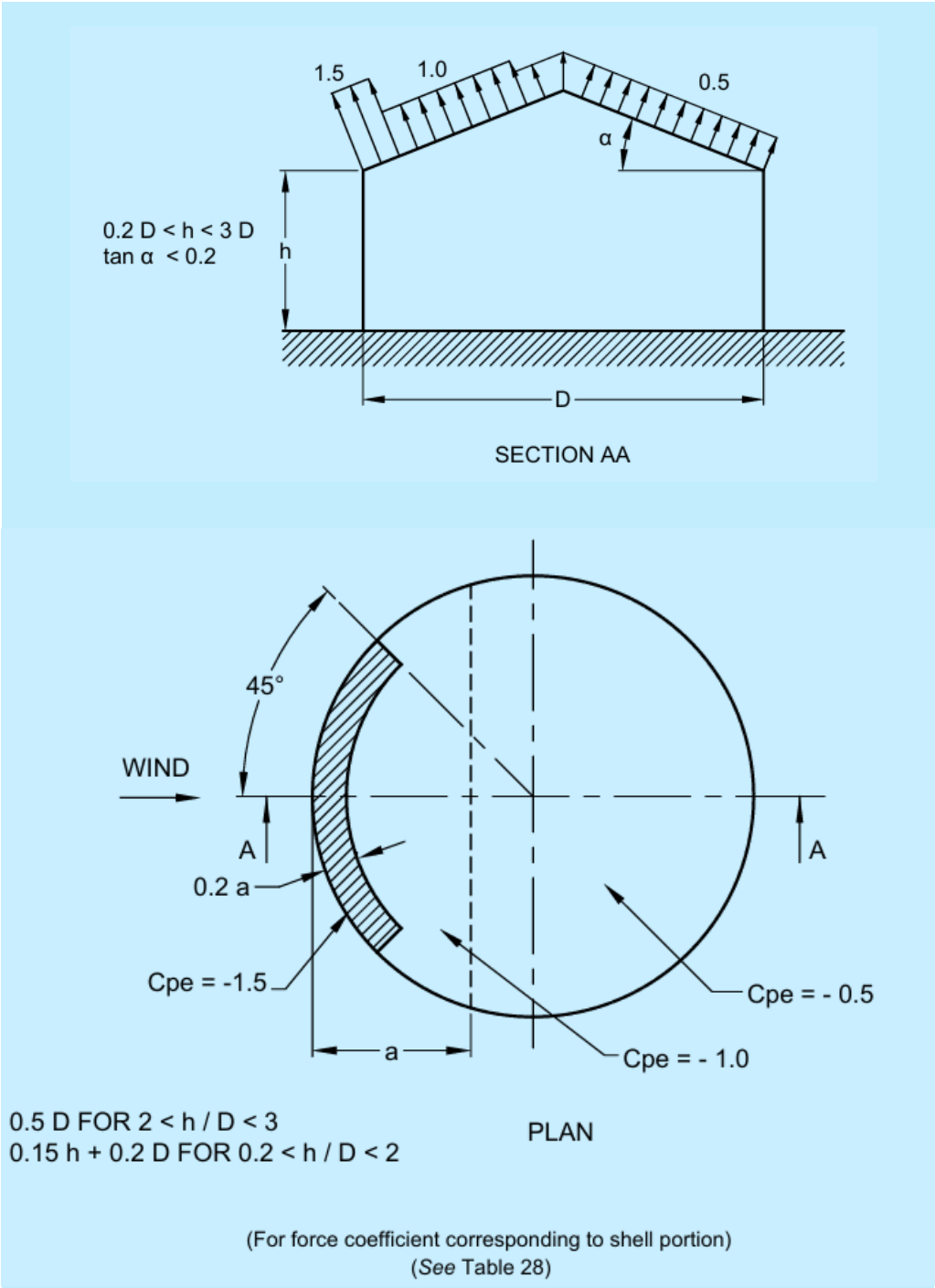

FIG. 3 EXTERNAL PRESSURE COEFFICIENTS ON THE UPPER ROOF SURFACE OF CYLINDRICAL STRUCTURES STANDING ON THE GROUND

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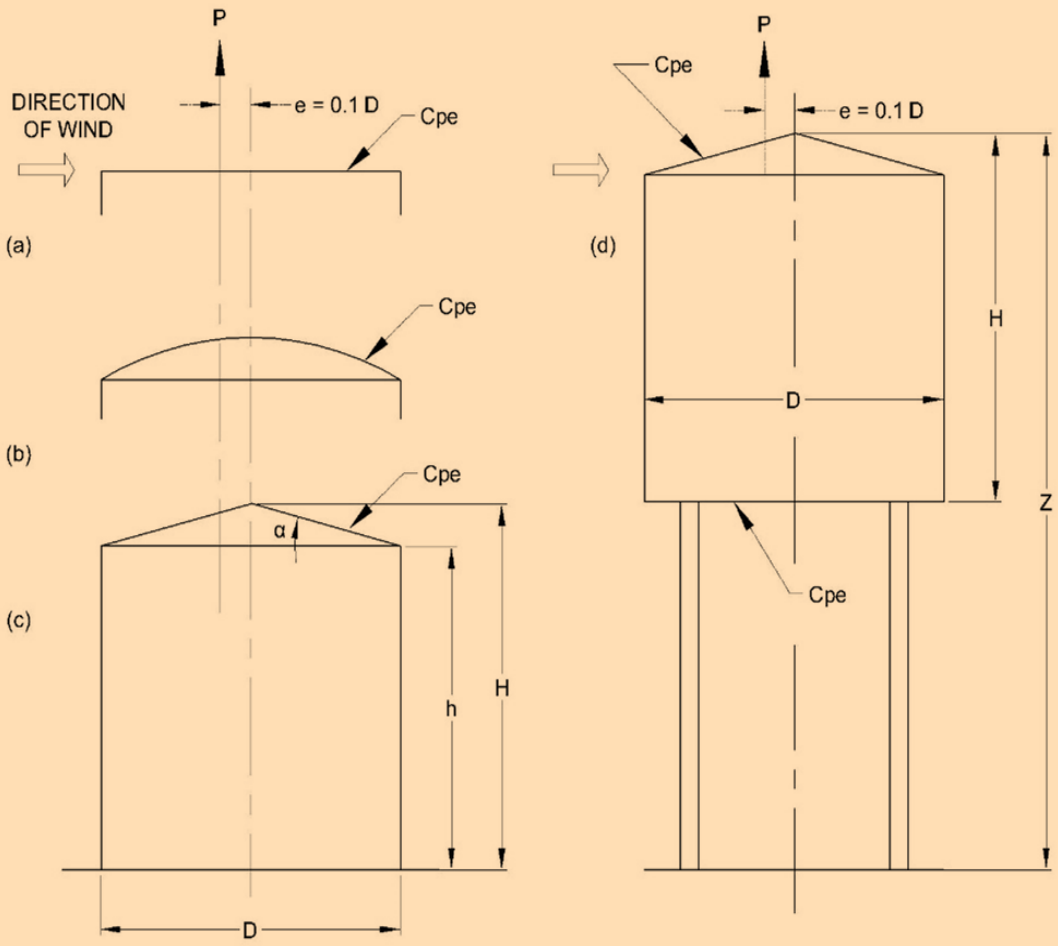
Table 23 External Pressure Coefficients (C_{pe}) for Roofs and Bottoms of Cylindrical Structures

(*Clause* 4.5.3.3.8)



COEFFICIENTS OF EXTERNAL PRESSURE C_{pe}				
STRUCTURE ACCORDING TO SHAPE				
a, b and c		d		
H / D	ROOF	(z / H) - 1	ROOF	BOTTOM
0.5	- 0.65	1.00	- 0.75	- 0.8
1.00	- 1.00	1.25	- 0.75	- 0.7
2.00	- 1.00	1.50	- 0.75	- 0.6

Total force acting on the roof of the structure, $P = 0.785\ D^2\ (C_{pi} - C_{pe})\ p_d$

The resultant of P lies eccentrically, e = 0.1D

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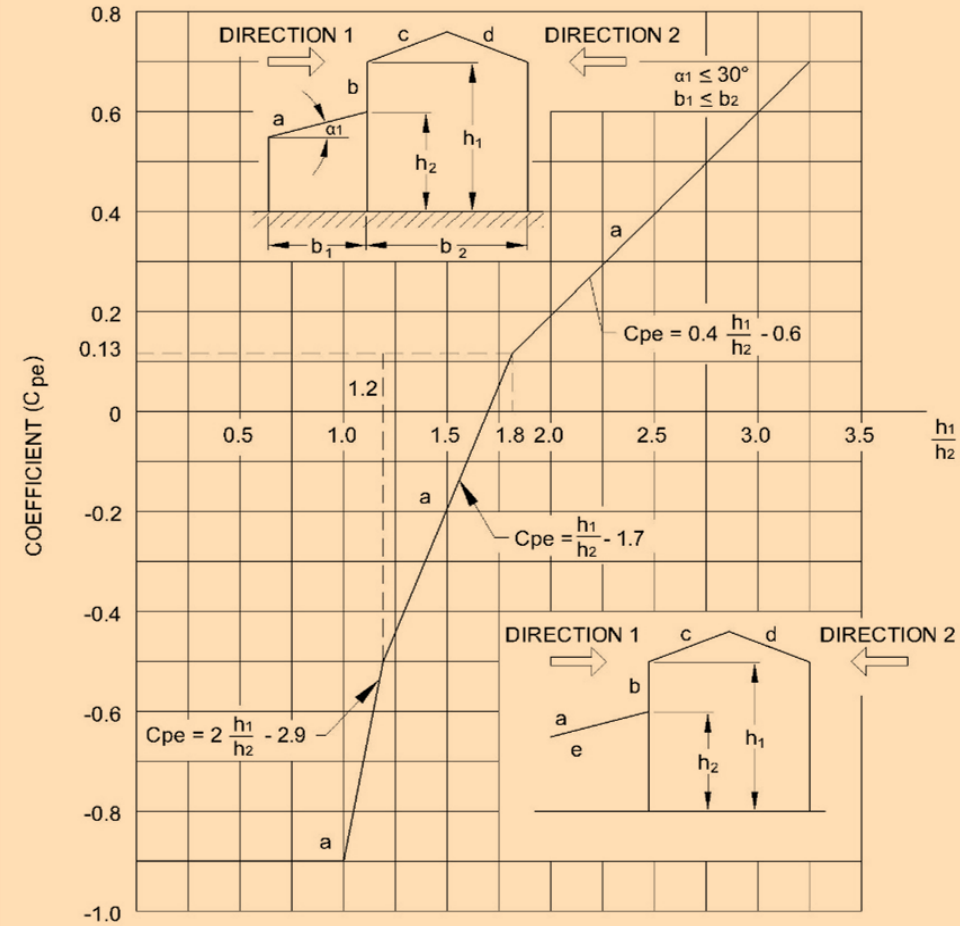
Table 24 External Pressure Coefficients (C_{pe}) for Combined Roofs

(*Clause* 4.5.3.3.9)



VALUES OF COEFFICIENTS (C_{pe})		
PORTION	DIRECTION 1	DIRECTION 2
a	FROM THE DIAGRAM	- 0.4
b	$C_{pe} = -0.5$, $\frac{h_1}{h_2} \leq 1.5$ $C_{pe} = +0.7$; $\frac{h_1}{h_2} > 1.5$	
c and d	See Table 6	
e	See Clause 6.3.3.5	

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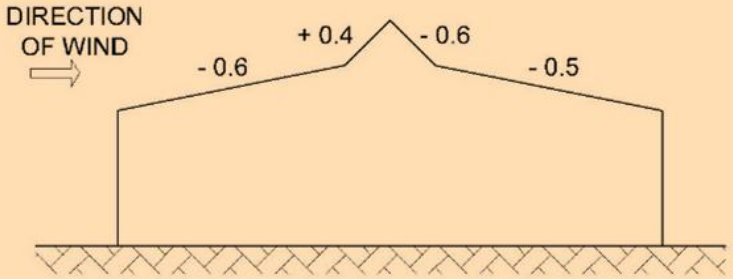
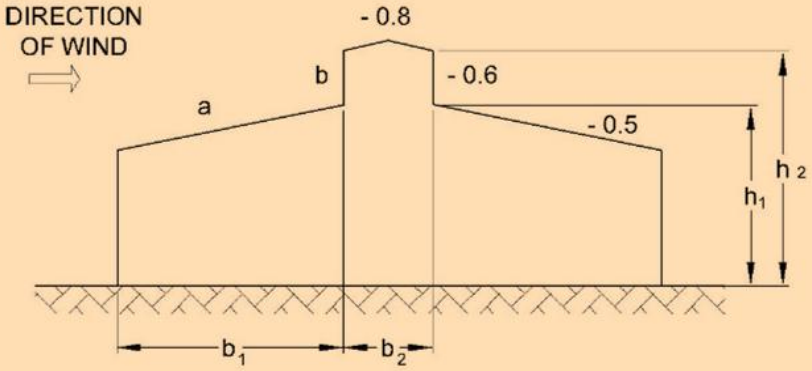
Table 25 External Pressure Coefficients (C_{pe}) for Roofs with a Sky Light

(*Clause* 4.5.3.3.10)





b) ROOFS WITH A SKY LIGHT

VALUES OF COEFFICIENTS (C_{pe})			
PORTION	$b_1 > b_2$		$b_1 \leq b_2$
	a	*b*	*a* and *b*
C_{pe}	- 0.6	+ 0.7	See Table for combined roofs

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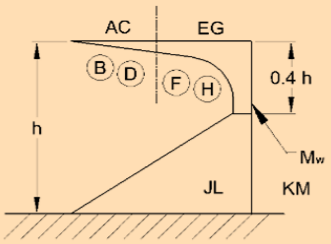
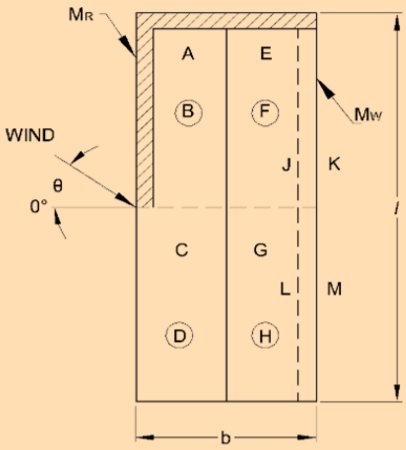
Table 26 Pressure Coefficients at Top and Bottom Roof of Grand Stands Open Three Sides (Roof Angle Up to 3°)

(*Clause* 4.5.3.3.11)





(Shaded area to scale)

FRONT AND BACK OF WALL				
θ	*J*	*K*	*L*	*M*
0°	+ 0.9	- 0.5	+ 0.9	- 0.5
45°	+ 0.8	- 0.6	+ 0.4	- 0.4
135°	- 1.1	+ 0.6	- 1.0	+ 0.4
180°	- 0.3	+ 0.9	- 0.3	+ 0.9
60°		*Mw* - Cp of K = - 1.0		
60°		*Mw* - Cp of J = + 1.0		

TOP AND BOTTOM ROOF								
θ	*A*	*B*	*C*	*D*	*E*	*F*	*G*	*H*
0°	- 1.0	+ 0.9	- 1.0	+ 0.9	- 0.7	+ 0.9	+ 0.7	+ 0.9
45°	- 1.0	+ 0.7	- 0.7	+ 0.4	- 0.5	+ 0.8	- 0.5	+ 0.3
135°	- 0.4	- 1.1	- 0.7	- 1.0	- 0.9	- 1.1	- 0.9	- 1.0
180°	- 0.6	- 0.3	- 0.6	- 0.3	- 0.6	- 0.3	- 0.6	- 0.3
45°	'M_R' - Cp (top) = - 2.0							
45°	'M_R' - Cp (bottom) = + 1.0							

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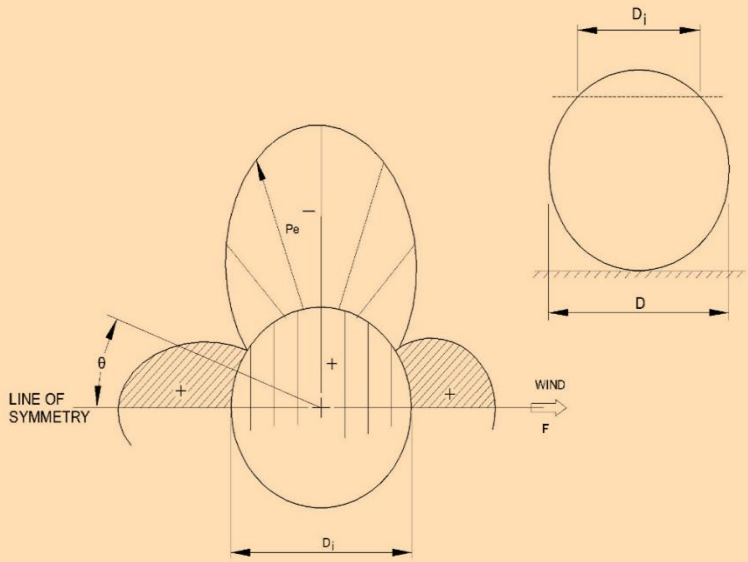
Table 27 External Pressure Coefficients (C_{pe}) Around Spherical Structures

(*Clause* 4.5.3.3.12)



POSITION OF PERIPHERY, θ IN DEGREES	Cpe
0	+ 1.0
15	+ 0.9
30	+ 0.5
45	- 0.1
60	- 0.7
75	- 1.1
90	- 1.2
105	- 1.0
120	- 0.6
135	- 0.2
150	+ 0.1
165	+ 0.3
180	+ 0.4

4.5.4.1 *Frictional drag*

In certain buildings of special shape, a force due to frictional drag shall be taken into account in addition to those loads specified in 4.5.3. For rectangular clad buildings, this addition is necessary only where the ratio d/h or d/b is more than 4. The frictional drag force, F', in the direction of the wind is given by the following formulae:

a) If $h \leq b$, $F' = C_f' (d - 4h)\, b\, p_d + C_f' (d - 4h)\, 2h\, p_d$; and

b) If $h > b$, $F' = C_f' (d - 4b)\, b\, p_d + C_f' (d - 4b)\, 2h\, p_d$.

The first term in each case gives the drag on the roof and the second on the walls. The C_f' has the following values:

a) 0.01 for smooth surfaces without corrugations or ribs across the wind direction;

b) 0.02 for surfaces with corrugations across the wind direction; and

c) 0.04 for surfaces with ribs across the wind direction.

For other buildings, the frictional drag has been indicated, where necessary, in the tables of pressure coefficients and force coefficients.

4.5.4.2 *Force coefficients for clad buildings*

4.5.4.2.1 *Clad buildings of uniform section*

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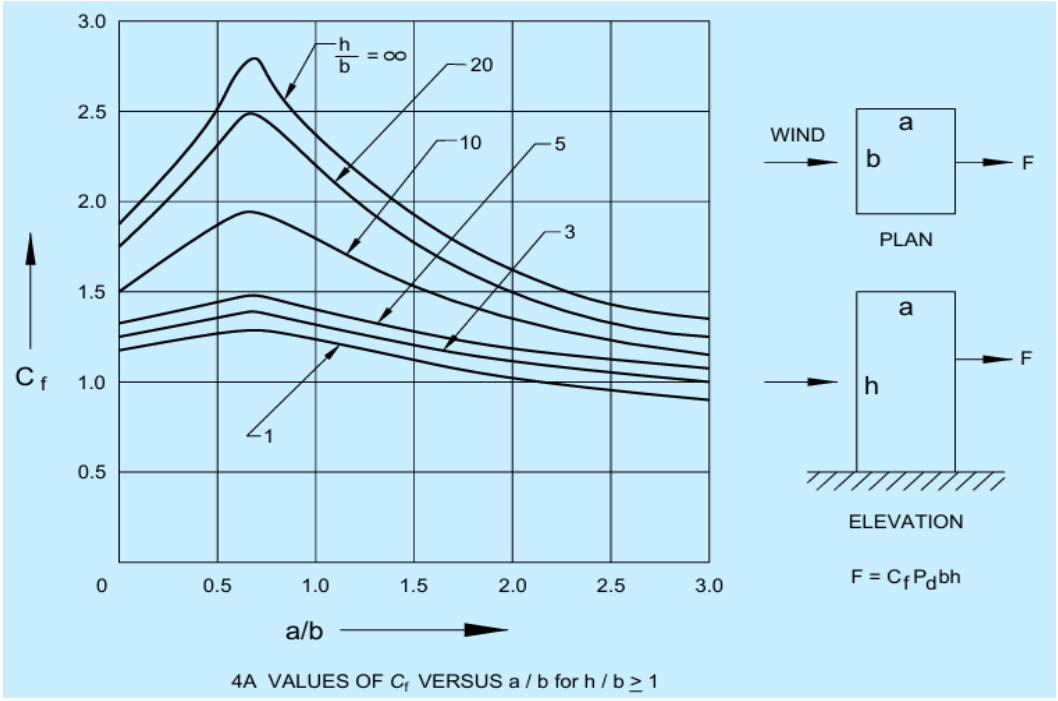
The overall force coefficients for rectangular clad buildings of uniform section with flat roofs in uniform flow shall be as given in Fig. 4 and for other clad buildings of uniform section (without projections, except where otherwise shown) shall be as given in Table 28.

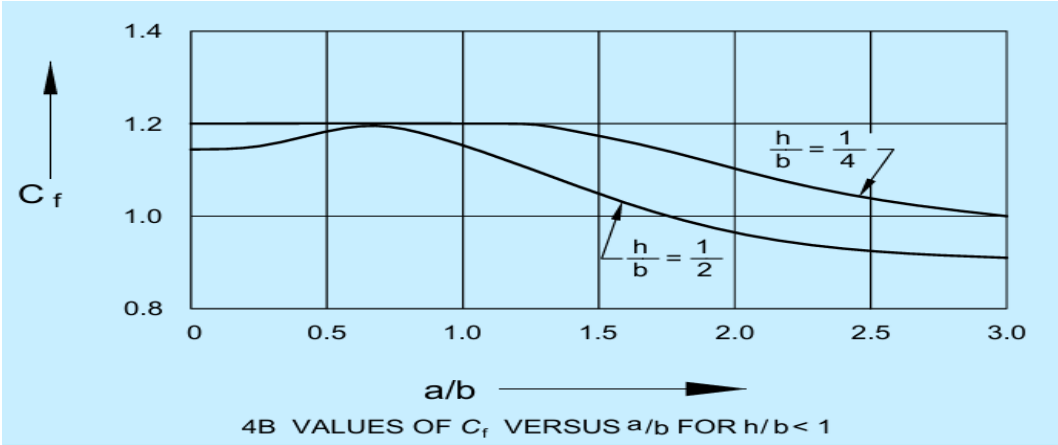
NOTE — Structures that are in the supercritical flow regime, because of their size and design wind velocity, may need further calculation to ensure that the greatest loads do not occur at some wind speed below the maximum, when the flow will be sub-critical.

The coefficients are for buildings without projections, except where otherwise shown.

In Table 28, $\overline{V}_d b$ is used as an indication of the airflow regime.



4A VALUES OF C_f VERSUS a / b for h / b ≥ 1



4B VALUES OF C_f VERSUS a/b FOR h/b < 1

FIG. 4 FORCE COEFFICIENT FOR RECTANGULAR CLAD BUILDING IN UNIFORM FLOW

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Table 28 Force Coefficients, C_f for Clad Buildings of Uniform Section (Acting in the Direction of Wind)

(*Clause* 4.5.4.2.1)

PLAN SHAPE	$\frac{\overline{V}_d\, b}{m^2/s}$	C_f FOR HEIGHT / BREADTH RATIO						
		UPTO 1/2	1	2	5	10	20	∞
WIND $\overline{V}_d$, b See also Appendix D	ALL SURFACES < 6	0.7	0.7	0.7	0.8	0.9	1.0	1.2
	ROUGH or WITH PROJECTION ≥ 6							
	SMOOTH ≥ 6	0.5	0.5	0.5	0.5	0.5	0.6	0.6
d, b Ellipse b/d = 1/2	< 10	0.5	0.5	0.5	0.5	0.6	0.6	0.7
	≥ 10	0.2	0.2	0.2	0.2	0.2	0.2	0.2
d, b Ellipse b/d = 2	< 8	0.8	0.8	0.9	1.0	1.1	1.3	1.7
	≥ 8	0.8	0.8	0.9	1.0	1.1	1.3	1.5
r b/d = 1 r/b = 1/3	< 4	0.6	0.6	0.6	0.7	0.8	0.8	1.0
	≥ 4	0.4	0.4	0.4	0.4	0.5	0.5	0.5
r b/d = 1 r/b = 1/6	< 10	0.7	0.8	0.8	0.9	1.0	1.0	1.3
	≥ 10	0.5	0.5	0.5	0.5	0.6	0.6	0.6
d, r, b b/d = 1/2 r/b = 1/2	< 3	0.3	0.3	0.3	0.3	0.3	0.3	0.4
	≥ 3	0.2	0.2	0.2	0.2	0.3	0.3	0.3
d, r, b b/d = 1/2 r/b = 1/6	All values	0.5	0.5	0.5	0.5	0.6	0.6	0.7
d, b b/d = 2 r/b = 1/12	All values	0.9	0.9	1.0	1.1	1.2	1.5	1.9

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Table 28 (*Continued*)

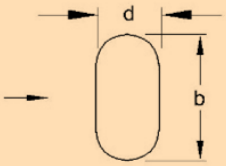
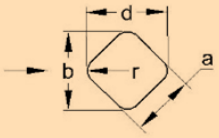
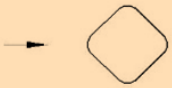
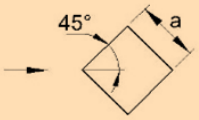
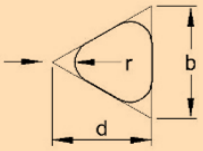
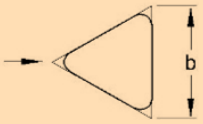
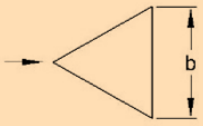
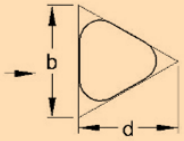
PLAN SHAPE	$\frac{\bar{V}_d\ b}{m^2/s}$	C_f FOR HEIGHT / BREADTH RATIO						
		UPTO 1/2	1	2	5	10	20	∞
 b/d = 2 r/b = 1/4	< 6	0.7	0.8	0.8	0.9	1.0	1.2	1.6
	≥ 6	0.5	0.5	0.5	0.5	0.5	0.6	0.6
 r/a = 1/3	< 10	0.8	0.8	0.9	1.0	1.1	1.3	1.5
	≥ 10	0.5	0.5	0.5	0.5	0.5	0.6	0.6
 r/a = 1/12	All values	0.9	0.9	0.9	1.1	1.2	1.3	1.6
 r/a = 1/48	All values	0.9	0.9	0.9	1.1	1.2	1.3	1.6
 r/b = 1/4	< 11	0.7	0.7	0.7	0.8	0.9	1.0	1.2
	≥ 11	0.4	0.4	0.4	0.4	0.5	0.5	0.5
 r/b = 1/12	All values	0.8	0.8	0.8	1.0	1.1	1.2	1.4
 r/b = 1/48	All values	0.7	0.7	0.8	0.9	1.0	1.1	1.3
 r/b = 1/4	< 8	0.7	0.7	0.8	0.9	1.0	1.1	1.3
	≤ 8	0.4	0.4	0.4	0.4	0.5	0.5	0.5

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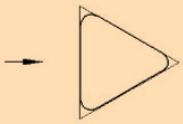
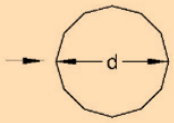
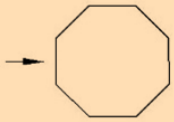
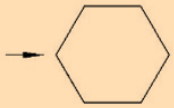
Table 28 (*Concluded*)

PLAN SHAPE	$\frac{\bar{V}_d\, b}{m^2/s}$	C_f FOR HEIGHT / BREADTH RATIO						
		UPTO 1/2	1	2	5	10	20	∞
 1/48 < r/b < 1/12	All values	1.2	1.2	1.2	1.4	1.6	1.7	2.1
 12 SIDED POLYGON	< 12	0.7	0.7	0.8	0.9	1.0	1.1	1.3
	≥ 12	0.7	0.7	0.7	0.7	0.8	0.9	1.1
 OCTAGON	All values	1.0	1.0	1.1	1.2	1.2	1.3	1.4
 HEXAGON	All values	1.0	1.1	1.2	1.3	1.4	1.4	1.5

4.5.4.2.2 *Buildings of circular shapes*

Force coefficients for buildings of circular cross section shapes shall be as given in Table 28. However, more precise estimation of force coefficients for circular shapes of infinite length can be obtained from Fig. 5 taking into account the average height of surface roughness ε. When the length is finite the values obtained from Fig. 5 shall be reduced by the multiplication factor K (*See also* Table 31 and Annex G).

4.5.4.2.3 *Free standing walls and hoardings*

Force coefficients for free standing walls and hoardings shall be as given in Table 29.

To allow for oblique winds, the design shall also be checked for net pressure normal to the surface varying linearly from a maximum of 1.7 C_f at the windward edge to 0.44 C_f at the leeward edge.

The wind load on appurtenances and supports for hoardings shall be accounted for separately by using the appropriate net pressure coefficients. Allowance shall be made for shielding effects of one element on another.

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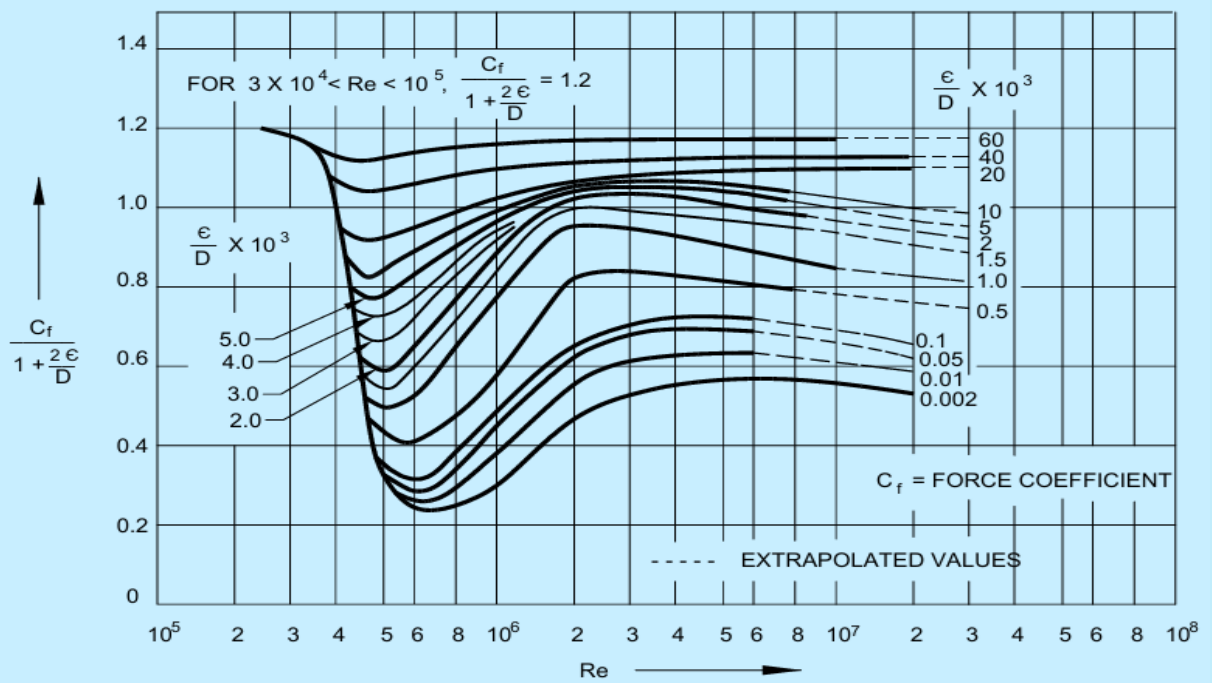
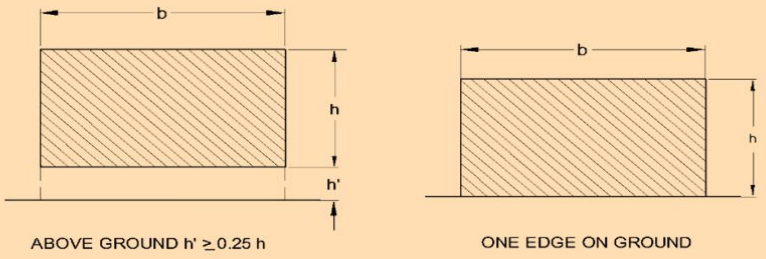


Fig. 5 VARIATION OF $\frac{C_f}{1+\frac{2\epsilon}{D}}$ WITH Re $< 3 \times 10^4$ FOR CIRCULAR SECTIONS

Table 29 Force Coefficients, C_f for Low Walls or Hoardings (< 15m High)

(*Clause* 4.5.4.2.3)



(WIND NORMAL TO FACE)

WIDTH TO HEIGHT RATIO, *b/h*		FORCE COEFFICIENT DRAG COEFFICIENT C_f
WALL ABOVE GROUND	WALL ON GROUND	
FROM 0.5 TO 6	FROM 1 TO 12	1.2
10	20	1.3
16	32	1.4
20	40	1.5
40	80	1.75
60	120	1.8
80 OR MORE	160 OR MORE	2.0

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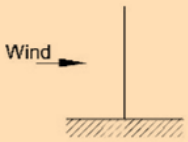
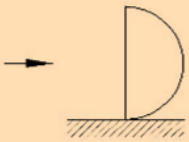
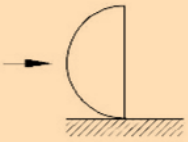
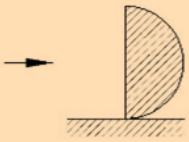
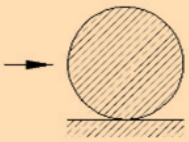
Table 30 Force Coefficients, C_f for Solid Shapes Mounted on a Surface
(*Clause* 4.5.4.2.4)

SIDE ELEVATION	DESCRIPTION OF SHAPE	C_f
	CIRCULAR DISC	1.2
	HEMISPHERICAL BOWL	1.4
	HEMISPHERICAL BOWL	0.4
	HEMISPHERICAL SOLID	1.2
	SPHERICAL SOLID	0.5 FOR V_z D < 7 0.2 FOR V_z D ≥ 7

4.5.4.2.4 *Solid circular shapes mounted on a surface*

The force coefficients for solid circular shapes mounted on a surface shall be as given in Table 30.

4.5.4.3 *Force coefficients for unclad buildings*

4.5.4.3.1 This section applies to permanently unclad buildings and to frameworks of buildings while temporarily unclad. In the case of buildings whose surfaces are well-rounded, such as those with elliptic, circular or oval cross sections, the total force can be more at a wind speed much less than maximum due to transition in the nature of boundary layer on them. Although this phenomenon is well known in the case of circular cylinders, the same phenomenon exists in the case of many other well-rounded structures and this possibility must be checked.

4.5.4.3.2 *Individual members*

a) The force coefficient given in Table 32 refers to members of infinite length. For members of finite length, the coefficients should be multiplied by a factor K that depends on the ratio l/b where l is the length of the member and b is the width across the direction of wind. Table 31 gives the required values of K. The following special cases must be noted while estimating K:

   1) when any member abuts on to a plate or wall in such a way that free flow of air around that end of the member is prevented, then the ratio of l/b shall be doubled for the purpose of determining K; and

   2) when both ends of a member are so obstructed, the ratio shall be taken as infinity for the purpose of determining K.

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b) *Flat-sided members* — Force coefficients for wind normal to the longitudinal axis of flat-sided structural members shall be as given in Table 32.

The force coefficients are given for two mutually perpendicular directions relative to a reference axis on the structural member. They are denoted by C_{fn} and C_{ft} and give the forces normal and transverse, respectively to the reference plane as shown in Table 32.

Normal force $F_n = (C_{fn}\, p_d\, K)/b$

Transverse force $F_t = (C_{ft}\, p_d\, K)/b$

c) *Circular sections* — Force coefficients for members of circular section shall be as given in Table 28 (*See also* Annex G).

d) Force coefficients for wires and cables shall be as given in Table 33 according to the diameter (D), the design wind speed (V_d) and the surface roughness.

Table 31 Reduction Factor *K* for Individual Members

[*Clauses* 4.5.4.2.2, 4.5.4.3.2(a) *and* 4.5.4.3.3(b)(2)]

Sl No.	***l*/*b* or *l*/*D***	**2**	**5**	**10**	**20**	**40**	**50**	**100**	∞
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
i)	Circular cylinder, sub-critical flow	0.58	0.62	0.68	0.74	0.82	0.87	0.98	1.00
ii)	Circular cylinder, supercritical flow ($DV_d \geq 6$ m²/s)	0.80	0.80	0.82	0.90	0.98	0.99	1.00	1.00
iii)	For plate perpendicular to wind ($bV_d \geq 6$ m²/s)	0.62	0.66	0.69	0.81	0.87	0.90	0.95	1.00

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Table 32 Force Coefficients, C_f for Individual Structural Members of Infinite Length

[*Clause* 4.5.4.3.2(b)]

θ DEGREE																				
	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}	C_{fn}	C_{ft}
0	+ 1.9	+ 0.95	+ 1.8	+ 1.8	+ 1.75	+ 0.1	+ 1.6	0	+ 2.0	0	+ 2.0	0	+ 1.6	0	+ 2.0	0	+ 2.05	0		
45	+ 1.8	+ 0.8	+ 2.1	+ 1.8	+ 0.85	+ 0.85	+ 1.5	- 0.1	+ 1.2	+ 0.9	+ 1.8	+ 0.1	+ 1.5	+ 1.5	+ 1.55	+ 1.55	+ 1.85	+ 0.6		
90	+ 2.0	+ 1.7	- 1.9	- 1.0	+ 0.1	+ 1.75	- 0.95	+ 0.7	- 1.6	+ 2.15	+ 1.55	+ 1.55	0	0	0	0	0	+ 0.6		
135	- 1.8	- 0.1	- 2.0	+ 0.3	- 0.75	+ 0.75	- 0.5	+ 1.05	- 1.1	+ 2.4							- 1.6	+ 0.4		
180	- 2.0	+ 0.1	- 1.4	- 1.4	- 1.75	- 0.1	- 1.5	0	- 1.7	+ 2.1							- 1.8	0		

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Table 33 Force Coefficients, C_f for Wires and Cables (l/D = 100)

[*Clause* 4.5.4.3.2(d)]

Sl No.	Flow Regime	Force Coefficient, C_f for			
		Smooth Surface	Moderately Smooth Wire (Galvanized or Painted)	Fine Stranded Cables	Thick Stranded Cables
(1)	(2)	(3)	(4)	(5)	(6)
i)	$D\bar{V}_d < 6$ m²/s	1.2	1.2	1.2	1.3
ii)	$D\bar{V}_d \geq 6$ m²/s	0.5	0.7	0.9	1.1

4.5.4.3.3 *Single frames*

Force coefficients for a single frame having either:

a) all flat sided members; or

b) all circular members in which all the members of the frame have either:

1) $D\bar{V}_d$ less than 6 m²/s; or

2) $D\bar{V}_d$ more than or equal to 6 m²/s. shall be as given in Table 31 according to the type of the member, the diameter (D), the design hourly mean wind speed ($\bar{V}_d$) and the solidity ratio (Φ). Force coefficients for a single frame not complying with the above requirements shall be calculated as follows:

$$C_f = \gamma C_{f\,super} + (1-\gamma)\frac{A_{circ\,sub}}{A_{sub}} C_{f\,sub} + (1-\gamma)\frac{A_{flat}}{A_{sub}} C_{f\,flat}$$

where

$C_{f\ super}$ = force coefficient for the supercritical circular members as given in Table 34 or Annex G;

γ = $\frac{\text{Area of the frame in a supercritical flow}}{A_e}$;

A_e = effective frontal area;

$A_{circ\ sub}$ = effective area of subcritical circular members;

$C_{f\ sub}$ = force coefficient for subcritical circular members as given in Table 34 or Annex G;

$C_{f\ flat}$ = force coefficient for the flat sided members as given in Table 34;

A_{flat} = effective area of flat-sided members; and

A_{sub} = Acirc sub + Aflat.

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Table 34 Force Coefficients for Single Frames

(*Clause* 4.5.4.3.3)

Sl No.	Solidity Ratio, Φ	Force Coefficient, C_f For		
		Flat Sided Members	Circular Sections	
			Subcritical Flow ($D\bar{V}_d < 6$ m²/s)	Super Critical Flow ($D\bar{V}_d \geq 6$ m²/s)
(1)	(2)	(3)	(4)	(5)
i)	0.1	1.9	1.2	0.7
ii)	0.2	1.8	1.2	0.8
iii)	0.3	1.7	1.2	0.8
iv)	0.4	1.7	1.1	0.8
v)	0.5	1.6	1.1	0.8
vi)	0.75	1.6	1.5	1.4
vii)	1.00	2.0	2.0	2.0

NOTE — Linear interpolation between the values is permitted.

4.5.4.3.4 *Multiple frame buildings*

This Section applies to structures having two or more parallel frames where the windward frames may have a shielding effect upon the frames to leeward side. The windward frame and any unshielded parts of other frames shall be calculated in accordance with **4.5.4.3.3**, but the wind load on the parts of frames that are sheltered should be multiplied by a shielding factor which is dependent upon the solidity ratio of the windward frame, the types of members comprising the frame and the spacing ratio of the frames. The values of the shielding factors are given in Table 35.

Where there are more than two frames of similar geometry and spacing, the wind load on the third and subsequent frames should be taken as equal to that on the second frame. The loads on the various frames shall be added to obtain total load on the structure.

a) The frame spacing ratio is equal to the centre to centre distance between the frames, beams or girders divided by the least overall dimension of the frames, beam or girder measured in a direction normal to the direction of wind. For triangular framed structures or rectangular framed structures diagonal to the wind, the spacing ratio should be calculated from the mean distance between the frames in the direction of the wind.

b) Effective solidity ratio, Φ_e:

$\Phi_e = \Phi$ for flat-sided members.

Φ_e is to be obtained from Fig. 6 for members of circular cross sections.

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Table 35 Shielding Factor, η for Multiple Frames

(*Clause* 4.5.4.3.4)

Sl No.	Effective Solidity Ratio, Φ_e	Frame Spacing Ratio				
		< 0.5	1.0	2.0	4.0	> 8.0
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	0	1.0	1.0	1.0	1.0	1.0
ii)	0.1	0.9	1.0	1.0	1.0	1.0
iii)	0.2	0.8	0.9	1.0	1.0	1.0
iv)	0.3	0.7	0.8	1.0	1.0	1.0
v)	0.4	0.6	0.7	1.0	1.0	1.0
vi)	0.5	0.5	0.6	0.9	1.0	1.0
vii)	0.7	0.3	0.6	0.8	0.9	1.0
viii)	1.0	0.3	0.6	0.6	0.8	1.0

NOTE — Linear interpolation between the values is permitted.

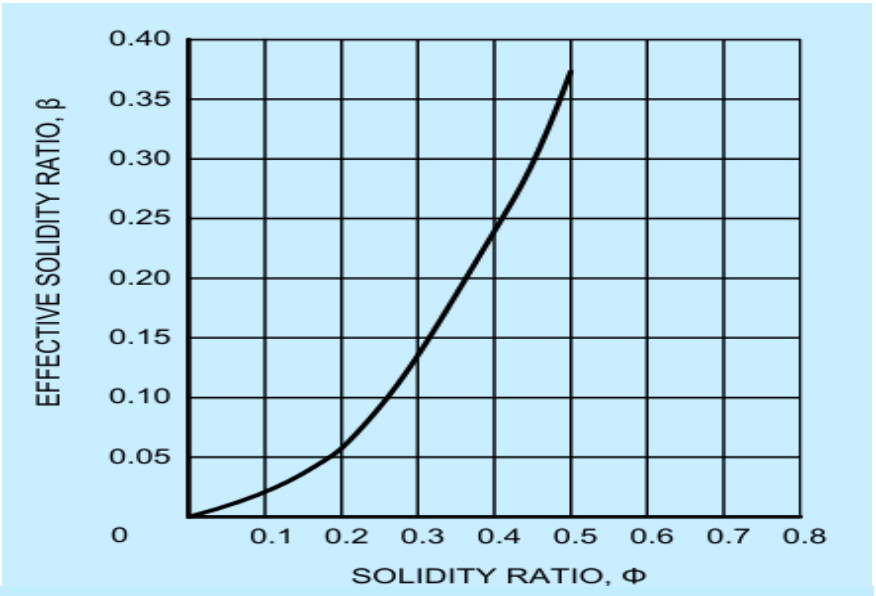

FIG. 6 EFFECTIVE SOLIDITY RATIO FOR CIRCULAR SECTION MEMBERS

4.5.4.3.5 *Lattice towers*

a) Force coefficient for lattice towers of square or equilateral triangle section with flat sided members for wind blowing against any face shall be as given in Table 36;

b) For square lattice towers with flat-sided members the maximum load, which occurs when the wind blows into a corner, shall be taken as 1.2 times the load for the wind blowing against a face;

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c) For equilateral triangle lattice towers with flat-sided members, the load may be assumed to be constant for any inclination of wind to a face;

d) Force coefficients for lattice towers of square section with circular members, all in the same flow regime, may be as given in Table 37; and

e) Force coefficients for lattice towers of equilateral-triangle section with circular members all in the same flow regime may be as given in Table 38.

4.5.4.3.6 *Tower appurtenances*

The wind loading on tower appurtenances, such as ladders, conduits, lights, elevators, etc shall be calculated using appropriate net pressure coefficients for these elements. Allowance may be made for shielding effect from other elements.

Table 36 Overall Force Coefficients for Towers Composed of Flat Sided Members
[*Clause* 4.5.4.3.5(a)]

Sl No.	Solidity Ratio, Φ	Force Coefficient	
		Square Towers	Equilateral Triangular Towers
(1)	(2)	(3)	(4)
i)	< 0.1	3.8	3.1
ii)	0.2	3.3	2.7
iii)	0.3	2.8	2.3
iv)	0.4	2.3	1.9
v)	0.5	2.1	1.5

Table 37 Overall Force Coefficients for Square Towers Composed of Circular Members
[*Clause* 4.5.4.3.5(d)]

Sl No.	Solidity Ratio of Front Face Φ	Force Coefficient			
		Subcritical Flow ($D\bar{V}_d < 6$ m²/s)		Super Critical Flow ($D\bar{V}_d \geq 6$ m²/s)	
		Onto Face	Onto Corner	Onto Face	Onto Corner
(1)	(2)	(3)	(4)	(5)	(6)
i)	< 0.05	2.4	2.5	1.1	1.2
ii)	0.1	2.2	2.3	1.2	1.3
iii)	0.2	1.9	2.1	1.3	1.6
iv)	0.3	1.7	1.9	1.4	1.6
v)	0.4	1.6	1.9	1.4	1.6
vi)	0.5	1.4	1.9	1.4	1.6

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Table 38 Overall Force Coefficients for Equilateral Triangular Towers Composed of Circular Members

[*Clause* 4.5.4.3.5(e)]

Sl No.	Solidity Ratio of Front Face Φ	Force Coefficient	
		Subcritical flow ($D\overline{V}_d < 6$ m²/s)	Super critical flow ($D\overline{V}_d \geq 6$ m²/s)
		All wind directions	All wind directions
(1)	(2)	(3)	(4)
i)	< 0.05	1.8	0.8
ii)	0.1	1.7	0.8
iii)	0.2	1.6	1.1
iv)	0.3	1.5	1.1
v)	0.4	1.5	1.1
vi)	0.5	1.4	1.2

4.6 Interference Effects

4.6.1 *General*

Wind interference is caused by modification in the wind characteristics produced by the obstruction caused by an object or a building/structure in the path of the wind. If such wind strikes another structure, the wind pressures usually get enhanced, though there can also be some shielding effect between two very closely spaced buildings/structures. The actual phenomenon is too complex to justify generalization of the wind forces/pressures produced due to interference which can only be ascertained by detailed wind tunnel/CFD studies. However, some guidance can be provided for the purpose of preliminary design. To account for the effect of interference, a wind interference factor (IF) has been introduced as a multiplying factor to be applied to the design wind pressure/force. Interference effects can be more significant for tall buildings. The interference factor is defined as the ratio between the enhanced pressure/force in the grouped configuration to the corresponding pressure/force in isolated configuration. Since the values of IF can vary considerably based on building geometry and location, the given values of IF are a kind of median values and are meant only for preliminary design estimates. The designer is advised that for assigning values of IF for final design particularly for tall buildings, specialist literature be consulted or a wind tunnel study carried out.

4.6.2 *Roof of Low Rise Buildings*

Maximum increase in wind force on the roof due to interference from similar buildings in case of closely spaced low rise buildings with flat roofs may be up to 25 percent for c/c distance (x) between the buildings of 5 times the dimension (b) of the interfering building normal to the direction of wind (*see* Fig. 7). Interference effect beyond 20b may be considered to be negligible. For intermediate spacing linear interpolation may be used.

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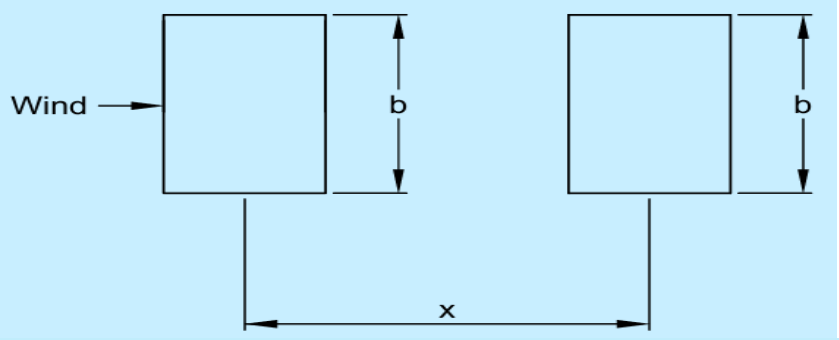


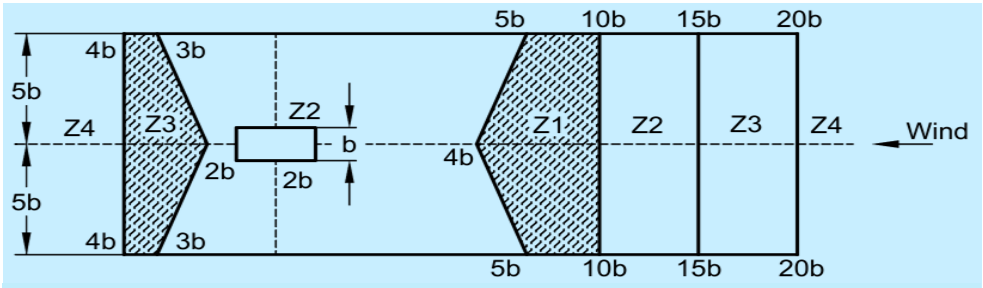
FIG. 7 LOW-RISE BUILDING IN TANDEM CAUSING INTERFERENCE EFFECT

4.6.3 *Tall Buildings*

Based on studies on tall rectangular buildings, Fig. 8 gives various zones of interference. The interference factor (IF), which needs to be considered as a multiplication factor for wind loads corresponding to isolated building, may be assumed as follows for preliminary estimate of the wind loads under interference caused by another interfering tall building of same or more height located at different zone Z1 to zone Z4 as shown in Fig. 8:

Zone	Z1	Z2	Z3	Z4
IF	1.35	1.25	1.15	1.07

The interference effect due to buildings of height less than one-third of the height of the building under consideration may be considered to be negligible while for interference from a building of intermediate height, linear interpolation may be used between one-third and full height.



Z1 Zone of high interference
Z2 Zone of moderate interference
Z3 Zone of low interference
Z4 Zone of insignificant interference

FIG. 8 INTERFERENCE ZONES FOR TALL RECTANGULAR BUILDINGS OF SAME OR GREATER HEIGHT

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4.7 Dynamic Effects

4.7.1 *General*

Flexible slender structures and structural elements shall be investigated to ascertain the importance of wind induced oscillations or excitations in along wind and across wind directions.

In general, the following guidelines may be used for examining the problems of wind-induced oscillations:

a) Buildings and closed structures with a height to minimum lateral dimension ratio of more than about 5.0, or

b) Buildings and structures whose natural frequency in the first mode is less than 1.0 Hz.

Any building or structure which satisfies either of the above two criteria shall be examined for dynamic effects of wind.

NOTES

1 The fundamental time period (T), in second, may either be established by experimental observations on similar buildings or calculated by any rational method of analysis. In the absence of such data, T may be determined as follows for multi-storied buildings:

a) For moment resistant frames without bracings or shear walls resisting the lateral loads:

$$T = 0.1\ n$$

where

n = number of storeys, including basement storeys.

b) For all others:

$$T = \frac{0.09H}{\sqrt{d}}$$

where

H = total height of the main structures of the building, in m; and

d = maximum base dimension of building in metres in a direction parallel to the applied wind force.

2 If preliminary studies indicate that wind-induced oscillations are likely to be significant, investigations should be pursued with the aid of analytical methods or if necessary, by means of wind tunnel tests on models.

3 Across-wind motions may be due to lateral gustiness of the wind, unsteady wake flow (for example, vortex shedding), negative aerodynamic damping or due to a combination of these effects. These cross-wind motions may become critical in the design of tall buildings/structures.

4 Motions in the direction of wind (also known as buffeting) are caused by fluctuating wind force associated with gust. The excitation depends on gust energy available at the resonant frequency.

5 The eddies shed from an upstream body may intensify motion in the direction of the wind and may also affect cross-wind motion.

6 The designer should also be aware of the following three forms of wind-induced motion which are characterized by increasing amplitude of oscillation with the increase of wind speed.

a) *Galloping* — Galloping is transverse oscillations of some structures due to the development of aerodynamic forces which are in phase with the motion. It is characterized by the progressively increasing amplitude of transverse vibration with increase of wind speed. The cross sections which are particularly prone to this type of excitation include the following:

1) All structures with non-circular cross sections, such as triangular, square, polygons, as well as angles, crosses and T sections.
2) Twisted cables and cables with ice encrustations.

b) *Flutter* — Flutter is unstable oscillatory motion of a structure due to coupling between aerodynamic force and elastic deformation of the structure. Perhaps the most common form is oscillatory motion due to combined bending and torsion. Although oscillatory motion in each degree of freedom may be damped, instability can set in due to energy transfer from one mode of oscillation to another and the structure is seen to execute sustained or divergent oscillations with a type of motion which is a combination of the individual modes of vibration. Such energy transfer takes place when the natural frequencies of modes taken individually are close to each other (ratio being typically less than 2.0). Flutter can set in at wind speeds much less than those required for exciting the individual modes of motion. Long span suspension bridge decks or any member of a structure with large values of d/t (where d is

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the length of the member and t is its dimension parallel to wind stream) are prone to low speed flutter. Wind tunnel testing is required to determine critical flutter speeds and the likely structural response. Other types of flutter are single degree of freedom stall flutter, torsional flutter, etc.

c) *Ovalling* — Thin walled structures with open ends at one or both ends such as oil storage tanks and natural draught cooling towers in which the ratio of the diameter or minimum lateral dimension to the wall thickness is of the order of 100 or more are prone to ovalling oscillations. These oscillations are characterized by periodic radial deformation of the hollow structure.

7 Buildings and structures that may be subjected to significant wind excited oscillations require careful investigations. It is to be noted that wind induced oscillations may occur at wind speeds lower than the design wind speed.

8 Analytical methods for the evaluation of response of dynamic structures to wind loading can be found in the special publications.

9 In assessing wind loads due to such dynamic phenomenon as galloping, flutter and ovalling, in the absence of the required information either in the special publications or other literature, expert advice should be sought including experiments on models in boundary layer wind tunnels.

4.7.2 *Motion due to Vortex Shedding*

4.7.2.1 *Slender structures*

For a structure, the vortex shedding frequency, fs, shall be determined by the following formula:

$$fs = \frac{S_t \overline{V}_{z,H}}{b}$$

where

S_t = strouhal number

$\overline{V}_{z,\ H}$ = hourly mean wind speed at height z; and

b = breadth of a structure or structural member normal to the wind direction in the horizontal plane.

a) *Circular structures* — For structures of circular cross section:

S_t = 0.20 for $D\overline{V}_{z,H}$ less than 6 m^2/s; and

= 0.25 for $D\overline{V}_{z,H}$ more than or equal to 6 m^2/s.

b) *Rectangular structures* — For structures of rectangular cross section:

$$S_t = 0.10$$

NOTES

1 Significant cross wind motions may be produced by vortex shedding, if the natural frequency of the structure or structural element is equal to the frequency of the vortex shedding within the range of expected wind speeds. In such cases, further analysis should be carried out on the basis of special publications.

2 Unlined welded steel cylindrical structures are prone to excitations by vortex shedding.

3 Intensification of the effects of periodic vortex shedding has been reported in cases where two or more similar structures are located in close proximity, for example at less than 20b apart, where b is the dimension of the structure normal to the wind.

4 The formulae given in 4.7.2.1(a) is valid for infinitely long cylindrical structures. The value of S_t decreases slowly as the ratio of length to maximum transverse width decreases, the reduction being up to about half the value, if the structure is only three times higher than its width. Vortex shedding need not be considered, if the ratio of length to maximum transverse dimension is less than 2.0.

4.8 Dynamic Wind Response

4.8.1 *General*

Tall buildings/structures which are 'wind sensitive' shall be designed for dynamic wind loads. Hourly mean wind speed is used as a reference wind speed to be used in dynamic wind analysis. For calculation of along wind loads and response (bending moments, shear forces, or tip deflections) the Gust factor (GF) method is used as specified in 4.8.2. The across wind design peak base overturning moment and tip deflection shall be calculated using 4.8.3.

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4.8.2 *Along Wind Response*

For calculation of along-wind load effects at a level s on a building/structure, the design hourly mean wind pressure at height *z* shall be multiplied by the Gust factor (*G*). This factor is dependent on both the overall height *h* and the level *s* under consideration (*see* Fig. 9). For calculation of base bending moment and deflection at the top of the building/structure, '*s*' should be taken as zero.

Note that 0 < *s* < h and *s* < *z* < *h*.

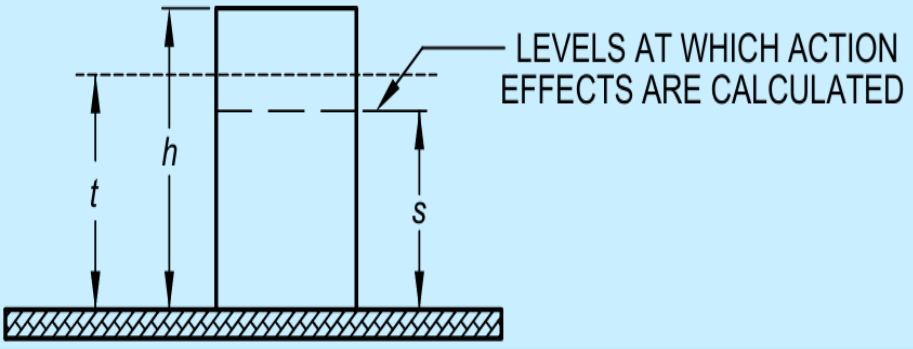

FIG. 9 NOTATIONS FOR HEIGHTS

The design peak along wind base bending moment (M_a) shall be obtained by summing the moments resulting from design peak along wind loads acting at different heights, z, along the height of the building/structure and can be obtained from:

$$M_a = \sum F_z z$$

$$F_z = C_{f,z} A_z \bar{p}_d G$$

where

F_z = design peak along wind load on the building/structure at any height *z;*

A_z = the effective frontal area of the building/structure at any height *z*, in m^2;

$\bar{p}_d$ = design hourly mean wind pressure corresponding to $\bar{V}_{z,d}$ and obtained as $0.6\bar{V}^2_{z,d}$ (N/m^2);

$\bar{V}_{z,d}$ = design hourly mean wind speed at height *z*, in m/s (*see* **4.4.4**);

$C_{f,z}$ = the drag force coefficient of the building/structure corresponding to the area A_z;

G = Gust Factor and is given by:

$$1 + r\sqrt{\left[g_v^2 B_s (1+\phi)^2 + \frac{H_s g_R^2 SE}{\beta} \right]}$$

r = roughness factor which is twice the longitudinal turbulence intensity, $I_{h,i}$ (*see* **4.4.5**);

g_v = peak factor for upwind velocity fluctuation

= 3.0 for category 1 and 2 terrains; and

= 4.0 for category 3 and 4 terrains;

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B_s = background factor indicating the measure of slowly varying component of fluctuating wind load cused by the lower frequency wind speed variations

$$= \frac{1}{\left[1+\frac{\sqrt{0.26(h-s)^2+0.46b_{sh}^2}}{L_h}\right]}$$

where

b_{sh} = average breadth of the building/structure between heights s and h;

L_h = measure of effective turbulence length scale at the height, h, in m;

$= 85\left(\frac{h}{10}\right)^{0.25}$ for terrain category 1 to 3; and

$= 70\left(\frac{h}{10}\right)^{0.25}$ for terrain category 4

ϕ = factor to account for the second order turbulence intensity;

$= \frac{g_v I_{h,i}\sqrt{B_s}}{2}$

$I_{h,i}$ = turbulence intensity at height h in terrain category i;

H_s = height factor for resonance response;

$$= 1+\left(\frac{s}{h}\right)^2$$

S = size reduction factor given by

$$= \frac{1}{\left[1+\frac{3.5f_a h}{\overline{V}_{h,d}}\right]\left[1+\frac{4f_a b_{0h}}{\overline{V}_{h,d}}\right]}$$

where

b_{0h} = average breadth of the building/structure between 0 and h;

E = spectrum of turbulence in the approaching wind stream:

$$\frac{\pi N}{\left(1+70.8N^2\right)^{5/6}}$$

where

N = an effective reduced frequency

$$= \frac{f_a L_h}{\overline{V}_{h,d}}$$

f_a = first mode natural frequency of the building/structure in along wind direction, in Hz

$\overline{V}_{h,d}$ = design hourly mean wind speed at height, h in m/s (*see* **4.4.4**);

β = damping coefficient of the building/structure (*see* Table 39); and

g_R = peak factor for resonant response.

$$= \sqrt{[2\ln(3600 f_a)]}$$

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Table 39 Suggested Values of Structural Damping Coefficients

(*Clauses* 4.8.2 *and* 4.8.3.1)

Sl No.	Kind of Structure	Damping Coefficient, β
(1)	(2)	(3)
i)	Welded steel structures	0.010
ii)	Bolted steel structures/RCC structures	0.020
iii)	Prestressed concrete structures	0.016

4.8.2.1 *Peak acceleration in along wind direction*

The peak acceleration at the top of the building/structure in along wind direction ($\hat{\ddot{x}}$ in m/s^2) is given by the following equation:

$$\hat{\ddot{x}} = (2\pi f_a)^2 \bar{x}\, g_R\, r \sqrt{\frac{SE}{\beta}}$$

where

$\bar{x}$ = mean deflection at the position where the acceleration is required. Other notations are same as given in 9.2. For computing the peak acceleration in the along wind direction, a mean wind speed at the height of the building/structure, $\overline{V_h}$ corresponding to a 5 year mean return period shall be used. A reduced value of 0.011 is also suggested for the structural damping, β for reinforced concrete structures.

4.8.3 *Across Wind Response*

This section gives method for determining equivalent static wind load and base overturning moment in the across wind direction for tall enclosed buildings and towers of rectangular cross section. Calculation of across wind response is not required for lattice towers.

The across wind design peak base bending moment M_c for enclosed buildings and towers shall be determined as follows:

$$M_c = 0.5 g_h \bar{p}_h b h^2 (1.06 - 0.06k) \sqrt{\frac{\pi C_{fs}}{\beta}}$$

where

g_h = a peak factor;

= $\sqrt{[2\ln(3600\, f_c)]}$ in cross wind direction;

$\bar{p}_h$ = hourly mean wind pressure at height h, in Pa;

b = breadth of the structure normal to the wind, in m;

h = height of the structure, in m;

k = a mode shape power exponent for representation of the fundamental mode shape as represented by:

$$\psi(z) = \left(\frac{z}{h}\right)^k \text{ ; and}$$

f_c = first mode natural frequency of the building/structure in across wind direction, in Hz.

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The across wind load distribution on the building/structure can be obtained from M_c using linear distribution of loads as given below:

$$F_{z,c} = \left(\frac{3M_c}{h^2}\right)\left(\frac{z}{h}\right)$$

where

$F_{z,c}$ = across wind load per unit height at height z.

4.8.3.1 *Peak acceleration in across wind direction*

The peak acceleration at the top of the building/structure in across-wind direction ($\hat{\ddot{y}}$ in m/s^2) with approximately constant mass per unit height shall be determined as follows:

$$\hat{\ddot{y}} = 1.5\frac{g_h\,\bar{p}_h\,b}{m_0}(0.76+0.24k)\sqrt{\left(\frac{\pi C_{fs}}{\beta}\right)}$$

where

C_{fs} = across wind force spectrum coefficient generalized for a linear mode, (*see* Fig. 10);

β = damping coefficient of the building/structure (*see* Table 39); and

m_0 = the average mass per unit height of the structure, in kg/m.

Typical values of the mode shape power exponent, k are as follows:

a) uniform cantilever, k = 1.5

b) slender framed structure (moment resisting), k = 0.5

c) building with a central core and moment resisting façade, k = 1.0

d) lattice tower decreasing in stiffness with height, or a tower with a large mass at the top, k = 2.3

4.8.4 *Combination of Along Wind and Across Wind Load Effects*

The along wind and across wind loads have to be applied simultaneously on the building/structure during design.

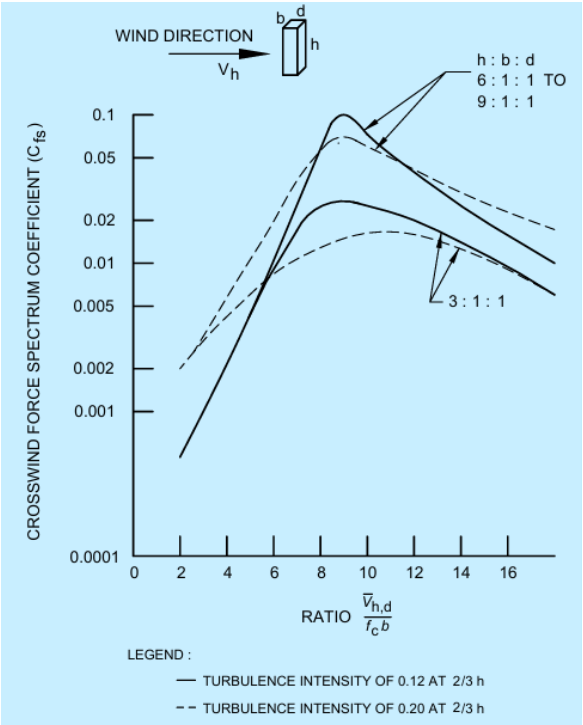

FIG. 10(A) VALUES OF THE CROSS WIND FORCE SPECTRUM COEFFICIENT FOR SQUARE SECTION BUILDINGS

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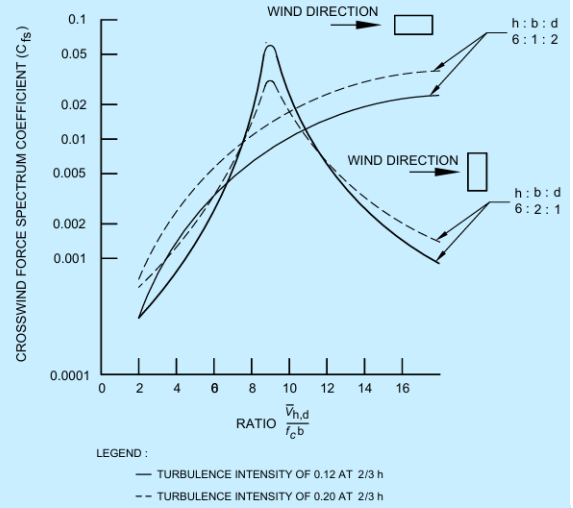

FIG. 10(B) VALUES OF THE CROSS WIND FORCE SPECTRUM COEFFICIENT FOR 2 : 1 AND 1 : 2 RECTANGULAR SECTION BUILDINGS

FIG. 10 WIND FORCE SPECTRUM COEFFICIENTS

5 EARTHQUAKE EFFECTS — GENERAL

For assessment of earthquake loads, determination of seismic zones and earthquake resistant design of buildings, good practice [C-1(6)] shall be referred to. Additionally, it outlines the principles of earthquake-resistant design, desirable and unacceptable features of earthquake-resistant structural system configurations and methods of analysis and treatment of geotechnical aspects for all types of structures.

6 EARTHQUAKE EFFECTS — BUILDINGS

6.1 *Masonry Buildings*

6.1.1 Provisions given in Part C 'Structural Design/Section 4 Masonry' of this NBCS deals with the selection of materials, special aspects related to design and construction of new earthquake-resistant masonry buildings, including masonry buildings using rectangular masonry units with cast in-situ or prefabricated flooring and roofing elements.

6.1.2 Guidelines for earthquake-resistant design of buildings constructed using masonry of low strength and earthen buildings are covered in as per good practice [C-1(7)].

6.2 *Concrete Buildings*

Provisions given in Part C 'Structural Design/ Section 5 Structural Concrete' of this NBCS and good practice [C-1(8)] deal with the selection of materials and special aspects related to design and construction for cast in-situ earthquake-resistant reinforced concrete (RC) buildings.

6.3 *Steel Buildings*

Provisions given in Part C 'Structural Design/Sec 6A Steel' of this NBCS and good practice [C-1(9)] deal with the selection of materials, special aspects related to design and construction for earthquake-resistant steel buildings.

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7 SNOW LOAD

7.1 This clause deals with snow loads on roofs of buildings. Roofs should be designed for the actual load due to snow or for the imposed loads specified in 3, whichever is more severe.

NOTE — Mountainous regions in northern and eastern parts of India are subjected to snow-fall. In northern India, parts of union territory of Jammu and Kashmir (J and K), union territory of Ladakh, Himachal Pradesh, Uttarakhand and in eastern India, parts of Arunachal and Sikkim experience snow-fall of varying depths two to three times in a year.

7.2 Notations

a) μ (Dimensionless) — Nominal values of the shape coefficients, taking into account snow drifts, sliding snow, etc, with subscripts, if necessary;

b) l_i (m) — Horizontal dimension with numerical subscripts, if necessary;

c) h_i (m) — Vertical dimensions with numerical subscripts, if necessary;

d) β_i (degree) — Roof and other surface slope;

e) s_0 (kPa) — Characteristic value of snow load on the ground with a specified annual exceedance probability;

f) s (kPa) — Snow load on roofs other surfaces; and

g) A (m) — Site altitude above sea level.

7.3 Snow Load in Roof(s)

7.3.1 The minimum design snow load on a roof area or any other area above ground which is subjected to snow accumulation is obtained by multiplying the snow load on ground, s_o by the shape coefficient μ, as applicable to the particular roof area considered, expressed as:

$$s = \mu s_o$$

where

s = design snow load on plan area of roof, in Pa;

μ = shape coefficient (*see* 7.4); and

s_o = ground snow load in Pa (1 Pa=1 N/m^2).

NOTE — Ground snow load at any place depends on the critical combination of the maximum depth of undisturbed aggregate cumulative snow fall and its average density. In due course the characteristic snow load on ground for different regions will be included based on studies. Till such time the users of this NBCS are advised to contact either Defence Geoinformatics Research Establishment (DGRE), Chandigarh of Defence Research and Development Organization (DRDO), or India Meteorological Department (IMD), Pune in the absence of any specific information for any location.

7.3.2 *Characteristic Ground Snow Load* (s_0)

The characteristic ground snow load (s_0) at any site is defined based on an annual probability of exceedance of 0.034 and corresponds to a mean return period of 30 years. The value of s_0 (kPa) for any site in snow-bound regions of Indian Himalayas should be obtained using the procedure described in Annex H.

NOTE — Annex H gives the characteristic ground snow load map for union territory of Jammu and Kashmir, union territory of Ladakh, Himachal Pradesh, Uttarakhand and Sikkim resulting from studies conducted by DGRE, Chandigarh.

7.3.3 *Partial Loading Due to Melting, Sliding, Snow Redistribution and Snow Removal*

A loading corresponding to severe imbalances resulting from snow removal, redistribution, sliding, melting, etc (for example, zero snow load on specific parts of the roof) should always be considered. Such considerations are particularly important for structures which are sensitive to unbalance loading (for example, curved roofs, arches, domes, collar beam roofs, continuous beam systems). Water load due to ponding in flat roof as well as valley of pitched roof shall be taken as not less than 0.24 kPa.

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7.3.4 *Ponding Instability*

7.3.4.1 Roofs shall be designed to preclude ponding.

7.3.4.2 For flat roofs (or with a small slope), roof deflections caused by snow loads shall be investigated when determining the likelihood of ponding from rain-on-snow or from snow meltwater. The angle for snow melting is 15º.

7.4 Shape Coefficients

7.4.1 *General Principles*

In perfectly calm weather, falling snow would cover roofs and the ground with a uniform blanket of snow and the design snow load could be considered as a uniformly distributed load. Truly uniform loading conditions, however, are rare and have usually only been observed in areas that are sheltered on all sides by high trees, buildings, etc. In such a case, the shape coefficient would be equal to unity.

In most regions, snow-falls are accompanied or followed by winds. The winds will redistribute the snow and on some roofs especially multilevel roofs, the accumulated drift load may reach a multiple of the ground load. Roofs which are sheltered by other buildings, vegetation, etc, may collect more snow load than the ground level. The phenomenon is of the same nature as that illustrated for multilevel roofs in 7.4.2.4.

So far, sufficient data are not available to determine the shape coefficient on a statistical basis. Therefore, a nominal value is given. A representative sample of roofs is shown in 7.4.2. However, in special cases such as strip loading, cleaning of the roof periodically by deliberate heating of the roof, etc, have to be treated separately. However, no reduction in load of snow on roof having underside insulation and/or internal heating system shall be done.

The distribution of snow in the direction parallel to the caves is assumed to be uniform.

7.4.2 *Shape Coefficients for Selected Types of Roofs*

7.4.2.1 *Monopitched and simple pitched roofs*

	Simple Flat and Monopitch Roofs	Simple Pitched Roofs (Positive Roof Slope)*
$0° < \beta \leq 15°$		$\mu_2 = \mu_1 = 0.80$

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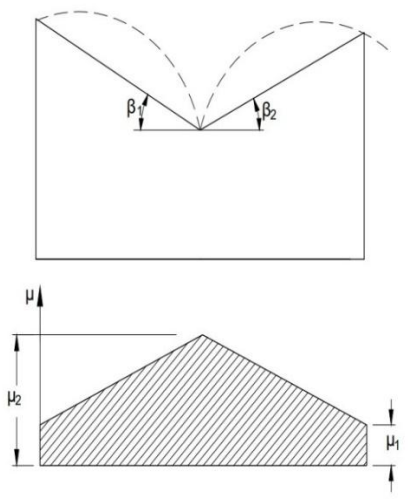
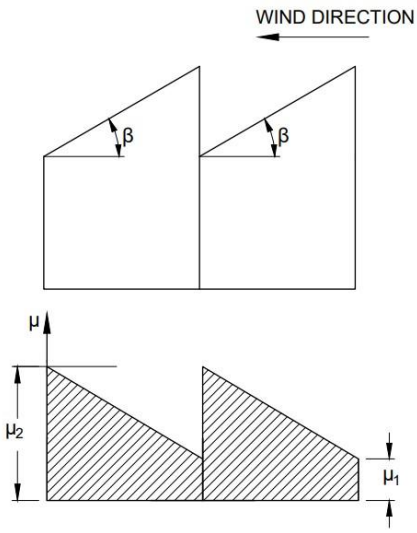
$15° < \beta \leq 30°$	$\mu_1 = 0.80$	$\mu_2 = 0.80 + 0.40\left(\frac{\beta - 15}{15}\right)$ $\mu_1 = 0.80$
$30° < \beta \leq 60°$	$\mu_1 = 0.80\left(\frac{60 - \beta}{30}\right)$	$\mu_2 = 1.20\left(\frac{60 - \beta}{30}\right)$ $\mu_1 = 0.80\left(\frac{60 - \beta}{30}\right)$
$\beta > 60°$	$\mu_1 = 0$	$\mu_2 = \mu_1 = 0$
*For asymmetrical simple pitched roofs, each side of the roof shall be treated as one half of corresponding symmetrical roofs.		

NOTES

1 For monopitch roof, wind direction will not affect the value of μ.

2 The values of $\mu 1$ and $\mu 2$ shall be reversed in case the wind direction changes from left to right for simple pitched roof.

7.4.2.2 *Multiple pitched roofs*

	Simple on Multiple Pitched Roofs (Negative Roof Slope)	Two – Span or Multispan Roofs
		
$0° < \beta \leq 30°$	$\mu_2 = 0.80\left(\frac{30 + \beta}{30}\right)$ $\mu_1 = 0.80$	$\mu_2 = 0.80\left(\frac{30 + \beta}{30}\right)$ $\mu_1 = 0.80$
$30° < \beta \leq 60°$	$\mu_1 = 1.60$ $\mu_1 = 0.80\left(\frac{60 - \beta}{30}\right)$	$\mu_2 = 1.60$ $\mu_1 = 0.80\left(\frac{60 - \beta}{30}\right)$

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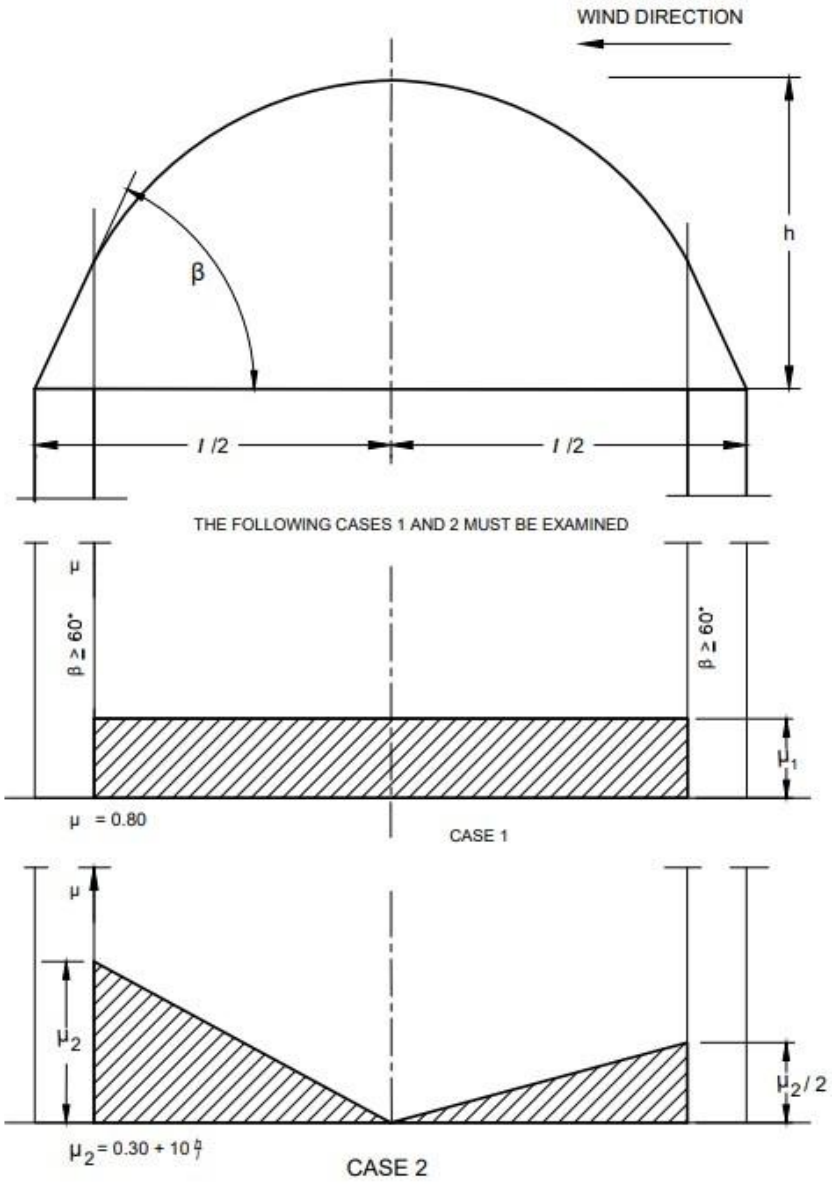
$\beta > 60°$	$\mu_1 = 1.60$ $\mu_1 = 0$	$\mu_2 = 1.60$ $\mu_1 = 0$

7.4.2.3 *Simple curved roofs*



Restrictions:
$\mu_2 \leq 2.3$
$\mu = 0 \; if \; \beta \geq 60^o$

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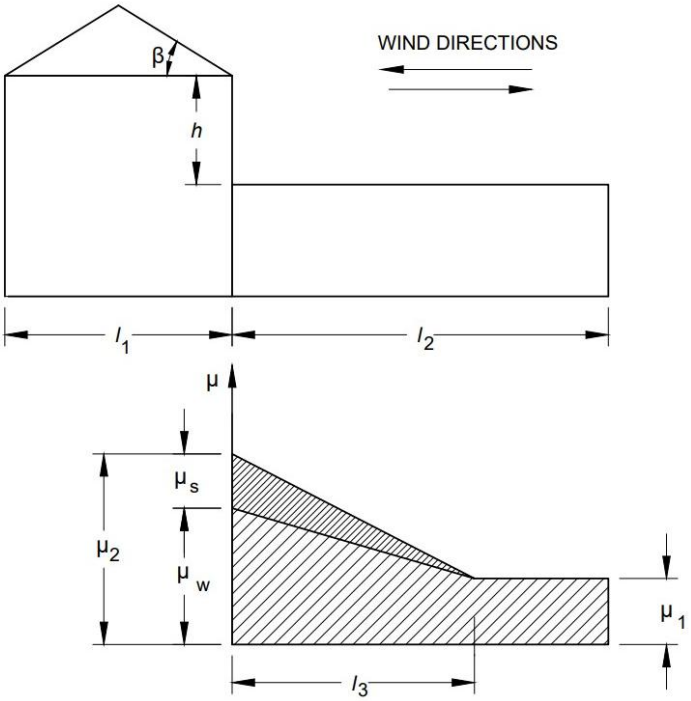
7.4.2.4 *Multilevel roofs*[1)]



$$\mu_1 = 0.80$$

$$\mu_2 = \mu_s + \mu_w$$

where

μ_s = due to sliding; and

μ_w = due to wind.

$l_3 = 2h$ [2)] but is restricted as follows:

$$5\ m \leq l_3 \leq 15\ m$$

$$\mu_w = \frac{l_1 + l_2}{2h} \leq \frac{kh}{s_0} \text{ with the restriction } 0.80 \leq \mu_w \leq 4.0$$

where

h is in metres

s_0 is in kPa (kN/m^2)

$k = 2\,kN/m^2$

NOTES

1 A more extensive formula for μ_w is described in Annex J.

2 If $l_2 < l_3$, the coefficient μ is determined by interpolation between μ_1 and μ_2.

If $\beta > 15^\circ$: μ_s is determined from an additional load amounting to 50 percent of the maximum total load on the adjacent slope of the upper roof and is distributed linearly as shown in the figure. The load on the upper roof is calculated according to **7.4.2.1** or **7.4.2.2**.

If $\beta \leq 15^\circ$: $\mu_s = 0$

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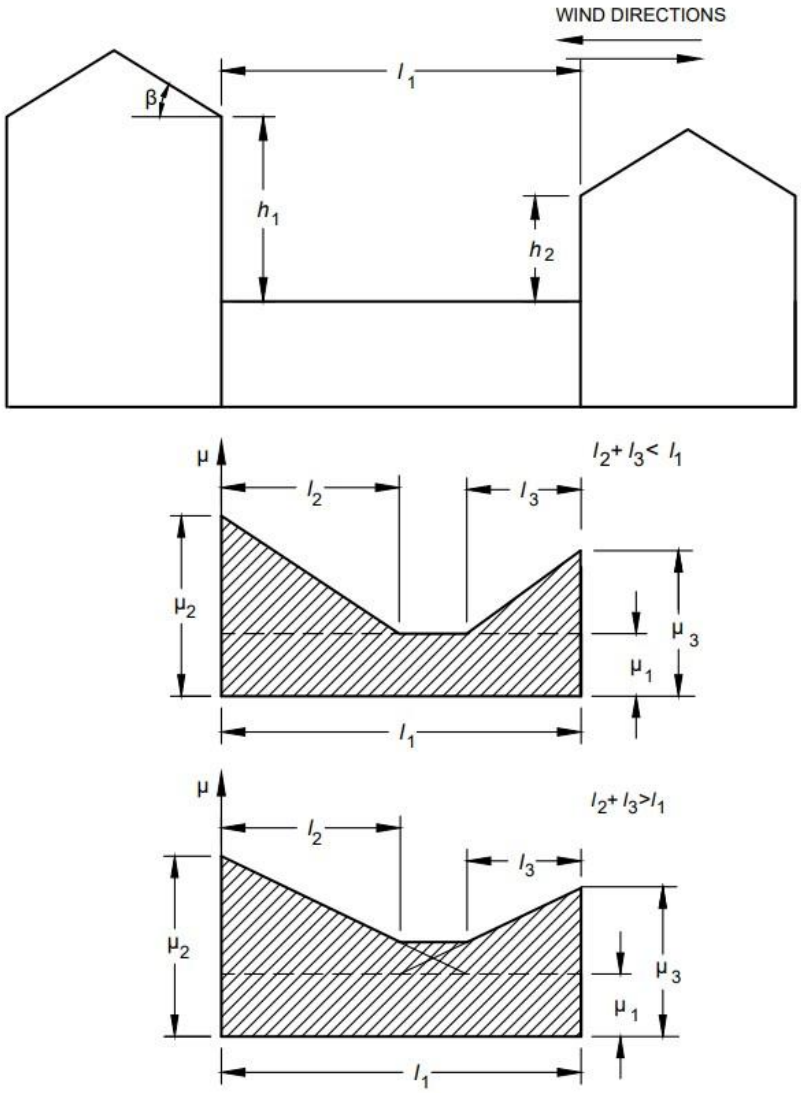
7.4.2.5 *Complex multilevel roofs*



$l_2 = 2h_1$; $l_3 = 2h_2$; $\mu_1 = 0.80$;

Restrictions:

$5\ m \leq l_2 \leq 15\ m$

$5\ m \leq l_3 \leq 15\ m$

μ_2 and $\mu_3 = \mu_s + \mu_w$, are calculated according to 7.4.2.1, 7.4.2.2 and 7.4.2.4

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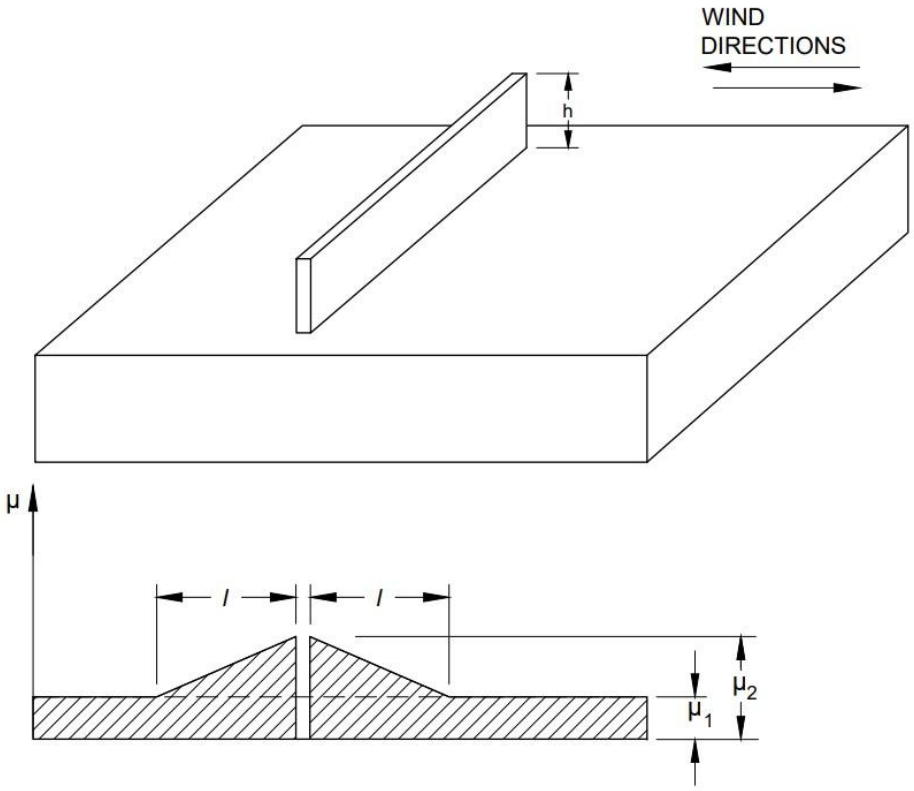
7.4.2.6 *Roofs with local projections and obstructions*



$$\mu_2 = \frac{kh}{s_0}$$

$$\mu_1 = 0.80$$

where

h is in metres

s_0 is in kPa ($\mathrm{kN/m^2}$)

$k = 2\ \mathrm{kN/m^2}$

$l = 2\ h$

Restriction:

$0.80 \leq \mu_2 \leq 2.0$

$5\ m \leq l \leq 15\ m$

7.4.3 *Shape Coefficients in Areas Exposed to Wind*

The shape coefficients given in **7.4.2** and Annex J may be reduced by 15 percent, provided the designer has demonstrated that the following conditions are fulfilled:

a) The building is located in an exposed location such as open level terrain with only scattered buildings, trees or other obstructions so that the roof is exposed to the winds on all sides and is not likely to become shielded in the future by obstructions higher than the roof within a distance from the building equal to ten times the height of the obstruction above the roof level; and

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b) The roof does not have any significant projections such as parapet walls which may prevent snow from being blown off the roof.

NOTE — In some areas, winter climate may not be of such a nature as to produce a significant reduction of roof loads from the snow load on the ground. This typically occurs in winter calm valleys in mountainous regions, where layers of snow may accumulate on roofs without any appreciable removal by wind, as well as in relatively higher temperature areas where the maximum snow load may result from a single snowstorm, occasionally without significant wind removal. In such conditions, the determination of shape coefficients should be based on local experience, with due consideration given to the likelihood of wind drifting and sliding.

8 SPECIAL LOADS

8.1 This clause gives guidance on loads and load effects due to temperature changes, soil and hydrostatic pressures, internally generating stresses (due to creep, shrinkage, differential settlement, etc), accidental loads, etc, to be considered in the design of buildings as appropriate. This clause also includes guidance on load combinations. The nature of loads to be considered for a particular situation is to be based on engineering judgment (*See also* **3.6**)

8.2 Temperature Effects

8.2.1 Expansion and contraction due to changes in temperature of the materials of a structure shall be considered in design. Provision shall be made either to relieve the stress by the provision of expansion/contraction joints in accordance with good practice [C-1(10)] or design the structure to carry additional stresses due to temperature effects as appropriate to the problem.

8.2.1.1 The temperature range varies for different regions and under different diurnal and seasonal conditions. The absolute maximum and minimum temperature which may be expected in different localities in the country are indicated in good practice [C-1(11)]. These figures may be used for guidance in assessing the maximum variations of temperature.

8.2.1.2 The temperatures indicated are the air temperatures in the shade. The range of variation in temperature of the building materials may be appreciably greater or less than the variation of air temperature and is influenced by the condition of exposure and the rate at which the materials composing the structure absorb or radiate heat. This difference in temperature variations of the material and air should be given due consideration.

8.2.1.3 The structural analysis must take account of changes of the mean (through the section) temperature in relation to the initial (st) and the temperature gradient through the section:

a) it should be borne in mind that the changes of mean temperature in relation to the initial are liable to differ as between one structural element and another in buildings or structures, as for example, between the external walls and the internal elements of a building. The distribution of temperature through section of single-leaf structural elements may be assumed linear for the purpose of analysis; and

b) the effect of mean temperature changes t_1 and t_2 and the temperature gradients v_1 and v_2 in the hot and cold seasons for single-leaf structural elements shall be evaluated on the basis of analytical principles.

NOTES

1 For portions of the structure below ground level, the variation of temperature is generally insignificant. However, during the period of construction, when the portions of the structure are exposed to weather elements, adequate provision should be made to encounter adverse effects, if any.

2 If it can be shown by engineering principles, or if it is known from experience, that neglect of some or all the effects of temperature do not affect the structural safety and serviceability, they need not be considered in design.

8.3 Hydrostatic and Soil Pressure

8.3.1 In the design of structures or parts of structures below ground level such as retaining walls and other walls in basement floors, the pressure exerted by the soil or water or both shall be duly accounted for on the basis of established theories. Due allowance shall be made for possible surcharge from stationary or moving loads. When a portion or whole of the soil is below the free water surface, the lateral earth pressure shall be evaluated for weight of soil diminished by buoyancy and the full hydrostatic pressure.

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8.3.1.1 All foundation slabs and other footings subjected to water pressure shall be designed to resist a uniformly distributed uplift equal to the full hydrostatic pressure. Checking of overturning of foundation under submerged condition shall be done considering buoyant weight of foundation.

8.3.2 While determining the lateral soil pressure on column like structural members such as pillars which rest in sloping soils, the width of the member shall be taken as follows (*see* Fig. 11).

Sl No. (1)	*Actual Width of Member* (2)	*Ratio of Effective Width to Actual Width* (3)
i)	Less than 0.5 m	3.0
ii)	Beyond 0.5 m and up to 1 m	3.0 to 2.0
iii)	Beyond 1 m	2.0

The relieving pressure of soil in front of the structural member concerned may generally not be taken into account.

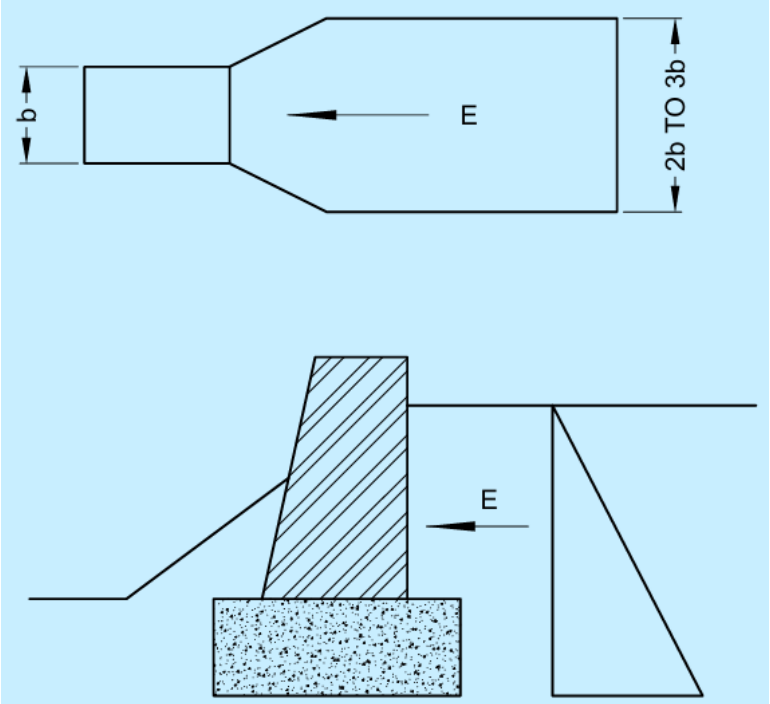

FIG. 11 SKETCH SHOWING EFFECTIVE WIDTH OF PILLAR FOR CALCULATING SOIL PRESSURE

8.3.3 Safe-guarding of structures and structural members against overturning and horizontal sliding shall be verified. Imposed loads having favourable effect shall be disregarded for the purpose. Due consideration shall be given to the possibility of soil being permanently or temporarily removed.

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8.4 Fatigue

8.4.1 *General*

Fatigue cracks are usually initiated at points of high stress concentration. These stress concentrations may be caused by or associated with holes (such as bolt or rivet holes in steel structures), welds including stray or fusions in steel structures, defects in materials and local and general changes in geometry of members. The cracks usually propagate if the loading is cyclic and repetitive.

Where there is such cyclic and repetitive loading, sudden changes of shape of a member or part of a member, especially in regions of tensile stress and/or local secondary bending, shall be avoided. Suitable steps shall be taken to avoid critical vibrations due to wind and other causes.

8.4.2 Where necessary, permissible stresses shall be reduced to allow for the effects of fatigue. Allowance for fatigue shall be made for combinations of stresses due to dead load and imposed load. Stresses due to wind and earthquakes may be ignored, when fatigue is being considered, unless otherwise specified in relevant Codes of practice.

Each element of the structure shall be designed for the number of stress cycles of each magnitude to which it is estimated that the element is liable to be subjected during the expected life of the structure. The number of cycles of each magnitude shall be estimated in the light of available date regarding the probable frequency of occurrence of each type of loading.

NOTE — Apart from the general observations made herein, the section is unable to provide any precise guidance in estimating the probabilistic behaviour and response of structures of various types arising out of repetitive loading approaching fatigue conditions in structural members, joints, materials, etc.

8.5 Structural Safety During Construction

All loads required to be carried by the structures or any part of it due to storage or positioning of construction materials and erection equipment including all loads due to operation of such equipment, shall be considered as erection loads. Proper provision shall be made, including temporary bracings, to take care of all stresses due to erection loads. The conjunction with the temporary bracings shall be capable of sustaining these erection loads, without exceeding the permissible stresses specified in respective Codes of practice. Dead load, wind load and such parts of imposed load, as would be imposed on the structure during the period of erection, shall be taken as acting together with erection loads.

8.6 Accidental Loads

The occurrence of which, with a significant value, is unlikely on a given structure over the period of time under consideration and also in most cases, is of short duration. The occurrence of an accidental load could, in many cases, be expected to cause severe consequences, unless special measures are taken.

The accidental loads arising out of human action include the following:

a) Impacts and collisions;

b) Explosions; and

c) Fire.

Characteristic of the above stated loads are that they are not a consequence of normal use and that they are undesired and that extensive effects are made to avoid them. As a result, the probability of occurrence of an accidental load is small whereas the consequences may be severe.

The causes of accidental loads may be:

a) inadequate safety of equipment (due to poor design or poor maintenance); and

b) wrong operation (due to insufficient teaching or training, indisposition, negligence or unfavourable external circumstances).

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In most cases, accidental loads only develop under a combination of several unfavourable occurrences. In practical applications, it may be necessary to neglect the most unlikely loads. The probability of occurrence of accidental loads, which are neglected, may differ for different consequences of a possible failure. A data base for a detailed calculation of the probability will seldom be available.

8.6.1 *Impact and Collisions*

8.6.1.1 *General*

During an impact, the kinetic impact energy has to be absorbed by the vehicle hitting the structure and by the structure itself. In an accurate analysis, the probability of occurrence of an impact with a certain energy object hitting the structure and the structure itself at the actual place must be considered. Impact energies for dropped object should be based on the actual loading capacity and lifting height.

Common sources of impact are:

a) vehicles;

b) dropped objects from cranes, fork lifts, etc;

c) cranes out of control, crane failures; and

d) flying fragments.

The codal requirements regarding impact from vehicles and cranes are given in **8.6.1.2** and **8.6.1.3**.

8.6.1.2 The requirements for loads as a result of collision between vehicle and structural or non-structural elements shall be as per **8.6.1.2.1** and **8.6.1.2.2**.

8.6.1.2.1 *Collisions between vehicles and structural elements*

In road traffic, the requirement that a structure shall be able to resist collision may be assumed to be fulfilled, if it is demonstrated that the structural element is able to stop a fictitious vehicle, as described below. It is assumed that the vehicle strikes the structural element at a height of 1.2 m in any possible direction and at a speed of 10 m/s (36 km/h).

The fictitious vehicle shall be considered to consist of two masses m_1 and m_2 which during compression of the vehicle, produce an impact force increasing uniformly from zero, corresponding to the rigidities C_1 and C_2. It is assumed that the mass m_1 is broken completely before the breaking of mass m_2 begins.

The following numerical values should be used:

a) m_1 = 400 kg, C_1 = 10 000 kN/m, the vehicle is compressed; and

b) m_2 = 12 000 kg, C_2 = 300 kN/m, the vehicle is compressed.

NOTE — The described fictitious collision corresponds in the case of a non-elastic structural element to a maximum static force of 630 kN for the mass m_1 and 600 kN for the mass m_2 irrespective of the elasticity, it will therefore be on the safe side to assume the static force to be 630 kN.

In addition, breaking of the mass m_1 will result in an impact wave, the effect of which will depend, to a great extent, on the kind of structural element concerned. Consequently, it will not always be sufficient to design for the static force.

8.6.1.2.1.1 *For structural elements in stilts and car parking levels*

For structural elements in stilts and car parking levels, where driveways and car parking are provided in the buildings which have limited vehicle speeds, the design shall take care of the following considerations:

a) Speed of the vehicle to be 20 km/h maximum (since within a building and inside the car parks, speed of the vehicle may not generally be more than 20 km/h); and

b) Impact to be considered at a height of 450 mm to 600 mm.

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Structural elements such as columns, RC walls or vertical load resisting elements shall be designed for loads due to the above unless protected by separate resisting system.

8.6.1.2.2 *Collision between vehicles and non-structural elements:*

Non-structural elements, such as vehicle barriers for car parks shall be designed to resist the impact and the momentum as outlined below:

a) The horizontal force F (in kN), normal to and uniformly distributed over any length of 1.5 m of a barrier for a car park, required to withstand the impact of a vehicle shall be as given below:

$$F = \frac{0.5mv^2}{\delta_c + \delta_b}$$

where

m = gross mass of the vehicle, in kg;

v = velocity of the vehicle normal to the barrier, in m/s;

δ_c = deformation of the vehicle, in mm; and

δ_b = deflection of the barrier, in mm.

b) Where the car park has been designed on the basis that the gross mass of the vehicles using it will not exceed 2500 kg, the following values shall be used to determine the force F:

m = 1500 kg;
v = 4.5 m/s;
δ_c = 100 mm;

For a rigid barrier, for which δ_b may be taken as zero, the force, F, appropriate to vehicles up to 2500 kg gross mass, shall be taken as 150 kN.

c) Where the car park has been designed for vehicles whose gross mass exceeds 2500 kg, the following values shall be used to determine the force F:

m = actual mass of the vehicle for which the car park is designed, in kg;

v = 4.5 m/s;

δ_c = 100 mm;

d) The force determined as in (b) or (c) may be considered to act at bumper height. In the case of car parks intended for motor cars whose gross mass does not exceed 2500 kg, this height may be taken as 375 mm above the floor level.

e) Barriers to access ramps of car parks have to withstand one-half of the force determined in (b) or (c) acting at a height of 610 mm above the ramp.

Opposite to the ends of straight ramps intended for downward travel, which exceed 20 m in length, the barrier shall be designed to withstand twice the force determined in (b) or (c) acting at a height of 610 mm above the ramp.

NOTE – The mass of 1500 kg is taken as being more representative of the vehicle population than the extreme value of 2500 kg.

8.6.1.3 *Safety railings*

With regard to safety, railings put up to protect structures against collision due to road traffic, it should be shown that the railings are able to resist the impact as described in **8.6.1.2**.

NOTE — When a vehicle collides with safety railings, the kinetic energy of the vehicle will be absorbed partly by the deformation of the railings and partly by the deformation of the vehicle. The part of the kinetic energy which the railings should be able to absorb without breaking down may be determined on the basis of the assumed rigidity of the vehicle during compression.

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8.6.1.4 *Crane impact load on buffer stop*

The basic horizontal load P_y (tonne), acting along the crane track produced by impact of the crane on the buffer stop, is calculated by the following formula:

$$P_y = MV^2/F$$

where

V = speed at which the crane is traveling at the moment of impact (assumed equal to half the nominal value) (m/s).

F = maximum shortening of the buffer, assumed equal to 0.1 m for light duty, mediuL-duty and heavy-duty cranes with flexible load suspension and loading capacity not exceeding 50 t and 0.2 m in every other cranes.

M = reduced crane mass, (t.s^2/m); and is obtained by the formula:

$$M = \frac{1}{g}\left[\frac{P_h}{2} + (P_t + kQ)\frac{L_k - l}{L_k}\right]$$

where

P_h = crane bridge weight (t);

P_t = crab bridge weight (t);

k = coefficient, assumed equal to zero for cranes with flexible load suspension and to one for cranes with rigid suspension;

Q = crane loading capacity (t);

L_k = crane span (m); and

ℓ = nearness of crab (m); and

g = acceleration due to gravity (9.81 m/s^2).

8.6.2 *Explosions*

8.6.2.1 *General*

Explosions may cause impulsive loading on a structure. The following types of explosions are particularly relevant:

a) Internal gas explosions which may be caused by leakage of gas piping (including piping outside the room), evaporation from volatile liquids or unintentional evaporation from surface material (for example, fire);

b) Internal dust explosions;

c) Boiler failure;

d) External gas cloud explosions; and

e) External explosions of high explosives (TNT, dynamite).

The codal requirement regarding internal gas explosions is given in **8.6.2.2**.

8.6.2.2 *Explosion effect in closed rooms*

Gas explosion may be caused, for example by leaks in gas pipes (inclusive of pipes outside for room), evaporation from volatile liquids or unintentional evaporation of gas from wall sheathings (for example, caused by fire).

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NOTES

1 The effect of explosions depends on the exploding medium, the concentration of the explosion, the shape of the room, possibilities of ventilation of the explosion and the ductility and dynamic properties of the structure. In rooms with little possibility for relief of the pressure from the explosion, very large pressures may occur.

2 Internal over pressure from an internal gas explosion in rooms of sizes comparable to residential rooms and with ventilation areas consisting of window glass breaking at a pressure of 4 kN/m^2 (3 mm to 4 mm machine made glass) may be calculated from the following method:

a) The over pressure is assumed to depend on a factor A/V, where A is the total windows area, in m^2 and V is the volume, in m^3, of the room considered;

b) The internal pressure is assumed to act simultaneously upon all walls and floors in one closed room; and

c) The action q_o may be taken as static action.

3 If account is taken of the time curve of the action, the schematic correspondence between pressure and time is assumed (*see* Fig. 12), where t_1 is the time from the start of combustion until maximum pressure is reached and t_2 is the time from maximum pressure to the end of combustion. For t_1 and t_2, the most unfavourable values should be chosen in relation to the dynamic properties of the structures. However, the values should be chosen within the intervals as given in Fig. 13.

Figure 12 is based on tests with gas explosions in room corresponding to ordinary residential flats and should, therefore, not be applied to considerably different conditions. The figure corresponds to an explosion caused by town gas and it might, therefore, be somewhat on the safe side in rooms where there is only the possibility of gases with a lower rate of combustion.

The pressure may be applied solely in one room or in more rooms at the same time. In the latter case, all rooms are incorporated in the volume V. Only windows or other similarly weak and light weight structural elements may be taken to be ventilation areas even though certain limited structural parts break at pressures less than q_o.

Figure 12 is given purely as guide and probability of occurrence of an explosion should be checked in each case using appropriate values.

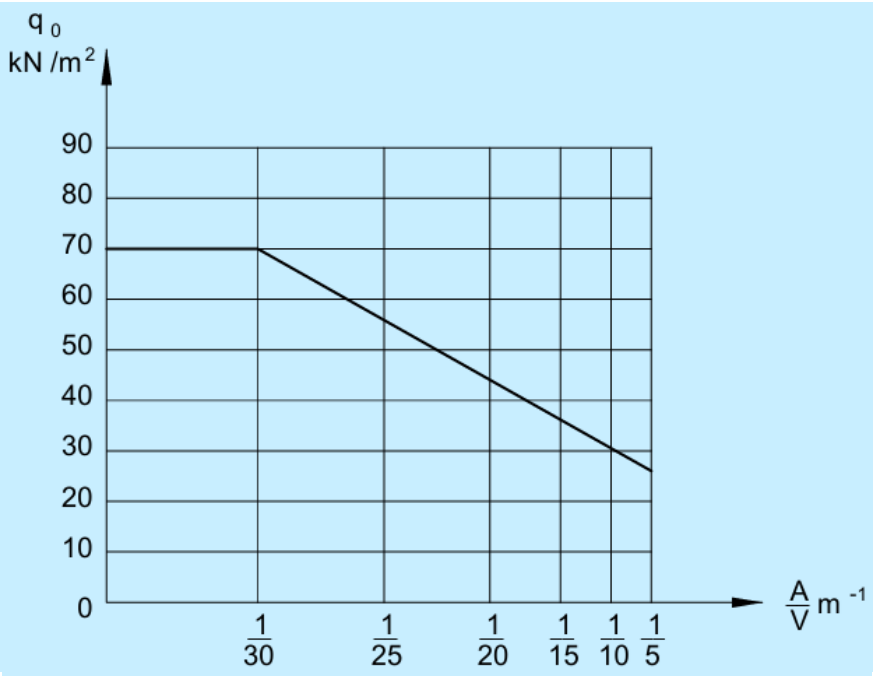

FIG. 12 SKETCH SHOWING RELATION BETWEEN PRESSURE AND TIME

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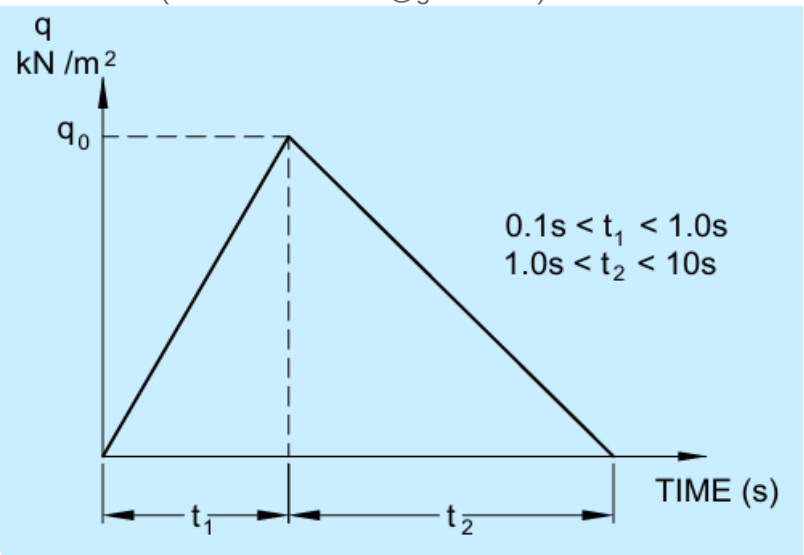

FIG. 13 SKETCH SHOWING TIME INTERVAL AND PRESSURE

8.6.3 *Vertical Load on Air Raid Shelters*

8.6.3.1 *Characteristic values*

As regards buildings in which the individual floors are acted upon by a total characteristic imposed action of up to 5.0 kN/m^2, vertical actions on air raid shelters generally located below ground level, for example, basement, etc, should be considered to have the following characteristic values:

a)	Buildings with up to 2 storeys	:	28 kN/m^2
b)	Buildings with 3-4 storeys	:	34 kN/m^2
c)	Buildings with more than 4 storeys	:	41 kN/m^2
d)	Buildings of particularly stable construction irrespective of the number of storeys	:	28 kN/m^2

In the case of buildings with floors that are acted upon by a characteristic imposed action larger than 5.0 kN/m^2, the above values should be increased by the difference between the average imposed action on all storeys above the one concerned and 5.0 kN/m^2.

NOTES

1 By storeys it is understood, every utilizable storey above the shelter.

2 By buildings of a particular stable construction, it is understood, buildings in which the load- bearing structures are made from reinforced *in-situ* concrete.

8.6.4 *Fire*

8.6.4.1 *General*

Possible extraordinary loads during a fire may be considered as accidental actions. Examples are loads from people along escape routes and loads on another structure from structure failing because of a fire.

8.6.4.2 *Thermal effects during fire*

The thermal effect during fire may be determined from one of the following methods:

a) Time-temperature curve and the required fire resistance (in min); and

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b) An energy balance method.

If the thermal effect during fire is determined from an energy balance method, the fire load is taken to be:

$$q = 12\ t_b$$

where

q = fire action (KJ/m^2 floor); and

t_b = required fire resistance (in min) {*see* [C-1(12)]}.

NOTE — The fire action is defined as the total quantity of heat produced by complete combustion of all combustible material in the fire compartment, inclusive of stored goods and equipment together with building structures and building materials.

8.7 Vibrations

For general details on loads due to vibrations, reference may be made to Annex K.

8.8 Blast Loads

For provisions related to blast loads, *see* Annex L.

8.9 Other Loads

Other loads not included in this Section, such as special loads due to technical process, moisture and shrinkage effects, etc, should be taken into account where stipulated by building design codes or established in accordance with the performance requirement of the structure.

8.10 Landslide Control Measures

Mountainous and hilly terrains are prone to slope instability arising from geological, hydrological and seismic factors, further aggravated by unplanned developmental activities. The following good practices address the specific aspects towards hazard assessment, design and construction:

Sl No.	*Aspects*	*Accepted Standard*(*s*)
(1)	(2)	(3)
i)	Guidelines for preparation of landslide hazard zonation maps	[C-1(13)]
ii)	Preparation of landslide risk assessment maps, combining hazard probability with potential damage through a risk assessment matrix	[C-1(13)]
iii)	Site-specific investigation and stability analysis, including geological mapping, subsurface exploration, laboratory testing and numerical modelling	[C-1(13)]
iv)	Landslide control measures with emphasis on drainage management, slope modification, reinforcement, bio-engineering and protective works	[C-1(13)]
v)	Design and construction of different types of retaining walls	[C-1(13)]
vi)	Micropiles for slope stabilization	[C-1(13)]
vii)	Surface water management in hilly areas	[C-1(13)]
viii)	Siting, design and selection of materials for residential buildings in hilly areas	[C-1(13)]

8.11 Cyclone Resistant Buildings

Cyclone-prone regions are vulnerable to high wind speeds, wind-borne debris and storm surges, which can cause extensive damage to buildings, particularly low-rise and kutcha houses. The following good practices address aspects of loads, design, construction and assessment for improving resilience:

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Sl No.	*Aspect*	*Accepted Standard*
(1)	(2)	(3)
i)	Guidelines for improving the cyclonic resistance of low-rise houses and other buildings	[C-1(14)]
ii)	Guidelines for survey of housing and building typology in cyclone prone areas for assessment of vulnerability of regions and post-cyclone damage estimation	[C-1(14)]
iii)	Guidelines for design and construction of cyclone shelters	[C-1(14)]
iv)	Method for determination of performance relating to safety of external building envelope impacted by wind-borne debris	[C-1(14)]
v)	Guidelines for improving the cyclonic resistance capacity of kutcha houses	[C-1(14)]

9 LOAD COMBINATIONS

9.1 General

A judicious combination of the loads keeping in view the probability of:

a) their acting together; and

b) their disposition in relation to other loads and severity of stresses or deformations caused by the combinations of the various loads is necessary to ensure the required safety and economy in the design of a structure.

9.2 Load Combinations

Keeping the aspect specified in **9.1**, the various loads should, therefore, be combined in accordance with the stipulation in the relevant design Codes. In the absence of such recommendations, the following loading combinations, whichever combination produces the most unfavourable effect in the building, foundation or structural member concerned may be adopted (as a general guidance). It should also be recognized in load combinations that the simultaneous occurrence of maximum values of wind, earthquake, imposed and snow loads is not likely.

a) *DL*;

b) *DL* and *IL*;

c) *DL* and W*L*;

d) *DL* and E*L*;

e) *DL* and T*L*;

f) *DL*, *IL* and *WL*;

g) *DL*, *IL* and *EL*;

h) *DL*, *IL* and *TL*

j) *DL*, W*L* and *TL*;

k) *DL*, *EL* and *TL*;

m) *DL*, *IL*, *WL* and *TL*; and

n) *DL*, *IL*, *EL* and *TL*

(*DL* = dead load, *IL* = imposed load, *WL* = wind load, *EL* = earthquake load and *TL* = temperature load).

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NOTES

1 When snow load is present on roofs, replace imposed load by snow load for the purpose of above load combinations.

2 The relevant design codes shall be followed for permissible stresses when the structure is designed by working stress method and for partial safety factors when the structure is designed by limit state design method for each of the above load combinations.

3 Whenever imposed load (*IL*) is combined with earthquake load (*EL*), the appropriate part of imposed load as specified in **5** should be used, both for evaluating earthquake effect and also for combined load effects used in such combination.

4 For the purpose of stability of the structure as a whole against overturning, the restoring moment shall be not less than 1.2 times the maximum overturning moment due to dead load plus 1.4 times the maximum overturning moment due to imposed loads. In cases where dead load provides the restoring moment, only 0.9 times the dead load shall be considered. The restoring moments due to imposed loads shall be ignored. In case of high water table, the effects of buoyancy have to be suitably taken into consideration.

5 In case of high water able, the factor of safety of 1.2 against uplift alone shall be provided.

6 The structure shall have a factor against sliding of not less than 1.4 under the most adverse combination of the applied loads/forces. In this case, only 0.9 times the dead load shall be taken into account.

7 Where the bearing pressure on soil due to wind alone is less than 25 percent of that due to dead load and imposed load, it may be neglected in design where this exceeds 25 percent, foundation may be so proportioned that the pressure due to combined effect of dead load, imposed load and wind load does not exceed the allowable bearing pressure by more than 25 percent. When earthquake effect is included, the permissible increase in allowable bearing pressure in the soil shall be in accordance with **5**.

8 Reduced imposed load specified in 3 for the design of supporting structures should not be applied in combination with earthquake forces.

9 Other loads and accidental load combination not included should be dealt with appropriately.

10 Crane load combinations are covered in **3.6.4**.

10 MULTI-HAZARD RISK IN VARIOUS DISTRICTS OF INDIA

10.1 Multi-hazard Risk Concept

The commonly encountered hazards are:

a) Earthquake;

b) Cyclone;

c) Wind storm;

d) Floods;

e) Landslides;

f) Liquefaction of soils;

g) Extreme winds;

h) Cloud bursts; and

j) Failure of slopes.

A study of the earthquake, wind/cyclone and flood hazard maps of India indicate that there are several areas in the country which run the risk of being affected by more than one of these hazards.

Further there may be instances where one hazard may cause occurrence or accentuation of another hazard such as landslides may be triggered/accelerated by earthquakes and wind storms and floods by the cyclones.

It is important to study and examine the possibility of occurrence of multiple hazards, as applicable to an area. However, it is not economically viable to design all the structures for multiple hazards. The special structures such as nuclear power plants and life line structures such as hospitals and emergency rescue shelters may be designed for multiple hazards. For such special structures, site specific data have to be collected and the design be carried out based on the accepted levels of risk. The factors that have to be considered in determining this risk are:

a) The severity of the hazard characterized by M.M. (or M.S.K.) intensity in the case of earthquake; the duration and velocity of wind in the storms; and unprotected or protected situation of flood prone areas; and

b) The frequency of occurrence of the severe hazards.

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Till such time that risk evaluation procedures are formalized, the special structures may be designed for multiple hazards using the historical data, that can be obtained for a given site and the available code for loads already covered. The designer may have to consider the loads due to any one of the hazards individually or in combination as appropriate.

10.2 Multi-hazard Prone Areas

10.2.1 At a site, more than one hazard may be prominent. In general, the natural hazards are not considered to act together, but need to be considered one at a time. Also, the proportion of combining the man-made hazards with natural hazards also are specified in the NBC. The NBC provides a minimum set of combinations of loads that need to be considered in the design of structures along with the corresponding partial safety factor of the loads in each of the said combinations.

10.2.2 The criteria adopted for identifying multi hazard prone areas may be as follows:

a) *Earthquake and flood risk prone* — Districts which have seismic zone of intensity 7 or more and also flood prone unprotected or protected area. Earthquake and flood can occur separately or simultaneously.

b) *Cyclone and flood risk prone* — Districts which have cyclone and flood prone areas. Here floods can occur separately from cyclones, but simultaneous also along with possibility of storm surge too.

c) *Earthquake, cyclone and flood risk prone* — Districts which have earthquake zone of intensity 7 or more, cyclone prone as well as flood prone (protected or unprotected) areas. Here the three hazards can occur separately and also simultaneously as in (a) and (b) above but earthquake and cyclone will be assumed to occur separately only.

d) *Earthquake and cyclone risk prone* — Districts which have earthquake zone of intensity 7 or more and prone to cyclone hazard too. The two will be assumed to occur separately.

Based on the approach given above, the districts with multi-hazard risk are given in Annex M.

10.3 Use of the List of the District with Multi-Hazard Risk

The list provides some ready information for use of the authorities involved in the task of disaster mitigation, preparedness and preventive action. This information gives the districts which are prone to high risk for more than one hazard. This information will be useful in establishing the need for developing housing design to resist such multi-hazard situation.

11 DOCUMENTATION

11.1 For the successful planning, design and construction of buildings, adequate documentation covering structural design sufficiency, sub-surface investigations, supervision, etc may be required. In particular, compliance with the structural safety requirements of this NBCS shall be demonstrated through appropriate documentation, including the preparation of a Structural Design Basis Report (SDBR), which establishes the basis and assumptions adopted in the structural design.

11.2 Structural Design Basis Report (SDBR)

The Structural design basis report (SDBR) is a document that establishes the basis and assumptions adopted for the structural design of high rise and special buildings, demonstrating that the structure has been designed to ensure safety against various loads, actions and effects, including those arising from natural hazards such as earthquakes, landslides, cyclones and floods, in accordance with the provisions of this NBCS, particularly Part C ‘Structural Design’ and other relevant Indian Standards.

The preparation and submission of an SDBR provides transparency and traceability of design decisions and enables verification that the structural system, materials, loading criteria and analysis methods adopted are consistent with the applicable provisions. It also serves as a reference document for reviewing authorities and stakeholders and ensures that the registered engineer/structural engineer maintains adequate documentation to substantiate the structural design.

The SDBR (Annex N) sets out the basis for designing the building and consists of the following parts:

a) Part 1 — General information/Data;

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b) Part 2 — Load-bearing masonry buildings;

c) Part 3 — Reinforced concrete buildings; and

d) Part 4 — Steel and steel–concrete composite buildings.

For demonstrating compliance with Part C ‘Structural Design’ of this NBCS and the relevant Indian Standards referenced therein, the registered engineer/structural engineer shall submit an SDBR for structures of different complexities along with the drawings and other documents required to be submitted with the application.

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ANNEX A

[*Foreword*, *Clause* 2.2]

DEAD LOADS — UNIT WEIGHTS OF CONSTRUCTION MATERIAL AND STORED MATERIALS

A-1 BUILDING MATERIALS

The unit weight of materials used in construction of buildings and structures are specified in Table 40. Also mentioned alongside are the corresponding Indian Standards, if applicable

Table 40 Unit Weight of Building Materials

(*Clause* A-1)

Sl No.	Materials	Nominal Size or Thickness mm	Weight	
			kN	per
(1)	(2)	(3)	(4)	(5)
i)	**Acoustical Material (*see* IS 2526) [C-1(15)]**			
	Eelgrass	10	0.57 to 0.765 × 10^{-2}	m^2
	Glass fibre, min	10	0.38 × 10^{-2}	m^2
	Hair	10	1.91 × 10^{-2}	m^2
	Mineral wool	10	1.345 × 10^{-2}	m^2
	Slag wool		2.65	m^3
	Cork		2.35	m^3
	Wood fibre board		5.9 to 6.15	m^3
	Wood particle Board			
	As per IS 3129 [C-1(15)]		3.92	m^3
	As per IS 12823 [C-1(15)]		4.9 to 9.9	m^3
	As per IS 3478 [C-1(15)]		8.9 to 11.8	m^3
	Compressed wood wool	-	3.95 to 4.45	m^3
	Mineral/glass wool quilts and mats	-	0.157 to 0.314	m^3
	Mineral compressed glass wool tiles	-	1.87 to 2.06	m^3
	Polyester board	-	1.87 to 2.06	m^3
ii)	**Aggregate, Coarse**			
	Broken stone ballast			
	Dry, well-shaken	–	15.70 to 18.35	m^3
	Perfectly wet	–	18.85 to 21.95	m^3
	Shingles, 3 to 38 mm	–	14.35	m^3
	Broken bricks:			
	Fine	–	14.20	m^3

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	Coarse	–	9.90	m^3
	Foam slag (foundry pumice)	–	6.85	m^3
	Cinder	–	7.85	m^3
	Slag			
	Iron slag		32.4	m^3
	Steel Slag		31 to 36	m^3
	Copper slag		31 to 36	m^3
	River pebbles		15	m^3
iii)	**Aggregate, Fine**			
	Sand			
	Dry, clean	–	15.10 to 15.70	m^3
	River	–	18.05	m^3
	Wet	–	17.25 to 19.60	m^3
	Brick dust (*SURKHI*)	–	9.90	m^3
	Crushed stone sand (*see* IS 383) [C-1(15)]		19 to 20	m^3
	Quarry dust		14 to 20	m^3
iv)	**Aggregate, Organic**			
	Saw dust, loose	–	1.55	m^3
	Peat			
	Dry	–	5.50 to 6.30	m^3
	Sandy, compact	–	7.85	m^3
	Wet, compact	–	13.35	m^3
v)	**Asbestos**			
	Felt	10	0.145	m^2
	Fibres:			
	Pressed	–	9.40	m^3
	Sprayed	10	0.02	m^2
	Natural	–	29.80	m^3
	Raw	–	5.90 to 8.85	m^3
vi)	**Asbestos Cement Building Pipes** (*see* Sl No. xliii 'pipes' in this Table)			
vii)	**Asbestos Cement Gutters** [*see* IS 1626 (Part 2)] [C-1(15)]			
	Boundary wall gutters			
	510 mm × 150 mm × 255 mm	12.5	0.180	m
	455 mm × 150 mm × 305 mm	12.5	0.158	m
	305 mm × 150 mm × 225 mm	12.5	0.129	m
	280 mm × 125 mm × 180 mm	12.5	0.106	m
	Valley gutters			

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	915 mm × 205 mm × 230 mm	12.5	0.250	m
	610 mm × 150 mm × 230 mm	12.5	0.162	m
	455 mm × 125 mm × 150 mm	12.5	0.146	m
	405 mm × 125 mm × 255 mm	12.5	0.131	m
	Half round gutters			
	150 mm	9.5	0.043	m
	230 mm	9.5	0.073	m
	305 mm	9.5	0.088	m
viii)	**Asbestos Cement Pressure Pipes** (*see* Sl No. xliii 'Pipes' in this Table)			
ix)	**Asbestos Cement Sheeting** (*see* IS 459) [C-1(15)]			
	Corrugated (pitch – 146 mm)	6	0.118 to 0.130	m^2
	Semi-corrugated (pitch 388 mm)	6	0.118 to 0.127	m^2
	Plain	5	0.09	m^2
x)	**Bitumen**	–	0.8-1	m^3
xi)	**Binders**			
	Fly Ash loose		5.05 to 7.05	m^3
	Fly Ash compacted		16.00	m^3
	Ground Granulated blast furnace slag		14.9	m^3
	Silica fume		3.5 to 7	m^3
	Rice husk ash		1.0 to 3.5	m^3
	Metakaolin		8 to 10	m^3
xii)	**Blocks**			
	Lime-based solid blocks (*see* IS 3115) [C-1(15)], *min*	–	9.8	m^3
	concrete blocks [*see* IS 2185 (Part 1)] [C-1(15)]			
	Grade A (hollow)	–	15	m^3
	Grade B (hollow)	–	11 to 15	m^3
	Grade C (Solid)	–	18	m^3
	Autoclaved cellular concrete blocks [*see* IS 2185 (Part 3)] [C-1(15)]		4.42 to 9.81	m^3
xiii)	**Boards**			
	Cork boards [*see* IS 4253 (Part 1 and Part 2)] [C-1(15)]			
	Compressed		2.16 to 3.73	m^3
	Ordinary		2.15 to 3.72	m^3
	Fibre hard boards (*see* IS 1658) [C-1(15)]			

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	Medium hardboard		3.43 to 7.84	m^3
	Standard hardboard		7.84 to 10.05	m^3
	Tempered hardboard		7.84 to 10.05	m^3
	Fibre cement boards (*see* IS 14862) [C-1(15)]		13 to 16	m^3
	Fibre insulation board, ordinary or flame-retardant type bitumen-bounded fibre insulation board (*see* IS 3348*)* [C-1(15)]*, Max*	9, 12, 18, 25	3.924	M^3
	Plain gypsum plaster boards [*see* IS 2095 (Part 1)] [C-1(15)]			
	Type A	9.5, 12.5, 15	5.39	m3
	Type H1 and H2	9.5, 12.5, 15	5.88	m3
	Type D	9.5, 12.5, 15	7.84	m3
	Type F and FH	9.5, 12.5, 15	7.84	m3
	Type R1	9.5, 12.5, 15	8.33	m3
	Type R2	9.5, 12.5, 15	9.31	m3
	Type W	9.5, 12.5, 15	8.33	m3
	Coated/laminated gypsum plaster boards [*see* IS 2095 (Part 2)] [C-1(15)]	9.5, 12.5, 15	11.27	M^3
	Reinforced gypsum plaster boards and ceiling tiles [*see* IS 2095 (Part 3)] [C-1(15)], *min*			
	Fibre reinforced Gypsum		7.84	m^3
	Glass reinforced Gypsum plaster board		24.52	m^3
	Glass reinforced Gypsum plaster tiles		5.88	m^3
	Insulating board (*fibre*) (*see* IS 3348) [C-1(15)], *mean*		0.981	m^3
	Laminated board (*fibre*) Medium density (*see* IS 14587) [C-1(15)]		5.88 to 8.82	m^3
	Wood particle boards (*see* IS 3087) [C-1(15)]	–	4.90 to 8.85	m^3
	Low density wood particles boards for insulation purposes (*see* IS 3129) [C-1(15)]	–	3.52	m^3
	High density wood particle boards (*see* IS 3478) [C-1(15)]			
	Type 1 & 2, Grade A	–	11.30	m^3
	Type 1 & 2, Grade B	–	7.94	m^3

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xiv)	**Bricks**			
	Common burnt clay bricks (*see* IS 1077) [C-1(15)]	–	15.70 to 18.85	m^3
	Engineering bricks	–	21.20	m^3
	Heavy duty bricks (*see* IS 2180) [C-1(15)]	–	24.50	m^3
	Pressed bricks	–	17.25 to 18.05	m^3
	Refractory bricks	–	17.25 to 19.60	m^3
	Sand cement bricks	–	18.05	m^3
	Sand lime bricks	–	20.40	m^3
	Pulverized fuel-ash lime bricks (*see* IS 12894) [C-1(15)]		12 to 15	m^3
	Pulverized fuel-ash cement bricks (*see* IS 16720) [C-1(15)]		10.8 to 19.6	m^3
	Acid resistance bricks (*see* IS 4860) [C-1(15)]		22.5 to 24.83	m^3
	Soil based bricks (*see* IS 1725) [C-1(15)]		17.5	m^3
	Perforated building bricks (*see* IS 2222) [C-1(15)]		6.8 to 7.75	m^3
	Calcium silicate bricks (*see* IS 4139) [C-1(15)]		25 to 29	m^3
xv)	**Brick Chips and Broken Bricks** (*see* under Sl No. ii 'Broken Bricks' in this Table)			
xvi)	**Brick dust (*SURKHI*)**	–	9.00	m^3
xvii)	**Cast Iron, Manhole Covers** (*see* IS 1726) [C-1(15)]			
	Double triangular (HD)	500	1.16	Cover
		560	1.37	Cover
	Circular (HD)	500	1.16	Cover
		560	1.37	Cover
	Circular (MD)	500	0.57	Cover
		560	0.63	Cover
	Rectangular (MD)	–	0.78	Cover
	Rectangular (LD) :			
	Single seal (Pattern 1)	–	0.23	Cover
	Single seal (Pattern 2)	–	0.15	Cover
	Double seal	–	0.28	Cover
	Square (LD):			
	Single seal	455	0.13	Cover
		610	0.25	Cover
	Double seal	455	0.23	Cover
		610	0.36	Cover

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xviii)	**Cast Iron, Manhole Frames** (*see* IS 1726) [C-1(15)]			
	Double triangular (HD)	500	1.09	Frame
		560	1.13	Frame
	Circular (HD)	500	0.83	Frame
		560	1.06	Frame
	Circular (MD)	500	0.57	Frame
		560	0.63	Frame
	Rectangular (MD)	–	0.63	Frame
	Rectangular (LD):			
	Single seal (Pattern 1)	–	0.15	Frame
	Single seal (Pattern 2)	–	0.10	Frame
	Double seal	–	0.23	Frame
	Square (LD):			
	Single seal	455	0.07	Frame
		610	0.13	Frame
	Double seal	455	0.15	Frame
		610	0.18	Frame
xix)	**Cast Iron Pipes** (*see* under Sl No. xliii 'Pipes' in this Table)			
xx)	**Cement**			
	Ordinary (*see* IS 269) and other types	–	14.10	m^3
	Rapid-hardening (*see* IS 8041) [C-1(15)]	–	12.55	m^3
xxi)	**Ceilings**			
	Plaster on tile or concrete	13 mm thick	0.25	m^2
	Plaster on wood lath	25 mm thick	0.39	m^2
	Suspended metal lath and cement plaster	25 mm thick	0.74	m^2
	Suspended metal lath and gypsum plaster	25 mm thick	0.49	m^2
xxii)	**Cement Concrete, Plain**			
	Aerated		7.45	m^3
	No-fines, with heavy aggregate	–	15.70 to 18.80	m^3
	No-fines, with light aggregate	–	8.65 to 12.55	m^3
	With burnt clay aggregate	–	17.25 to 21.20	m^3
	With expanded clay aggregate	–	9.40 to 16.50	m^3
	With clinker aggregate	–	12.55 to 17.25	m^3
	With pumice aggregate	–	5.50 to 11.00	m^3
	With sand and gravel or crushed natural stone aggregate	–	22.00 to 23.50	m^3

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	With saw dust	–	6.30 to 16.50	m^3
	With foamed slag aggregate	–	9.40 to 18.05	m^3
	With light weight cinder aggregate		6.4	m^3
	Light weight concrete		8 to 20	m^3
	Cellular concrete (Foam concrete) (*see* IS 6598) [C-1(15)]		3 to 5	m^3
	Grade A, *max*		3.13	m^3
	Grade B		3.13 to 3.92	m^3
	Grade C		3.93 to 4.9	m^3
	Transparent concrete		15	m^3
	Glass fiber reinforced concrete		18 to 22	m^3
	Alkali activated (Geopolymer) concrete (*see* IS 17452) [C-1(15)]		24 to 25	m^3
	Shotcrete (*see* IS 9012) [C-1(15)]		22 to 25	m^3
	Ferro cement		22 to 25	m^3
	Roller compacted concrete		22 to 25	m^3
	Controlled low strength mix		22 to 25	m^3
	Plum concrete (*see* IS 457) [C-1(15)]		22 to 25	m^3
	Autoclaved Aerated Concrete		4.42 to 9.81	m^3
	Vermiculite concrete		6 to 9	m^3
xxiii)	**Cement Concrete, Prestressed** (conforming to IS 1343) [C-1(15)]	–	23.50	m^3
xxiv)	**Cement Concrete, Reinforced:**			
	With 1 percent steel	–	22.75 to 24.20	m^3
	With 2 percent steel	–	23.25 to 24.80	m^3
	With 5 percent steel	–	24.80 to 26.50	m^3
xxv)	**Damp-proofing** (*see* Sl No. xxviii 'Felt bituminous for waterproofing and damp proofing')			
xxvi)	**Earth filling** (*see* Sl No. xlviii 'Soils and gravels in Table 1)			
xxvii)	**Expanded Metal** (conforming to IS 412) [C-1(15)]			
	Size of Mesh, Nominal			
	SWM	LWM		
	mm	mm		
	100	250	0.030	m^2
	100	250	0.024	m^2
	100	250	0.016	m^2

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	75	200	0.042	m^2
	75	200	0.032	m^2
	75	200	0.021	m^2
	40	115	0.080	m^2
	40	115	0.060	m^2
	40	75	0.060	m^2
	40	75	0.028	m^2
	40	115	0.039	m^2
	40	75	0.039	m^2
	40	115	0.020	m^2
	40	75	0.020	m^2
	25	75	0.054	m^2
	25	75	0.038	m^2
	25	75	0.028	m^2
	25	75	0.021	m^2
	20	60	0.070	m^2
	20	50	0.070	m^2
	20	60	0.050	m^2
	20	50	0.050	m^2
	20	60	0.036	m^2
	20	50	0.036	m^2
	20	50	0.021	m^2
	20	60	0.021	m^2
	12.5	50	0.050	m^2
	12.5	50	0.050	m^2
	12.5	40	0.040	m^2
	12.5	50	0.030	m^2
	12.5	40	0.030	m^2
	12.5	50	0.025	m^2
	12.5	40	0.025	m^2
	10	40	0.050	m^2
	10	40	0.035	m^2
	10	40	0.028	m^2
	95	285	0.050	m^2
	95	285	0.028	m^2
	95	285	0.020	m^2
	6	25	0.074	m^2
	6	25	0.048	m^2

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	6	25	0.038	m^2
	5	20	0.050	m^2
	3	15	0.041	m^2
xxviii)	**Felt, Bituminous for Waterproof and Damp–proofing** (*see* IS 1322) [C-1(15)]			
	Fibre base			
	Type 1 (under lay)	–	7.6×10^{-3}	m^2
	Type 2 (self-finished felt)	-	2.6×10^{-2}	m^2
	Hessian base			
	Type 3 (self-finished felt)			
	Grade 1	–	2.26×10^{-2}	m^2
	Grade 2	–	3.64×10^{-2}	m^2
xxix)	**Finishing** (*see* also 'Floor finishes' given under Sl No. xxxi 'Flooring' and xlv 'Roofing'in Table 1)			
	Plaster:			
	Acoustic	10	0.08	m^2
	Anhydrite	10	0.21	m^2
	Barium Sulphate	10	0.28	m^2
	Fibrous	10	0.09	m^2
	Gypsum or lime	10	0.19	m^2
	Hydraulic lime or cement	10	0.23	m^2
	Plaster ceiling on wire netting	10	0.26	m^2
	NOTE – When wood or metal lathing is used, add	–	0.06	
xxx)	**Float Glass** (*see* IS 2835) **and Safety Glass** [*see* IS 2553 (Part 1)] [C-1(15)]			
	Sheet	2	0.049	m^2
		2.5	0.062	m^2
		3	0.074	m^2
		4	0.098	m^2
		5	0.123	m^2
		5.5	0.134	m^2
		6.5	0.167	m^2
		8	0.200	m^2
		10	0.250	m^2
		12	0.300	m^2
		15	0.375	m^2
		19	0.475	m^2
xxxi)	**Flooring**			
	Asphalt flooring	10	0.22	m^2
	NOTE — For macadam finish, add	10	0.26	m^2

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	Compressed cork	10	0.04	m^2
	Floors, structural:			
	Hollow clay blocks including reinforcement and mortar jointing between blocks, but excluding any concrete topping	100	1.47	m^2
		125	1.67	m^2
		150	1.86	m^2
		175	2.16	m^2
		200	2.55	m^2
	NOTE — Add extra for concrete topping			
	Hollow clay blocks including reinforcement and concrete ribs between blocks, but excluding any concrete topping	100	1.18	m^2
		115	1.27	m^2
		125	1.37	m^2
		140	1.47	m^2
		150	1.57	m^2
		175	1.76	m^2
		200	1.96	m^2
	NOTE — Add extra for concrete topping.			
	Hollow concrete units including any concrete topping necessary for constructional purposes	100	1.67	m^2
		125	1.96	m^2
		150	2.16	m^2
		175	2.35	m^2
		200	2.65	m^2
		230	3.14	m^2
	Floors, wood:			
	Hard wood	22	0.16	m^2
		28	0.20	m^2
	Soft wood	22	0.11	m^2
		28	0.13	m^2
	Weight of mastic used in laying wood block flooring	–	0.015	m^2
	NOTE — All thicknesses are ‘finished thicknesses’.			
	Floor finishes:			
	Clay floor tiles (*see* IS 1478) [C-1(15)]	12.5 to 25.4	0.10 to 0.2	m^2
	NOTE – This weight is ‘as laid’ but excludes screeding.			
	Magnesium oxychloride:			
	Normal type (saw dust filler)	10	0.142	m^2
	Heavy duty type (mineral filler)	10	0.216	m^2
	Parquet flooring	–	0.08 to 0.12	m^2
	Rubber (*see* IS 809) [C-1(15)]	3.2	0.048 to 0.062	m^2

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		4.8	0.070 to 0.09	m^2
		6.4	0.093 to 0.130	m^2
	Terra cotta, filled 'as laid'	–	5.54 to 7.06	m^2
	Terrazzo paving 'as laid'	10	0.23	m^2
xxxii)	**Foam Slag, Foundry Pumice**	–	6.85	m^3
xxxiii)	**Gutters, Asbestos Cement** (*see* under vii '**Asbestos cement gutter**' in this Table)			
xxxiv)	**Gypsum**			
	Gypsum mortar [*see* IS 2547 (Part 1 and Part 2)] [C-1(15)]	–	18.6	m^3
	Gypsum powder (*see* IS 12679) [C-1(15)]	–	13.89 to 17.25	m^3
xxxv)	**Iron**			
	Pig	–	70.60	m^3
	Gray, cast	–	68.95 to 69.90	m^3
	White, cast	–	74.30 to 75.70	m^3
	Wrought	–	75.70	m^3
xxxvi)	**Lime**			
	Lime concrete with burn clay Aggregate (*see* IS 2541) [C-1(15)]	–	18.80	m^3
	Lime mortar	–	15.70 to 18.05	m^3
	Lime plaster (*see* IS 2394) [C-1(15)]	–	17.25	m^3
	Lime stored in lumps, uncalcined	–	12.55 to 14.10	m^3
	Lime, unslaked, freshly burnt in pieces	–	8.60 to 10.20	m^3
	Lime slaked, fresh	–	5.70 to 6.30	m^3
	Lime slaked, after 10 days	–	7.85	m^3
	Lime, unslaked (KANKAR)	–	11.55	m^3
	Lime, slaked (KANKAR)	–	10.00	m^3
xxxvii)	**Linoleum** (*see* IS 653) [C-1(15)]			
	Sheets and tiles	4.5	0.0569	m^2
		3.2	0.0402	m^2
		2.0	0.0265	m^2
		1.65	0.0215	m^2
xxxviii)	**Masonry**			
	Brick Masonry (excluding plaster)			
	With common burnt clay bricks (*see* IS 1077) [C-1(15)]	–	19.25	m^3

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	Engineering bricks	–	21.2	m^3
	Glazed bricks	–	20.40	m^3
	Pressed bricks	–	18.65	m^3
	Block Masonry			
	With lime based solid blocks		13.55	m^3
	Hollow concrete blocks			
	Grade A		15.4	m^3
	Grade B		12.00	m^3
	Solid Concrete Blocks		18.15	m^3
	Autoclaved cellular concrete blocks		9.1 to 11.1	m^3
xxxix	**Masonry, Stone**			
	Cast	–	21.75	m^3
	Dry rubble	–	20.40	m^3
	Granite ashlar	–	25.00	m^3
	Granite rubble	–	23.55	m^3
	Lime stone ashlar	–	25.10	m^3
	Marble dressed	–	26.50	m^3
	Sand stone	–	22.00	m^3
xl)	**Mastic asphalt**	10	0.216	m^2
xli)	**Metal sheeting, protected galvanized steel sheets, plain** (*see* IS 277) [C-1(15)]			
	Class A (corrugated)	1.60	0.131	m^2
		1.00	0.104	m^2
		0.9	0.084	m^2
		0.80	0.069	m^2
		0.63	0.056	m^2
	Class B (corrugated)	1.60	0.129	m^2
		1.00	0.102	m^2
		0.9	0.083	m^2
		0.80	0.067	m^2
		0.63	0.054	m^2
	Class C (corrugated)	1.60	0.128	m^2
		1.00	0.101	m^2
		0.9	0.081	m^2
		0.80	0.066	m^2
		0.63	0.053	m^2
	Class D (corrugated)	1.60	0.127	m^2
		1.00	0.100	m^2
		0.9	0.081	m^2

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		0.80	0.065	m^2
		0.63	0.052	m^2
xlii)	**Mortar**			
	Cement	–	20.40	m^3
	Gypsum	–	11.75	m^3
	Lime	–	15.70 to18.05	m^3
xliii)	**Pipes**			
	Asbestos cement pipes [*see* IS 1626 (Part 1)] [C-1(15)]	50	0.032 to 0.034	m
		60	0.032 to 0.043	m
		80	0.051 to 0.054	m
		90	0.052 to 0.060	m
		100	0.058 to 0.065	m
		125	0.072 to 0.086	m
		150	0.086 to 0.108	m
	Asbestos cement pressure pipes [(*see* IS 1592) [C-1(15)]	50	0.056	m
		80	0.067	m
		100	0.090	m
		125	0.139	m
		150	0.175	m
		200	0.264	m
		250	0.380	m
		300	0.539	m
	Cast iron pipes (*see* IS 1729) [C-1(15)]			
	Standard overall length 1.8 m with socket	50	0.073	pipe
		75	0.108	pipe
		100	0.137	pipe
		125	0.196	pipe
		150	0.255	pipe
	Standard overall length 1.5 m with socket	50	0.064	pipe
		75	0.093	pipe
		100	0.123	pipe
		125	0.172	pipe
		150	0.230	Pipe
	Pressure pipes for water, gas and sewage:			
	a) Centrifugally cast (*see* IS 1536) [C-1(15)]			
	Socket and spigot pipes			
	Barrel:			
	Class LA	80	0.145	m

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		100	0.182	m
		125	0.237	m
		150	0.297	m
		200	0.432	m
		250	0.582	m
		300	*0.750*	m
		350	0.944	m
		400	1.146	m
		450	1.383	m
		500	1.620	m
		600	2.156	m
		700	2.718	m
		750	3.111	m
	Class A	80	0.151	m
		100	0.201	m
		125	0.259	m
		150	0.326	m
		200	0.472	m
		250	0.637	m
		300	0.824	m
		350	1.030	m
		400	1.262	m
		450	1.530	m
		500	1.775	m
		600	2.367	m
		700	3.056	m
		750	3.422	m
	Class B	80	0.172	m
		100	0.216	m
		125	0.281	m
		150	0.352	m
		200	0.511	m
		250	0.692	m
		300	0.896	m
		350	1.112	m
		400	1.368	m
		450	1.657	m
		500	1.929	m

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		600	2.578	m
		700	3.317	m
		750	3.733	m
	Sockets for Class LA, Class A and Class B barrels	80	0.054	Socket
		100	0.069	Socket
		125	0.090	Socket
		150	0.113	Socket
		200	0.165	Socket
		250	0.225	Socket
		300	0.292	Socket
		350	0.368	Socket
		400	0.454	Socket
		450	0.549	Socket
		500	0.647	Socket
		600	0.876	Socket
		700	1.145	Socket
		750	1.292	Socket
	b) Flanged pipe with screwed flanges			
	Barrel:			
	Class A	80 to 300	Same as for centrifugally cast socket and spigot piles, Class A	
	Class B	80 to 300	Same as for centrifugally cast socket and spigot piles, Class B	
	Flanges for Class A and Class B barrels	80	0.042	Flange
		100	0.049	Flange
		125	0.065	Flange
		150	0.080	Flange
		200	0.112	Flange
		250	0.144	Flange
		300	0.182	Flange
	c) Vertically cast socket and spigot pipes (*see* IS 1537) [C-1(15)]			
	Barrel: Class A	80 to 750	Same as for centrifugally cast socket and spigot pipes, Class A	
		800	3.82	m
		900	4.65	m
		1000	5.59	m
		1100	6.59	m
		1200	7.67	m
		1500	11.98	m

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	Class B	80 to 750	–	–
		800	–	m
		900	5.07	m
		1000	6.07	m
		1100	7.23	m
		1200	8.35	m
		1500	13.07	m
	Socket of Class A and Class B barrels	80 to 750	–	–
		800	–	Socket
		900	179	Socket
		1000	218	Socket
		1100	260	Socket
		1200	360	Socket
		1500	491	Socket
	d) Sand cast (flanked pipes)			
	Barrel:			
	Class A	80 to 750	Same as for centrifugally cast socket and spigot pipes, Class A	
		800 to 1500	–	
	Class B	80 to 750	Same as for vertically cast socket and spigot pipes, CIass B	
		800 to 1500		
	Flanges for Class A and Class B barrels	80	0.036	Flange
		100	0.041	Flange
		125	0.052	Flange
		150	0.066	Flange
		200	0.091	Flange
		250	0.117	Flange
		300	0.145	Flange
		350	0.186	Flange
		400	0.229	Flange
		450	0.250	Flange
		500	0.315	Flange
		600	0.431	Flange
		750	0.587	Flange
		700	0.685	Flange
		800	0.792	Flange
		900	0.928	Flange
		1000	1.18	Flange

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		1100	1.38	Flange
		1200	1.70	Flange
		1500	2.71	Flange
	Concrete pipes (*see* IS 458) [C-1(15)]			
	Class NP 1 (unreinforced non-pressure pipes)	80	0.19	m
		100	0.22	m
		150	0.30	m
		250	0.40	m
		300	0.69	m
		350	0.84	m
		400	0.95	m
		450	1.17	m
	Class NP 2 (reinforced concrete, light duty, non-pressure pipes)	80	0.196	m
		100	0.215	m
		150	0.324	m
		250	0.510	m
		300	0.736	m
		350	0.902	m
		400	1.02	m
		450	1.26	m
		500	1.38	m
		600	1.89	m
		700	2.19	m
		800	2.81	m
		900	3.51	m
		1000	4.30	m
		1100	5.15	m
		1200	6.09	m
		1400	8.18	m
		1600	9.93	m
		1800	12.58	m
	Class NP 3 (reinforced concrete, heavy duty, non-pressure pipes)	350	2.35	m
		400	2.63	m
		450	2.91	m
		500	3.19	m
		600	4.02	m
		700	4.61	m
		800	5.92	m

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		900	7.39	m
		1000	8.13	m
		1100	10.34	m
		1200	11.18	m
	Class P1 (reinforced concrete pressure pipes safe for 20 MPa pressure tests)	80	0.196	m
		100	0.235	m
		150	0.324	m
		250	0.510	m
		300	0.736	m
		350	0.902	m
		400	1.02	m
		450	1.26	m
		500	1.38	m
		600	1.89	m
		700	2.19	m
		800	2.81	m
		900	3.51	m
		1000	4.30	m
		1100	5.15	m
		1200	6.09	m
	Class P2 (reinforced concrete pressure pipes safe for 40 MPa pressure tests)	80	0.196	m
		100	0.235	m
		150	0.324	m
		250	0.608	m
		300	1.01	m
		350	1.31	m
		400	1.67	m
		450	1.84	m
		500	1.56	m
		600	3.20	m
	Class P3 (reinforced concrete pressure pipes safe for 60 MPa pressure tests)	80	0.196	m
		100	0.235	m
		150	0.324	m
		250	0.736	m
		300	1.15	m
		350	1.65	m
		400	2.04	m

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	Lead pipes [*see* IS 404 (Part 1)] [C-1(15)] (service and distribution pipes 10 be laid underground) :			
	For working 40 MPa	10	0.018	m
		13	0.031	m
		20	0.042	m
		25	0.060	m
		30	0.074	m
		40	0.091	m
		50	0.142	m
	For working 70 MPa	10	0.022	m
		15	0.038	m
		20	0.050	m
		25	0.069	m
		32	0.126	m
		40	0.175	m
	For working 100 MPa	10	0.029	m
		15	0.048	m
		20	0.067	m
		25	0.105	m
	Service pipes to be fixed or laid above ground:			
	For working pressure 40 MPa	10	0.014	m
		15	0.021	m
		20	0.027	m
		25	0.036	m
		32	0.059	m
		40	0.091	m
		50	0.142	m
	For working pressure 70 MPa	10	0.018	m
		15	0.024	m
		20	0.030	m
		25	0.069	m
		32	0.126	m
		40	0.175	m
	For working pressure 100 MPa	10	0.029	m
		15	0.048	m
		20	0.067	m
		25	0.105	m

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	Cold water distribution pipes to be fixed or laid above ground:			
	For working pressure 25 MPa	10	0.014	m
		15	0.021	m
		20	0.027	m
		25	0.036	m
		32	0.048	m
		40	0.067	m
		50	0.084	m
	For working pressure 40 MPa	10	0.014	m
		15	0.021	m
		20	0.027	m
		25	0.036	m
		32	0.059	m
		40	0.091	m
		50	0.142	m
	Hot water distribution pipes to be fixed or laid above ground:			
	For working pressure 20 MPa	10	0.015	m
		15	0.023	m
		20	0.031	m
		25	0.041	m
		32	0.062	m
		40	0.082	m
		50	0.142	m
	For working pressure 35 MPa	10	0.015	m
		15	0.027	m
		20	0.045	m
		25	0.085	m
		32	0.132	m
	Soil, waste and soil and waste ventilation pipes	50	0.050	m
		75	0.073	m
		100	0.097	m
		150	0.160	m
	Flushing and warning pipes	20	0.020	m
		25	0.025	m
		32	0.032	m
		40	0.039	m
		50	0.049	m
	Gas pipes:			
	Heavy weight gas pipes	10	0.008	m

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		15	0.017	m
		20	0.025	m
		25	0.034	m
		32	0.045	m
		40	0.061	m
		50	0.071	m
	Light weight gas pipes	10	0.008	m
		15	0.012	m
		20	0.020	m
		25	0.029	m
		32	0.037	m
		40	0.047	m
		50	0.058	m
	Stoneware, salt-glazed pipes (*see* IS 651) [C-1(15)]	100	0.137	m
		150	0.216	m
		200	0.324	m
		230	0.412	m
		250	0.510	m
		300	0.775	m
		350	0.980	m
		400	1.26	m
		450	1.44	m
		500	1.77	m
		600	2.35	m
xliv)	**Plaster**			
	Cement	10	0.204	m^2
	Lime	10	0.172	m^2
	Acoustic	10	0.078	m^2
	Anhydrite	10	0.206	m^2
	Barium sulphate	10	0.284	m^2
	Fibrous	10	0.088	m^2
	Gypsum	10	0.186	m^2
xlv)	**Roofing**			
	Asbestos cement sheeting			
	(*see* 'Asbestos cement sheeting' in Table 1)			
	Allahabad tiles (single) including battens (*see* Note below)	–	0.83	m^2

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	Allahabad tiles (double) including battens (*see* Note below)	–	1.67	m^2
	Country tiles (single) including battens (*see* Note below)	–	0.69	m^2
	Country tiles (double) including battens (*see* Note below)	–	1.18	m^2
	Mangalore tiles battens (*see* Note below)	–	0.64	m^2
	Mangalore tiles bedded in mortar over flat tiles (*see* Note below)	–	1.08	m^2
	Mangalore tiles with flat tiles (*see* Note below)	–	0.78	m^2
	Copper sheet roofing including laps and rolls	0.56	0.08	m^2
	Flat roofs	0.72	0.10	m^2
	Clay tiles- hollow clay blocks/clay floor tiles (*see* Sl No. xxxi 'Flooring')			
	Concrete hollow units (*see* Sl No. xxxi 'Flooring' in this Table)			
	Galvanized iron sheeting (*see* Sl No. 'xli Metal sheeting, protected')			
	Glazed Roofing:			
	Glazing with aluminum alloy bars for spans up to 3 m	6.4	0.19	m^2
	Glazing with lead-covering steel bars at 0.6 m centres	6.4	0.25 to 0.28	m^2
	States on battens	–	0.34 to 0.49	m^2
	Thatch with battens	–	0.34 to 0.49	m^2
	NOTE – Weights acting vertically on horizontally projection to be multiplied by cosine of roof angle to obtain weights normal to the roof surface.			
	Roof finished			
	Bitumen macadam	10	0.22	m^2
	Felt roofing (*see* 28 'Felt bituminous for water-proofing and damp-proofing' in Table 1)	10	0.008	m^2
	Glass silk quilted	0.5	0.05	m^2
	Lead sheet	0.8	0.07	m^2
	Mortar screeding	10	0.21	m^2
xlvi)	**Sheeting**			
	Asbestos (*see* under Sl No. ix 'asbestos cement sheeting' in this Table)			
	Galvanized iron (*see* under Sl No. xli 'metal sheeting, protected' in this table)			
	Glass (*see* under Sl No. xxx 'Glass' in this table)			
	Plywood (*see* IS 303) [C-1(15)]	1	0.007	m^3

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xlvii)	**Slugwool**	–	2.65	m^3
xlviii)	**Soils and Gravels**			
	Alluvial ground, undisturbed	–	15.69	m^3
	Broken stone ballast:			
	Dry, well-shaken	–	15.70 to 18.35	m^3
	Perfectly wet	–	18.85 to 21.95	m^3
	Chalk		15.70 to 18.85	m^3
	Clay:			
	China, compact	–	21.95	m^3
	Clay fills:			
	Dry, lumps	–	10.20	m^3
	Dry, compact	–	14.10	m^3
	Damp, compact	–	17.25	m^3
	Wet , compact	–	20.40	m^3
	Undisturbed	–	18.85	m^3
	Undisturbed, gravelly	–	20.40	m^3
	Crush stone sand (Msand)	–	–	–
	Earth:			
	Dry	–	13.85 to 18.05	m^3
	Moist	–	15.70 to 19.60	m^3
	Gravel:			
	Loose	–	15.70	m^3
	Rammed	–	18.85 to 21.20	m^3
	Kaolin, compact	–	25.50	m^3
	Loam:			
	Dry, loose	–	11.75	m^3
	Dry, compact	–	15.70	m^3
	Wet, compact	–	18.85	m^3
	Loess, dry	–	14.10	m^3
	Marl, compact	–	17.25 to 18.85	m^3
	Mud, river, wet	–	17.25 to 18.85	m^3
	Peat:			
	Dry	–	5.50 to 6.30	m^3
	Sandy, compact	–	7.85	m^3
	Wet, compact	–	13.35	m^3
	Rip-rap	–	12.55 to 14.10	m^3
	Sand:			
	Dry, clean	–	15.10 to 15.70	m^3

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	River	–	18.05	m^3
	Wet	–	17.25 to 19.60	m^2
	Shingles:			
	Aggregate 3 to 38 mm	–	13.75	m^2
	Fine sand:			
	Dry	–	15.70	m^2
	Saturated	–	20.40	m^2
	Silt, wet	–	17.25 to 18.85	m^2
xlix)	**Steel sections**			
	Density of steel sections shall be taken as 78.50 kg/m^3			
	Weight of hot rolled sections (ISJB, ISLB, ISMB, ISWB, ISNPB, ISWPB, ISSC, ISHB, ISJC, ISLC, ISMC, ISMPC, ISA, ISPBP) shall be referred from (*see* IS 808) [C-1(15)]			
	Weight of hollow steel sections shall be referred from (*see* IS 4923) [C-1(15)]			
	Weight of cold formed light gauge structural steel sections shall be referred from (*see* IS 811) [C-1(15)]			
	Weight of rolled and slit T bars shall be referred from (*see* IS 1173) [C-1(15)]			
	Weight of steel sheet piling sections shall be referred from (*see* IS 2314) [C-1(15)]			
l)	**Stone**			
	Agate	–	25.50	m^3
	Aggregate	–	15.70 to 18.85	m^3
	Basalt	–	27.95 to 29.05	m^3
	Cast	–	21.95	m^3
	Chalk	–	21.50	m^3
	Dolomite	–	28.25	m^3
	Emery	–	39.25	m^3
	Flint	–	25.40	m^3
	Gneiss	–	23.55 to 26.40	m^3
	Granite	–	25.90 to 27.45	m^3
	Gravel:			
	Loose	–	15.70	m^3
	Moderately rammed, dry	–	18.85	m^3
	Green stone	–	28.25	m^3
	Gypsum	–	21.95 to 23.55	m^3
	Kota			
	Laterite	–	20.40 to 23.55	m^3
	Lime stone	–	23.55 to 25.90	m^3
	Marble	–	26.70	m^3
	Pumice	–	7.85 to 11.00	m^3
	Quartz rock	–	25.90	m^3

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	Sand stone	–	21.95 to 23.54	m^3
	Slate	–	27.45	m^3
	Soap stone	–	26.45	m^3
li)	**Tar, Coal**			
	Crude (*see* IS 212) [C-1(15)]	–	9.90	m^3
	Naphtha, light	–	9.90	m^3
	Naphtha, heavy	–	9.90	m^3
	Road tar	–	9.90	m^3
	Pitch (*see* IS 216) [C-1(15)]	–	9.90	m^3
lii)	**Thermal Insulation**			
	Unbonded glass wool	–	12.75 to 23.55	m^3
	Unbonded rock and slag wool	–	11.30 to 19.60	m^3
	Cellular concrete			
	Grade A	–	Up to 29.40	m^3
	Grade B	–	29.50 to 39.20	m^3
	Grade C	–	39.30 to 49.00	m^3
	Performed calcium silicate Insulation (for temperature up to 650°C)	–	19.60 to 34.30	m^3
liii)	**Tiles and stone cladding**			
	Terra-cotta	–	18.35 to 23.25	m^2
	Terrozzo			
	Paving	10	0.24	m^2
	Cast partitions	40	0.93	m^2
	Tiles			
	Mangalore pattern (*see* IS 654) [C-1(15)]	–	0.02 to 0.03	Tile
	Polystyrene wall tiles	99 x 99	0.013	m^2
		148.5 x 148.5	0.013	m^2
	Pressed ceramic tiles, glazed and unglazed (*see* IS 15622) [C-1(15)]		22	m^3
	Concrete paving blocks (*see* IS 15658) [C-1(15)]		24	m^3
liv)	**Timber**			
	Typical Indian timbers (*see* IS 399) [C-1(15)]			
	Aglaia	–	8.34	m^3
	Aini	–	5.83	m^3
	Alder	–	3.63	m^3
	Amari	–	6.13	m^3

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	Amla	–	7.85	m^3
	Amra	–	4.41	m^3
	Anjan	–	8.33	m^3
	Arjun	–	7.99	m^3
	Ash	–	7.06	m^3
	Axlewood	–	8.82	m^3
	Babul	–	7.70	m^3
	Baen	–	7.70	m^3
	Bahera	–	7.99	m^3
	Bakota	–	4.21	m^3
	Balasu	–	7.55	m^3
	Ballagi	–	11.13	m^3
	Bamboo			
	Banati	–	4.41	m^3
	Benteak	–	6.62	m^3
	Ber	–	6.91	m^3
	Bhendi	–	7.55	m^3
	Bijasal	–	7.85	m^3
	Black chuglam	–	6.13	m^3
	Birch	–	7.85	m^3
	Black locust	–	8.34	m^3
	Blue gum	–	8.34	m^3
	Blue pine	–	5.05	m^3
	Bola	–	6.42	m^3
	Bonsum	–	5.20	m^3
	Bullet wood	–	8.78	m^3
	Casuarina	–	8.34	m^3
	Cettis	–	6.42	m^3
	Champ	–	4.85	m^3
	Chaplash	–	5.05	m^3
	Chatian	–	4.07	m^3
	Chikrassy	–	6.62	m^3
	Chilauni	–	6.42	m^3
	Chilla	–	7.85	m^3
	Chir	–	5.64	m^3
	Chuglam:			
	Black	–	7.85	m^3
	White (silver grey-wood)	–	6.91	m^3

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	Cinnamon	–	6.42	m^3
	Cypress	–	5.05	m^3
	Debdaru	–	6.28	m^3
	Deodar	–	5.35	m^3
	Devdam	–	7.06	m^3
	Dhaman:			
	Grewia tiliofolia	–	7.70	m^3
	Grewia vestita	–	7.40	m^3
	Dhup	–	6.42	m^3
	Dilenia	–	6.13	m^3
	Dudhi	–	5.49	m^3
	Ebony	–	8.19	m^3
	Elim	–	5.20	m^3
	Eucalyptus	–	8.33	m^3
	Figs	–	4.56	m^3
	Fir	–	4.14	m^3
	Frash	–	6.62	m^3
	Gamari	–	5.05	m^3
	Gardenia	–	7.40	m^3
	Garuga	–	5.98	m^3
	Geon	–	4.07	m^3
	Gluta	–	7.06	m^3
	Gokul	–	4.07	m^3
	Grewia sp.	–	7.55	m^3
	Gurjan	–	7.70	m^3
	Gutel	–	4.41	m^3
	Haldu	–	6.62	m^3
	Hathipaila	–	5.84	m^3
	Hiwar	–	7.70	m^3
	Hollock	–	5.98	m^3
	Hollong	–	7.21	m^3
	Hoom	–	7.21	m^3
	Horse chestnut	–	5.05	m^3
	Imli	–	8.97	m^3
	Indian Chestnut	–	6.28	m^3
	Indian Hemlock	–	3.92	m^3
	Indian Oak	–	8.48	m^3
	Indian Olive	–	10.35	m^3

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	Irul	–	8.33	m^3
	Jack	–	5.83	m^3
	Jaman	–	7.70	m^3
	Jarul	–	6.13	m^3
	Jathtkai	–	5.05	m^3
	Jhingan	–	5.63	m^3
	Jutili	–	7.85	m^3
	Kadam	–	4.85	m^3
	Kail	–	5.05	m^3
	Kaim	–	6.42	m^3
	Kambli	–	4.07	m^3
	Kanchan	–	6.62	m^3
	Kanjuj	–	5.84	m^3
	Karada	–	8.34	m^3
	Karal	–	7.99	m^3
	Karani	–	6.28	m^3
	Karar	–	5.34	m^3
	Kardahi	–	9.27	m^3
	Karimgotta	–	3.92	m^3
	Kasi	–	5.83	m^3
	Kasum	–	10.84	m^3
	Kathal	–	5.85	m^3
	Keora	–	6.13	m^3
	Khair	–	9.00	m^3
	Khasipine	–	5.05	m^3
	Kindal	–	7.55	m^3
	Kokko	–	6.28	m^3
	Kongoo	–	9.76	m^3
	Kuchla	–	8.63	m^3
	Kumbi	–	7.70	m^3
	Kurchi	–	5.20	m^3
	Kurung	–	9.76	m^3
	Kusum	–	11.28	m^3
	Kuthan	–	4.71	m^3
	Lakooch	–	6.28	m^3
	Lambapatti	–	5.34	m^3
	Lampati	–	5.05	m^3
	Laurel	–	8.33	m^3

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	Lendi	–	7.40	m^3
	Machilus:			
	Gamblei	–	5.05	m^3
	Macrantha	–	5.20	m^3
	Maharukh	–	4.07	m^3
	Mahogany	–	6.62	m^3
	Mahua	–	8.97	m^3
	Maina	–	5.64	m^3
	Makai	–	3.14	m^3
	Malabar neem	–	4.41	m^3
	Mango	–	6.77	m^3
	Maniawga	–	7.40	m^3
	Maple	–	5.64	m^3
	Mesua	–	9.76	m^3
	Milla	–	9.12	m^3
	Mokha	–	7.99	m^3
	Mulberry	–	6.62	m^3
	Mullilam	–	7.21	m^3
	Mundani	–	6.77	m^3
	Murtenga	–	7.70	m^3
	Myrabolan	–	9.27	m^3
	Narikel	–	5.49	m^3
	Nedunar	–	5.05	m^3
	Oak	–	8.48	m^3
	Padauk	–	7.06	m^3
	Padri	–	7.06	m^3
	Palang	–	5.98	m^3
	Pali	–	6.28	m^3
	Papita	–	3.28	m^3
	Parrotia	–	8.48	m^3
	Persian lilac	–	5.84	m^3
	Piney	–	6.13	m^3
	Ping	–	8.97	m^3
	Pinus insignis	–	6.13	m^3
	Pipli	–	5.83	m^3
	Pitraj	–	6.77	m^3
	Poon	–	6.42	m^3
	Poplar	–	4.41	m^3

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	Pula	–	3.78	m^3
	Pyinma	–	3.98	m^3
	Rajbrikh	–	8.48	m^3
	Red sanders	–	10.84	m^3
	Rohini	–	11.33	m^3
	Rosewood (black wood)	–	8.19	m^3
	Rudrak	–	4.71	m^3
	Sal	–	8.48	m^3
	Salai	–	5.64	m^3
	Sandal wood	–	8.97	m^3
	Sadan	–	8.34	m^3
	Satin wood	–	9.41	m^3
	Saykaranji	–	7.40	m^3
	Seleng	–	4.85	m^3
	Semul	–	3.78	m^3
	Silver oak	–	6.28	m^3
	Siris	–	3.92	m^3
	Kala-siris	–	7.21	m^3
	Safed- siris	–	6.28	m^3
	Siaso	–	7.70	m^3
	Spruce	–	4.71	m^3
	Suji	–	2.65	m^3
	Sundri	–	9.41	m^3
	Talauma	–	5.64	m^3
	Tanaku	–	2.09	m^3
	Teak	–	6.28	m^3
	Toon	–	5.05	m^3
	Udal	–	2.50	m^3
	Upas	–	3.14	m^3
	Uriam	–	7.40	m^3
	Vakai	–	9.41	m^3
	Vellapine	–	5.83	m^3
	Walnut	–	5.64	m^3
	White bombwe	–	5.98	m^3
	White cedar	–	7.06	m^3
	White chuglam (silver grey-wood)	–	6.91	m^3
	White dhup	–	4.22	m^3
	Yon	–	8.33	m^3

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	NOTE — The unit of timbers correspond to average unit of typical Indian timbers at 12 percent moisture content.			
lv)	**Water**			
	Fresh	–	10.0	m^3
	Salt	–	10.05	m^3
lvi)	**Wood – Wool Building Slabs**	10mm	0.059	m^2
lvii)	**Walling** (*see* IS 6072) [C-1(15)]			
	Autoclaved reinforced cellular concrete wall slabs			
	Class A	–	8.35 to 9.80	m^2
	Class B	–	7.35 to 8.35	m^2
	Class C	–	6.35 to 7.35	m^2
	Class D	–	5.40 to 6.35	m^2
	Class E	–	4.40 to 5.40	m^2
	Brick masonry (*see* xxxviii 'Masonry Brick' in Table 1)			
	Blocks masonry (*see* xii 'Block' in Table 1)			
	Stone masonry (*see* xxxix 'Masonry Stone' in Table 1)			
	Partitions 39			
	Brick wall	100	1.91	m^2
	Cinder concrete	75	1.13	m^2
	Galvanized on sheet	–	0.15	m^2
	Hollow glass block (bricks)	100	0.88	m^2
	Hollow blocks per 200 mm of thickness			
	Ballast or stone concrete	200	0.201	m^2
	Clay	200	0.201	m^2
	Clinker concrete	200	0.220	m^2
	Coke breeze concrete	200	9.176	m^2
	Diatomaceous earth	200	0.093	m^2
	Gypsum	200	0.137	m^2
	Pumice concrete	200	0.177	m^2
	Slag concrete, air-cooled	200	0.196	m^2
	Slag concrete foamed	200	0.186	m^2
	Lath and plaster	–	0.392	m^2
	Solid blocks per 20 mm of thickness			
	Ballast or stone	20	0.451	m^2
	Clinker concrete	20	0.300	m^2
	Coke breeze concrete	20	0.221	m^2
	Pumice concrete	20	0.221	m^2
	Slag concrete, foamed	20	0.250	m^2
	Terrazzo cast partitions	40	0.932	m^2
	Timber studding plastered	–	9.981	m^2
NOTE — For unit weight of fixtures and fittings required to buildings including builder's hardware, reference may be made to appropriate Indian standards.				

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A-2 STORED AND MISCELLANEOUS MATERIALS

A-2.1 Units weights of stored and miscellaneous materials intended for dead load calculation and other general purposes are given in Table 41.

Table 41 Unit Weights of Stored and Miscellaneous Materials

(*Clause* A-2.1)

Sl No.	Materials	Weight KN/m^3	Angle of Friction, Degrees
(1)	(2)	(3)	(4)
i)	**Agriculture and Food Products**		
	Butter	8.45	–
	Coffee in bags	5.50	–
	Drinks in bottle, in boxes	7.35	–
	Eggs, packed	2.95	–
	Fats, oil	5.80	–
	Fish meal	4.90	45
	Flour in sacks up to 1 m height	2.20 to 5.90	–
	Forage (bales)	1.25	–
	Fruits	3.45	–
	Grains:		
	Barley	6.75	27
	Corn, shelled	7.55	27
	Flax seed	7.35	30
	Oats	5.30	30
	Rice	6.55	33
	Soyabeans	7.35	30
	Wheat	8.15	28
	Wheat flour	6.85	30
	Grain sheaves up to 4 m stack height	0.98	30
	Grain sheaves over 4 m stack height	1.45	30
	Grass and clover	3.45	–
	Hay:		–
	Compressed	1.65	–
	Loose up to about 3 m stack height	0.69	–
	Honey	14.10	–
	Hops:		
	In stacks	1.65	–
	In cylinder hop bins	4.60	–

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Table 41 (*Continued*)

Sl No.	Materials	Weight KN/m^3	Angle of Friction, Degrees
(1)	(2)	(3)	(4)
	Sewn up or compressed in cylindrical shape in hop cloth	2.85	–
	Malt:		
	Crushed	3.90	20
	Germinated	1.85	–
	Meat and meat products	7.05	–
	Milk	10.05	–
	Molasses	4.40	–
	Onion in bags	5.40	0
	Oil cakes, crushed	5.80	0
	Potatoes	7.05	30
	Preserves (tins in cases)	4.90 to7.85	–
	Bags	7.05	–
	Bulk	9.40	–
	Seeds:		
	Heaps	4.90 to 7.85	25
	Sacks	3.90 to 6.85	–
	Straw and chaff:		
	Loose up to about 3 m stack height	0.45	–
	Compressed	1.65	–
	Sugar:		
	Crystal	7.35	30
	Cube sugar in boxes	7.85	–
	Sugar beet, pressed out	7.85	–
	Tobacco bundles	3.45	–
	Vinegar	10.40	–
ii)	**Chemicals and Allied Materials**		
	Acid, hydrochloric	11.75	–
	Acid, nitric 91 percent	14.80	–
	Acid, sulphuric 87 percent	17.55	–
	Alcohol	7.65	–
	Alum, pearl in barrel	5.20	–
	Ammonia, liquid	8.85	–
	Ammonium, chloride, crystalline	8.15	30 to 40
	Ammonium nitrate	7.05 to 9.80	25
	Ammonium sulphate	7.05 to 9.00	32 to 45
	Beeswax	9.40	–
	Benzole	8.90	–
	Benzene hexachloride	8.75	–
	Bicarbonate of soda	6.40	–

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Table 41 (*Continued*)

Sl No.	Materials	Weight KN/m^3	Angle of Friction, Degrees
(1)	(2)	(3)	(4)
	Bone	18.65	–
	Borax	17.15	–
	Calcite	26.50	–
	Camphor	9.70	–
	Carbon disulphide	12.75	–
	Casein	13.25	–
	Caustic soda	13.85	–
	Creosole	10.50	–
	Dicalcium phosphate	6.65	–
	Disodium phosphate	3.90 to 4.80	30 to 45
	Iodine	48.55	–
	Oils in bottle or barrels	5.70 to 8.90	–
	Oil, linsee:		
	In barrels	5.70	–
	In drums	7.05	–
	Oil, turpentine	8.50	–
	Paints	9.40	–
	Paraffin wax	7.85 to 9.40	–
	Petroleum	9.90	–
	Phosphorus	17.85	–
	Plastics and polymers:		
	Cellulose acetate	12.25 to 13.35	–
	Cellulose nitrate	13.25 to 15.70	–
	Methyl methacrylate (IS 14753) [C-1(15)]	11.57 to 11.77	–
	Phenol formaldehyde	12.55	–
	Expanded Polystyrene (IS 4671) [C-1(15)]	0.15 to 0.35	–
	Extruded polystyrene	0.21	
	Polyvinyl chloride (Perspex)	11.75 to 13.25	–
	Resin bonded sheet	12.85 to 13.55	–
	Urea formaldehyde	13.25 to 13.55	–
	Extruded polystyrene	0.21	
	Polyurethane	0.59 to 0.66	
	Polyethylene (IS 2508) [C-1(15)]	9.81 to 19.6	
	Polyvinyl Butaryl	11	
	Polycarbonate (IS 14443) [C-1(15)]	12	
	Ethylene tetrafluoroethylene	17	
	Polytetrafluoroethylene [IS 14635 (part 2)] [C-1(15)]	22	
	Potash	14.40	–
	Potassium	8.65	–

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Table 41 (*Continued*)

Sl No.	Materials	Weight KN/m^3	Angle of Friction, Degrees
(1)	(2)	(3)	(4)
	Potassium nitrate	9.90	–
	Red lead, dry	20.70	–
	Red lead, paste	87.30	–
	Rosin in barrels	6.75	–
	Rubber:		
	Raw	8.90 to 9.40	–
	Vulcanized	8.90 to 9.10	–
	Saltpeter	9.91	–
	Sodium silicate in barrels	8.35	–
	Sulphur	20.10	–
	Talc	27.45	–
	Varnishes	9.40	–
	Vitriol, blue, in barrels	7.05	–
iii)	**Fuels**		
	Brown coal	6.83	–
	Brown coal briquettes heaped	7.85	35
	Brown coal briquettes stacked	12.75	–
	Charcoal	2.95	–
	Coal:		
	Untreated, mine-moist	9.80	35
	In washeries	11.75	0
	Dust	6.85	25
	All other sorts	8.35	35
	Coke:		
	Furnace or gas	4.90	35
	Brown coal, low-temperature	9.80	35
	Hard, raw coal	8.35	35
	Hard, raw coal, mine-damp	9.80	35
	Diesel oil	9.40	0
	Firewood, chopped	3.90	45
	Petrol	6.75	0
	Wood in chips	1.95	45
	Wood shaving, loose	1.45	35
	Wood shaving, shaken down	2.45	35
iv)	**Manures**		
	Animal manures:		
	Loosely heaped	11.75	45
	Stacked dung up to 25 m stack height	17.65	45
	Artificial manures	11.75	24 to 30
v)	**Metals and Alloys**		

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Table 41 (*Continued*)

Sl No.	Materials	Weight KN/m^3	Angle of Friction, Degrees
(1)	(2)	(3)	(4)
	Aluminium	25.30 to 26.60	
	Cast	25.30 to 26.60	–
	Wrought	25.90 to 27.45	–
	Sheet per mm of thickness per m^2	0.028	–
	Antimony, pure:		
	Amorphous	60.90	–
	Solid	65.70	–
	Bismuth:		
	Liquid	98.07	–
	Solid	95.02 to 97.9	–
	Cadmium:		
	Cast	83.75 to 84.05	–
	Wrought	85.03	–
	Calcium	15.60	–
	Chromium	63.95 to 66.00	–
	Cobalt:		
	Cast	83.25 to 85.10	–
	Wrought	88.45	–
	Copper:		
	Cast	86.20 to 87.65	–
	Wrought	86.70 to 87.65	–
	Sheet per mm of thickness	0.09	–
	Gold:		
	Cast	188.75 to 189.55	–
	Wrought	189.55	–
	Iron:		
	Pig	70.60	–
	Grey, cast	68.95 to 69.90	–
	White, cast	74.35 to 75.70	–
	Wrought	75.50	–
	Lead:		
	Cast	111.20	–
	Liquid	105.00	–
	Wrought	111.40	–
	Sheet per mm of thickness	0.11	–
	Magnesium	16.45 to 17.15	–
	Manganese	72.55	–
	Mercury	133.35	–
	Nickel	81.20 to 87.20	–
	Platinum	210.25	–

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Table 41 (*Continued*)

Sl No.	Materials	Weight KN/m^3	Angle of Friction, Degrees
(1)	(2)	(3)	(4)
	Silver:		
	Cast	102.0 to 102.85	–
	Liquid	93.15	–
	Wrough	103.35 to 103.55	–
	Sodium:		
	Liquid	9.10	–
	Solid	9.30	–
	Tungsten	188.30	–
	Uranium	180.45	–
	Zinc		
	Cast	68.95 to 70.20	–
	Wrought	70.50	–
	Sheet per mm of thickness	0.07	–
	Alloys:		
	Aluminium and copper		
	Aluminium 10 percent copper 90 percent	75.40	–
	Aluminium 5 percent copper 95 percent	82.00	–
	Aluminium 3 percent copper 97 percent	85.10	–
	Aluminium 91 percent zinc 9 percent	27.45	–
	Babbit metal tin 90 percent	71.70	–
	Lead 5 percent copper 5 percent		
	Wood's metal bismuth 50 percent	95.00	–
	Lead 25 percent cadmium 12.5 percent		
	Tin 12.5 percent		
	Brasses:		
	Muntz metal (copper 60 percent, zinc 40 percent)	80.60	–
	Red (copper 90 percent, zinc 10 percent)	84.25	–
	White (copper 50 percent, zinc 50 percent)	80.30	–
	Yellow (copper 70 percent, zinc 30 percent)		
	Cast	82.75	–
	Drawn	85.10	–
	Rolled	83.85	–
	Bronzes:		
	Bell metal (copper 80 percent, tin 20 percent)	85.60	–

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Table 41 (*Continued*)

Sl No.	Materials	Weight KN/m^3	Angle of Friction, Degrees
(1)	(2)	(3)	(4)
	Gun metal (copper 90 percent, tin 10 percent)	86.10	–
	Cadmium and tin	75.40	–
	German Silver:		
	Copper 52 percent, zinc 26 percent, nickel 22 percent	82.75	–
	Copper 59 percent, zinc 30 percent, nickel 11 percent	81.70	–
	Copper 63 percent, zinc 30 percent, nickel 7 percent	81.40	–
	Gold and Copper:		
	Gold 98 percent, copper 2 percent	184.75	–
	Gold 90 percent, copper 10 percent	168.20	–
	Lead and Tin:		
	Lead 87.5 percent, tin 12.5 percent	103.85	–
	Lead 30.5 percent, tin 69.5 percent	81.10	–
	Monel metal cast (nickel 70 percent, copper 30 percent)	87.00	–
	Steel:		
	Cast	77.00	–
	Wrought	76.30	–
	Black plate per mm of thickness	0.08	–
	Steel sections (*see* 46 'steel sections' in Table 1)		
vi)	**Miscellaneous Materials**		
	Aggregate, coarse	10.80 to 15.70	30
	Ashes, coal, dry, 12 mm and under	5.50 to 6.30	40
	Ashes, coal, dry, 75 mm and under	5.50 to 6.30	38
	Ashes, coal, wet 12 mm and under	7.05 to 7.85	52
	Ashes, coal, wet 75 mm and under	7.05 to 7.85	50
	Asphalt, crushed 12 mm and under	7.05	30 to 45
	Ammonium nitrate, polls	3.55 to 8.35	27
	Bone	18.65	–
	Books and files, stacked	8.35	–
	Calcium ammonium nitrate	9.80	28
	Copper sulphate, ground	11.75	30
	Chalk	21.95	–
	Chinaware, earthenware, stacked (including cavities)	10.80	–
	Clinker, furnace, clean	7.85	30
	Diammonium phosphate	7.85 to 8.50	29

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Table 41 (*Continued*)

Sl No.	Materials	Weight KN/m^3	Angle of Friction, Degrees
(1)	(2)	(3)	(4)
	Double salt (ammonium sulphate nitrate)	7.05 to 9.30	34
	Filling cabinet and cupboards with contents in records offices, libraries, archives	5.90	–
	Flue dust, boiler house, dry	5.50 to 7.05	30
	Fly ash, pulverised	5.50 to 7.05	–
	Glass, solid	23.50 to 26.70	–
	Wool	0.16 to 1.18	–
	In sheets	25.50	–
	Glue	12.55	–
	Gypsum, calcined, 12mm and under	8.60 to 9.40	40
	Gypsum, calcined, powdered	9.40 to 12.55	45
	Gypsum, raw, 25 mm and under	14.10 to 15.70	30 to 45
	Hides:		
	Dry } Only green	8.65	–
	Salted } Only green	8.90	–
	Ice } Only green		
	Leather put in rows	7.85	–
	Lime, ground 3 mm and under	9.40	> 45
	Lime, hydrated 3 mm and under	6.30	30 to 45
	Lime, hydrated, pulverized	5.00 to 6.30	30 to 45
	Lime pebble	8.25 to 8.75	
	Limestone, agricultural 3 mm and under	10.60	
	Limestone, crushed	13.30 to 14.10	30 to 45
	Limestone dust	8.65 to 14.90	30 to 45
	Magnesite, caustic, in powder form	7.85	–
	Magnesite sinter and Magnesite granular	19.60	–
	Phosphate, rock, pulverized	9.40	40 to 52
	Phosphate rock	11.75 to 13.35	30 to 45
	Phosphate sand	14.10 to 15.70	30 to 45
	Potassium carbonate	7.95	30 to 45
	Potassium chloride, pellets	18.85 to 20.40	30 to 45
	Potassium nitrate	4.85	> 30
	Potassium sulphate	6.55 to 7.45	45
	Pyrites, pellets	18.85 to 20.40	30 to 45
	Pumice	5.80 to 9.90	–
	Rubbish:		
	Building	13.80	–
	General	6.30	–
	Salt, common, dry, coarse	6.30 to 10.00	30 to 45
	Salt, common, dry, fine	11.00 to 12.55	30 to 45

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Table 41 (*Concluded*)

Sl No.	Materials	Weight KN/m^3	Angle of Friction, Degrees
(1)	(2)	(3)	(4)
	Salt cake, dry, coarse	13.35	30
	Salt cake, dry, pulverized	11.20 to 13.35	35
	Sand, bank. damp	17.25 to 20.40	45
	Sand, bank, dry	14.10 to 17.25	30
	Sand, silica, dry	14.10 to 15.70	30 to 45
	Saw dust	1.57	30
	Silica gel	4.40	30 to 45
	Soda ash, heavy	8.67 to 10.20	35
	Soda ash, light	4.70 to 6.00	37
	Sodium nitrate, granular	11.00 to 12.55	24
	Sulphur, crushed, 12 mm and under	7.85 to 8.25	35 to 45
	Sulphur, 76 mm and under	8.65 to 13.35	32
	Sulphur, powdered	7.85 to 9.40	30 to 45
	Single superphosphate (S.S.P), granulated	7.65 to 8.25	37
	Slag, furnace, crushed	14.90	35
	Steel goods:		
	Cylinders, usually stored for Carbonic acid, etc	13.80	
	Sheets, railway rails, etc, usually stored	44.00	
	Trisodium phosphate	9.40	30 to 45
	Triple superphosphate	7.85 to 8.65	30 to 45
	Turf	2.85 to 5.70	–
	Urea, prills	6.40	23 to 26
vii)	**Ores**		
	Antimony	29.80	–
	Ferrous sulphide	26.50	–
	Ferrous sulphide ore waste after roasting	13.85	–
	Iron ore, compact storing	29.80	–
	Magnesium ore	19.60	–
viii)	**Textiles, Paper and Allied Materials**		
	Cellulose in bundles	7.35	–
	Cotton, compressed	12.75	–
	Flax, piles and compressed in bales	2.95	–
	Furs	8.80	–
	Jute in bundles	6.85	–
	Paper:		
	In bundles and rolls	6.85	–
	Newspaper in bundles	3.90	–
	Put in rows	10.80	–
	Thread in bundles	4.90	–
	Wood, compressed	12.75	–

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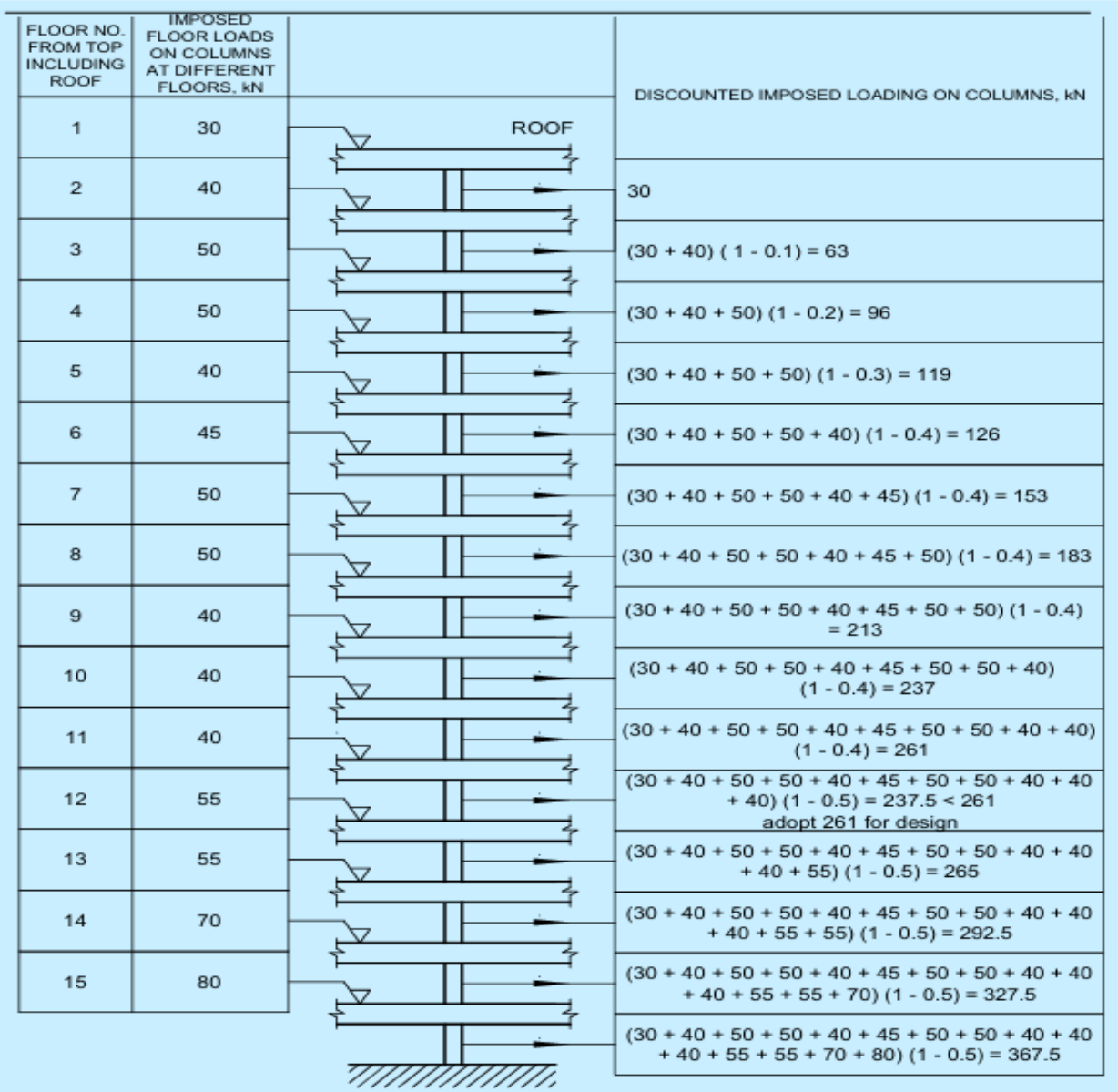
ANNEX B

[*Clause* 3.3.2.1(b)]

ILLUSTRATIVE EXAMPLE SHOWING REDUCTION OF UNIFORMLY DISTRIBUTED IMPOSED FLOOR LOADS IN MULTI-STOREYED BUILDINGS FOR DESIGN OF COLUMNS

B-1 The total imposed loads from different floor levels (including the roof) coming on the central column of a multi-storeyed building (with mixed occupancy) is shown in Fig. 14. Calculate the reduced imposed load for the design of column members at different floor levels using **3.3.2.1**. Floor loads do not exceed 5.0 kN/m^2.

B-1.1 Applying reduction coefficients in accordance with **3.3.2.1**, total reduced floor loads on the column at different levels is indicated along with Fig. 14.



FLOOR NO. FROM TOP INCLUDING ROOF	IMPOSED FLOOR LOADS ON COLUMNS AT DIFFERENT FLOORS, kN	DISCOUNTED IMPOSED LOADING ON COLUMNS, kN
1	30	
2	40	30
3	50	(30 + 40) (1 - 0.1) = 63
4	50	(30 + 40 + 50) (1 - 0.2) = 96
5	40	(30 + 40 + 50 + 50) (1 - 0.3) = 119
6	45	(30 + 40 + 50 + 50 + 40) (1 - 0.4) = 126
7	50	(30 + 40 + 50 + 50 + 40 + 45) (1 - 0.4) = 153
8	50	(30 + 40 + 50 + 50 + 40 + 45 + 50) (1 - 0.4) = 183
9	40	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50) (1 - 0.4) = 213
10	40	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40) (1 - 0.4) = 237
11	40	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40 + 40) (1 - 0.4) = 261
12	55	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40 + 40 + 40) (1 - 0.5) = 237.5 < 261 adopt 261 for design
13	55	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40 + 40 + 40 + 55) (1 - 0.5) = 265
14	70	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40 + 40 + 40 + 55 + 55) (1 - 0.5) = 292.5
15	80	(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40 + 40 + 40 + 55 + 55 + 70) (1 - 0.5) = 327.5
		(30 + 40 + 50 + 50 + 40 + 45 + 50 + 50 + 40 + 40 + 40 + 55 + 55 + 70 + 80) (1 - 0.5) = 367.5

FIG. 14 LOADING DETAILS

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ANNEX C

(*Clauses* 4.2)

NOTATIONS

C-1 The following notations to be followed unless otherwise specified in relevant clauses under wind loads:

A	=	Surface area of a structure or part of a structure
A_e	=	Effective frontal area
A_z	=	Effective frontal area of the building at height z
b	=	Breadth of a structure or structural member normal to the wind stream in the horizontal plane
B_s	=	Background factor
C_d	=	Drag coefficient
C_f	=	Force coefficient
C_{fn}	=	Normal force coefficient
C_{ft}	=	Transverse force coefficient
C_f'	=	Frictional drag coefficient
C_p	=	Pressure coefficient
C_{pe}	=	External pressure coefficient
C_{pi}	=	Internal pressure coefficient
C_{fs}	=	Cross-wind force spectrum coefficient
$C_{f,z}$	=	Drag force coefficient of the building corresponding to the area A_z
C	=	Coefficient, which depends on θ_s, used in the evaluation of k_3 factor
d	=	Depth of a structure or structural member parallel to wind stream in the horizontal plane
d_w	=	Wake width
D	=	Diameter of cylinder or sphere
E	=	Wind energy factor
F_z	=	Along wind load on the building/structure at any height z
F	=	Force normal to the surface
f_a	=	First mode natural frequency of the building/structure in along wind direction, in Hz
f_c	=	First mode natural frequency of the building/structure in across wind direction, in Hz
f_s	=	Vortex shedding frequency
F_n	=	Normal force
F_t	=	Transverse force
F'	=	Frictional force
G	=	Gust factor
g_R	=	Peak factor for resonant response
g_v	=	Peak factor for upwind velocity fluctuations
h	=	Height of structure above mean ground level
h_x	=	Height of development of a velocity profile at a distance x down wind from a change in terrain category
H_s	=	Height factor for resonant response

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H = Height above mean ground level on the topography feature

I = Turbulence intensity

$I_{h,i}$ = Turbulence intensity at height h in terrain category i

$I_{z,i}$ = Turbulence intensity at height z in terrain category i

IF = Interference factor

k = Mode shape power exponent

k_1, k_2, k_3, k_4 = Wind speed modification factors

$\overline{k}_{2,i}$ = Hourly mean wind speed factor

K = Force coefficient multiplication factor for individual members of finite length

K_a = Area averaging factor

K_c = Combination factor

K_d = Wind directionality factor

l = Length of the member or larger horizontal dimension of a building

L = Actual length of upwind slope

L_e = Effective length of upwind slope

L_h = Integral turbulence length scale at the height, h

m_0 = Average mass per unit height of the structure

M_a = Design peak along wind base bending moment

M_c = Design peak across wind base bending moment

N = Effective reduced frequency

p_d = Design wind pressure

p_z = Wind pressure at height z

$\overline{P}_d$ = Design hourly mean wind pressure corresponding to $\overline{V}_{z,d}$

p_e = External pressure

p_i = Internal pressure

r = Roughness factor which is twice the longitudinal turbulence intensity at height h, $I_{h\,i}$

R_e = Reynolds number

s = Level on a building/structure for the evaluation of along wind load effects

s_0 = Factor, which depends on H and X, used for the evaluation of k_3 factor

S_t = Strouhal number

S = Size reduction factor

V_b = Regional basic wind speed

V_z = Design wind speed at height z

$\overline{V}_d$ = Design hourly mean wind speed

$\overline{V}_{z,d}$ = Design hourly mean wind speed at height z

$\overline{V}_{z,H}$ = Hourly mean wind speed at height z

w = Lesser horizontal dimension of a building, or a structural member

w' = Bay width in multi-bay building

$\hat{\ddot{x}}$ = Peak acceleration at the top of the building/structure in along wind direction, in m/s^2;

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x	=	Distance down wind from a change in terrain category
X	=	Distance from the summit or crest of topography feature relative to the effective length, L_e
$\hat{\hat{y}}$	=	Peak acceleration at the top of the building/structure in across wind direction
z	=	Height or distance above the ground
$z_{0,i}$	=	Aerodynamic roughness height for i^{th} terrain
Z	=	Effective height of the topography feature
α	=	Inclination of the roof to the horizontal
β	=	Damping coefficient of the building/structure
η	=	Shielding factor
ϕ	=	Factor to account for the second order turbulence intensity
Φ	=	Solidity ratio
Φ_e	=	Effective solidity ratio
ε	=	Average height of the surface roughness
θ_s	=	Upwind slope of the topography feature in the wind direction
θ	=	Wind angle from a given axis

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ANNEX D

(*Clause* 4.4.2)

BASIC WIND SPEED AT 10 m HEIGHT FOR SOME IMPORTANT CITIES/TOWNS

City/Town	Basic Wind Speed m/s	City/Town	Basic Wind Speed m/s
Agra	47	Kanpur	47
Ahmedabad	39	Kohima	44
Ajmer	47	Kolkata	50
Almora	39	Kozhikode	39
Amritsar	50	Kurnool	39
Asansol	47	Lakshadweep	39
Aurangabad	39	Lucknow	50
Bahraich	50	Ludhiana	50
Bengaluru	33	Madurai	39
Barauni	47	Mandi	39
Bareilly	50	Mangaluru	39
Bhatinda	47	Moradabad	50
Bhilai	44	Mumbai	44
Bhopal	47	Mysuru	33
Bhubaneshwar	50	Nagpur	44
Bhuj	50	Nainital	47
Bikaner	47	Nasik	39
Bokaro	39	Nellore	50
Chandigarh	50	Panaji	39
Chennai	50	Patiala	50
Coimbatore	39	Patna	47
Cuttack	50	Puducherry	50
Darbhanga	47	Port Blair	44
Darjeeling	47	Pune	39
Dehra Dun	39	Raipur	44
Delhi	50	Rajkot	39
Durgapur	47	Ranchi	39
Gangtok	47	Roorkee	50
Guwahati	50	Rourkela	39
Gaya	39	Simla	39
Gorakhpur	47	Srinagar	39
Hyderabad	44	Surat	44
Imphal	44	Tiruchirrappalli	47

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City/Town	Basic Wind Speed m/s	City/Town	Basic Wind Speed m/s
Jabalpur	39	Thiruvananthapuram	39
Jaipur	47	Udaipur	47
Jamshedpur	47	Vadodara	39
Jhansi	47	Varanasi	47
Jodhpur	47	Vijaywada	50
		Visakhapatnam	50

ANNEX E

[*Clause* 4.4.3.2.4(b)(2)]

CHANGES IN TERRAIN CATEGORIES

E-1 LOW TO HIGH TERRAIN CATEGORY NUMBER

In cases of transition from a low terrain category number (corresponding to a low terrain roughness) to a higher terrain category number (corresponding to a rougher terrain), the velocity profile over the rougher terrain shall be determined as follows:

a) Below height h_x, the velocities shall be determined in relation to the rougher terrain; and

b) Above height h_x, the velocities shall be determined in relation to the less rough (more distant) terrain.

E-2 HIGH TO LOW TERRAIN CATEGORY NUMBER

In cases of transition from a more rough to a less rough terrain, the velocity profile shall be determined as follows:

a) Above height h_x, the velocities shall be determined in accordance with the rougher (more distant) terrain; and

b) Below height h_x, the velocity shall be taken as the lesser of the following:

1) That determined in accordance with the less rough terrain; and

2) The velocity at height h_x as determined in relation to the rougher terrain.

NOTE — Examples of determination of velocity profiles in the vicinity of a change in terrain category are shown in Fig. 15(A) and Fig. 15(B).

E-3 MORE THAN ONE CATEGORY

Terrain changes involving more than one category shall be treated in similar way to that described in **E-1** and **E-2**.

NOTE — Examples involving three terrain categories are shown in Fig. 15(B).

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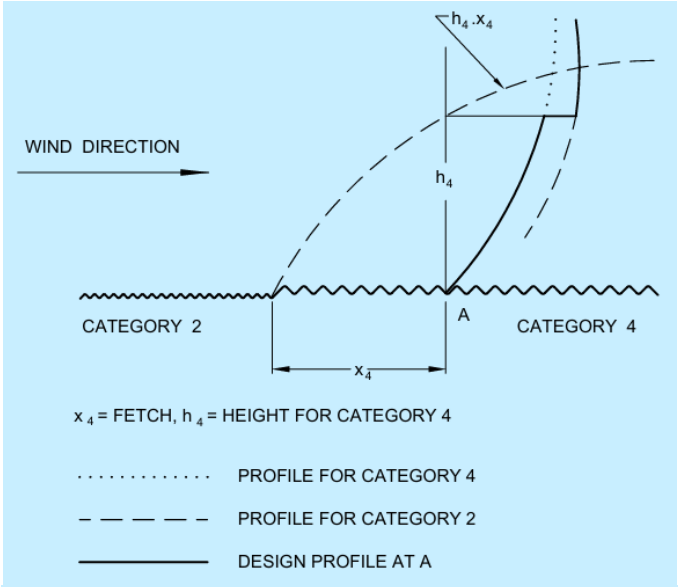

FIG. 15(A) DETERMINATION OF VELOCITY PROFILE NEAR A CHANGE IN TERRAIN CATEGORY (Less rough to more rough)

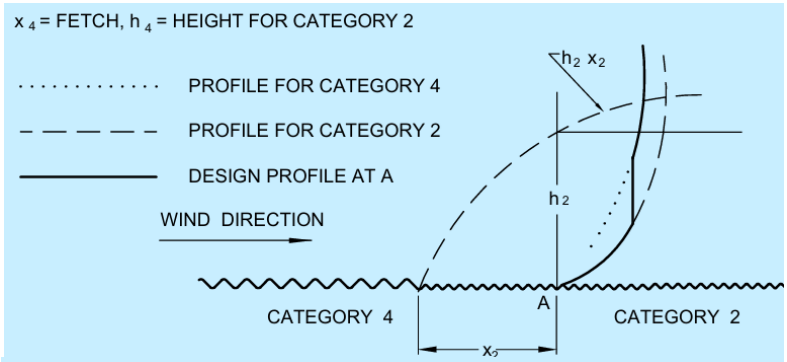

FIG. 15(B) DETERMINATION OF VELOCITY PROFILE NEAR A CHANGE IN TERRAIN CATEGORY (More rough to less rough)

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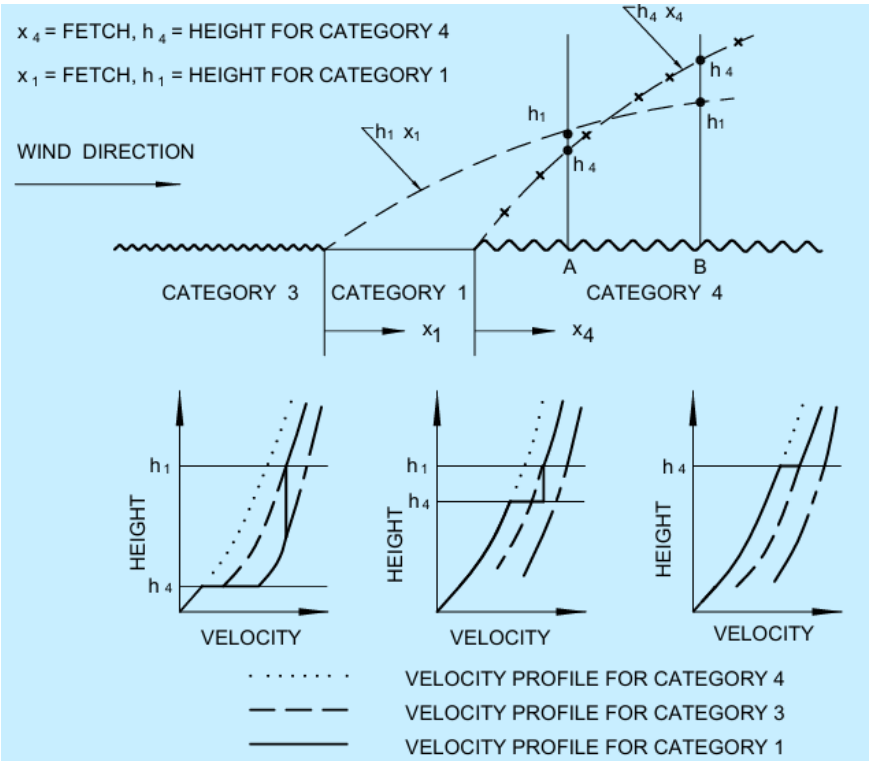

FIG. 15(C) DETERMINATION OF DESIGN PROFILE INVOLVING MORE THAN ONE CHANGE IN TERRAIN CATEGORY

FIG. 15 VELOCITY PROFILES IN THE VICINITY OF A CHANGE IN TERRAIN CATEGORY

ANNEX F

(*Clause* 4.4.3.3)

EFFECT OF A CLIFF OR ESCARPMENT ON EQUIVALENT HEIGHT ABOVE GROUND (k_3 FACTOR)

F-1 The influence of the topographic feature is considered to extend 1.5 L_e upwind and 2.5 L_e downwind of the summit of crest of the feature where *L*e is the effective horizontal length of the hill depending on slope as indicated below (*see* Fig. 16):

Slope	L_e
$3° < \theta_s \leq 17°$	L
$\theta_s > 17°$	$Z / 0.3$

where

L = actual length of the upwind slope in the wind direction,

Z = effective height of the topography feature and

θ_s = upwind slope in the wind direction.

In case, the zone in downwind side of the crest of the feature is relatively flat ($\theta_s < 3°$) for a distance exceeding L_e, then the feature should be treated as an escarpment. Otherwise, the feature should be treated as a hill or ridge. Examples of typical features are given in Fig. 16.

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NOTES

1 No difference is made, in evaluating k_3 between a three dimensional hill and two dimensional ridge.

2 In undulating terrain, it is often not possible to decide whether the local topography to the site is significant in terms of wind flow. In such cases, the average value of the terrain upwind of the site for a distance of 5 km should be taken as the base level from wind to assess the height, Z and the upwind slope θ, of the feature.

F-2 TOPOGRAPHY FACTOR, k_3

The topography factor k_3 is given by the following:

$$k_3 = 1 + C\, s_0$$

where C has the following values:

Slope	*C*
$3° < \theta_s \leq 17°$	1.2 (Z/L)
$\theta_s > 17°$	0.36

and s_0 is a factor derived in accordance with **F-2.1** appropriate to the height, H above mean ground level and the distance, X, from the summit or crest relative to the effective length, L_e.

F-2.1 The factor, s_0 should be determined from:

a) Fig. 17 for cliffs and escarpments; and

b) Fig.18 for ridges and hills.

NOTE — Where the downwind slope of a hill or ridge is more than 3°, there will be large regions of reduced accelerations or even shelter and it is not possible to give general design rules to cater for these circumstances. Values of s_0 from Fig. 18 may be used as upper bound values.

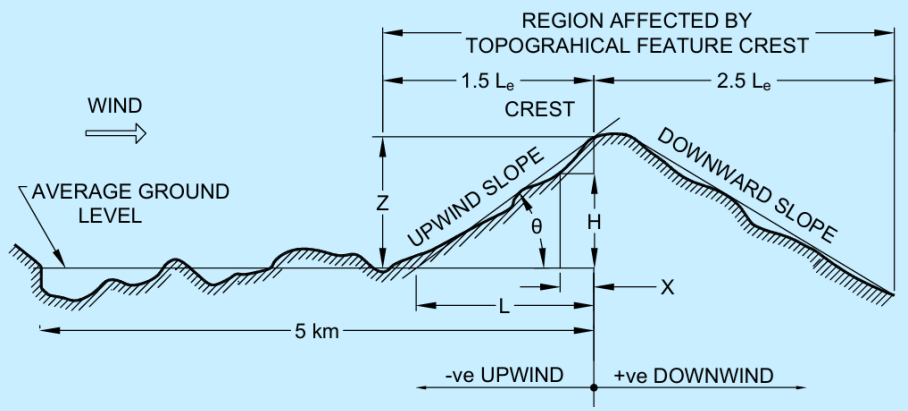
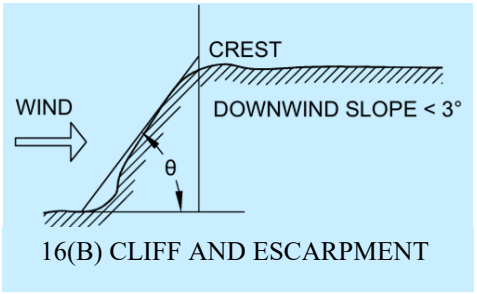
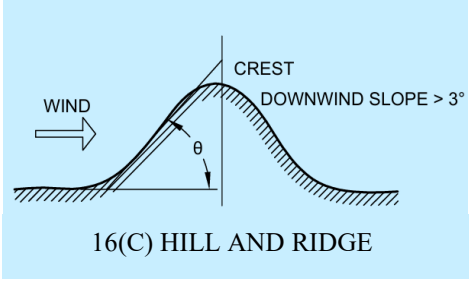


FIG. 16(A) GENERAL NOTATIONS



16(B) CLIFF AND ESCARPMENT



16(C) HILL AND RIDGE

FIG. 16 TOPOGRAPHICAL DIMENSIONS

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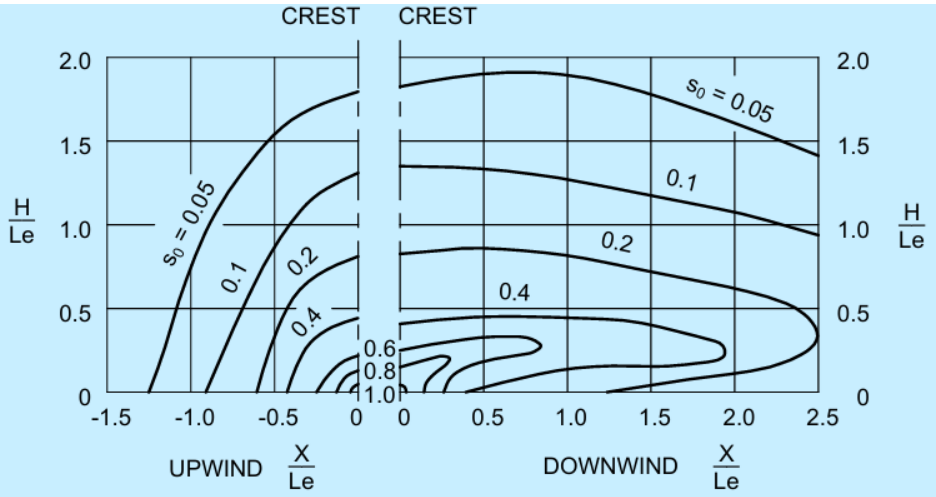

FIG. 17 FACTOR s_0 FOR CLIFF AND ESCARPMENT

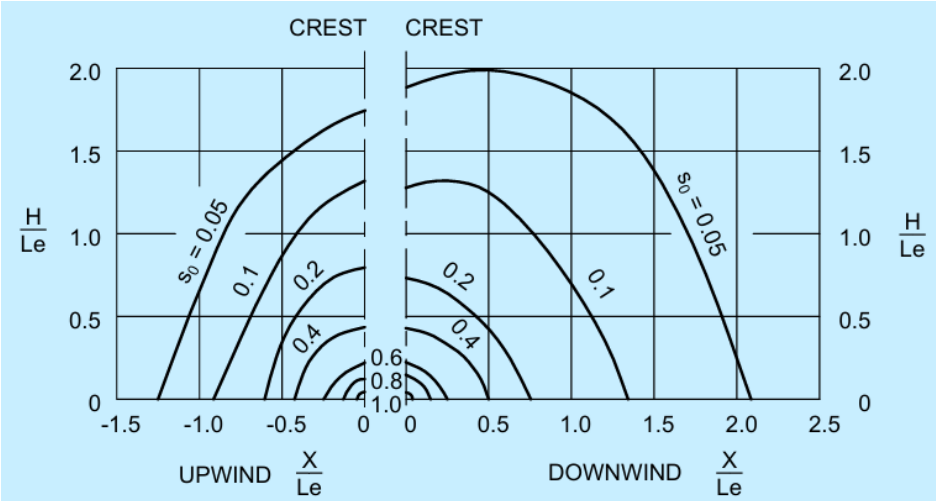

FIG. 18 FACTOR s_0 FOR RIDGE AND HILL

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ANNEX G

[*Clauses* 4.5.4.2.2, 4.5.4.3.2(c) *and* 4.5.4.3.3]

WIND FORCE ON CIRCULAR SECTIONS

G-1 The wind force on any object is given by:

$$F = C_f A_e p_d$$

where

C_f = force coefficient;
A_e = effective area of the object normal to the wind direction; and
p_d = design pressure of the wind.

For most shapes, the force coefficient remains approximately constant over the whole range of wind speeds likely to be encountered. However, for objects of circular cross section, it varies considerably.

For a circular section, the force coefficient depends on the way in which the wind flows around it and is dependent upon the velocity and kinematic viscosity of the wind and diameter of the section. The force coefficient is usually quoted against a non-dimensional parameter, called the Reynolds number, which takes into account of the velocity and viscosity of the flowing medium (in this case the wind) and the member diameter.

$$\text{Reynolds number, } R_e = \frac{D\bar{V}_d}{\nu}$$

where

D = diameter of the member;
$\bar{V}_d$ = design hourly mean wind speed; and
ν = kinematic viscosity of the air which is 1.46×10^{-5} m^2/s at 15° and standard atmospheric pressure.

Since in most natural environments likely to be found in India, the kinematic viscosity of the air is fairly constant, it is convenient to use $D\bar{V}_d$ as the parameter instead of Reynolds number and this has been done in this Section.

The dependence of a circular section's force coefficient on Reynolds number is due to the change in the wake developed behind the body.

At a low Reynolds number, the wake is as shown in Fig. 19 and the force coefficient is typically 1.2. As Reynolds number is increased, the wake gradually changes to that shown in Fig. 20, that is, the wake width d_w decreases and the separation point, denoted as s_p, moves from front to the back of the body.

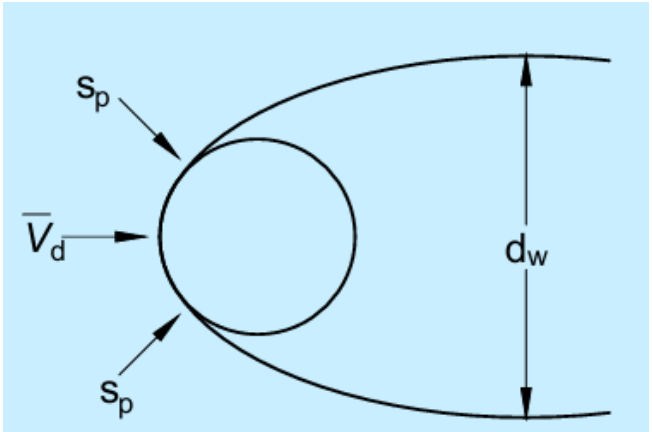

FIG. 19 WAKE IN SUB-CRITICAL FLOW

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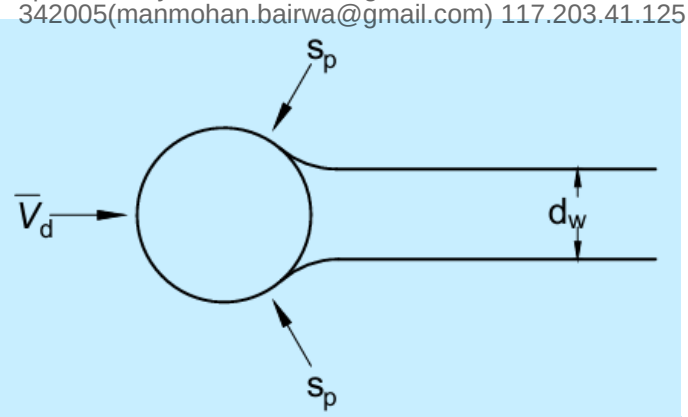

FIG. 20 WAKE IN SUPER-CRITICAL FLOW

As a result, the force coefficient shows a rapid drop at a critical value of Reynolds number followed by a gradual rise as Reynolds number is increased still further.

The variation of C_f with parameter $D\overline{V}_d$ is shown in Fig. 5 for infinitely long circular cylinders having various values of relative surface roughness (ε/D) when subjected to wind having an intensity and scale of turbulence typical of built-up urban areas. The curve for a smooth cylinder (ε/D) = 1×10^{-5} in a steady air stream, as found in a low-turbulence wind tunnel, is also shown for comparison.

It can be seen that the main effect of free-stream turbulence is to decrease the critical value of the parameter $D\overline{V}_d$. For subcritical flows, turbulence can produce a considerable reduction in C_f below the steady air-stream values. For supercritical flows, this effect becomes significantly smaller.

If the surface of the cylinder is deliberately roughened, such as by incorporating flutes, riveted construction, etc, then the data given in Fig. 5 for appropriate value of $\varepsilon/D > 0$ shall be used.

NOTE — In case of uncertainty regarding the value of ε to be used for small roughness, ε/D shall be taken as 0.001.

ANNEX H

(*Foreword* and clause 7.3.2)

CHARACTERISTIC GROUND SNOW LOAD

H-1 This annex presents the characteristic ground snow load computation procedure for the union territory of Jammu and Kashmir, union territory of Ladakh, Himachal Pradesh, Uttarakhand and Sikkim which is the result of scientific work carried out by Defence Geoinformatics Research Establishment (DGRE), Chandigarh under Defence Research and Development Organization (DRDO).

H-2 The characteristic value of the ground snow load at any site is defined with a 3.4 percent annual exceedance probability or, equivalently, with a mean recurrence interval, MRI = 30 years. The site altitude is the altitude at which the structure is or will be located, measured from mean sea level. The procedure shall be used for the computation of characteristic ground snow load on sites with an altitude higher than 1000 m and lower than 4000 m from mean sea level.

H-3 The characteristic ground snow load, s_0 (kPa) to be used for any site in the union territory of Jammu and Kashmir, union territory of Ladakh, Himachal Pradesh, Uttarakhand should be obtained from the zone map shown in Fig. 21 and equation given below:

$$s_0 = 1.624 \times [Z - 0.5] \times \left[1 + \left(\frac{A}{2677}\right)^2\right] \text{kPa}$$

where

s_0 = characteristic ground snow load (kPa);

Z = zone number (for example 1, 2, 3, 4 or 5) obtained from the zone; and

A = map in Fig. 21.

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H-4 The characteristic ground snow load s_0 (kPa) to be used for any site in Sikkim should be directly obtained from the characteristic ground snow load map shown in Fig. 22.

H-5 The ground snow loads for any mean recurrence interval of n years, different to that for the characteristic snow load, s_0, (which by definition is based on annual probability of exceedance of 0.034 that is MRI = 30 years) shall be calculated by the equation given below:

$$s_n = s_0 \left\{ \frac{1 - C_v \frac{\sqrt{6}}{\pi} \left[ln(-ln(1-P_n)) + 0.577\ 22 \right]}{(1 + 2.188\ 7 C_v)} \right\}$$

where

s_0 = characteristic snow load on the ground with MRI = 30 years

s_n = characteristic snow load on the ground with MRI = 30 years

P_n = annual probability of exceedance (equivalent to $1/n$, where n is the corresponding recurrence interval in years)

C_v = coefficient of variation of annual maximum snow load.

H-5.1 The coefficient of variation of maximum snow load at different snow-meteorological observatories of DGRE located in union territory of J and K, union territory of Ladakh, HP and UK ranges between 0.35 to 0.65. For application of above equation, a constant value of $C_v = 0.5$ is recommended for all the zones to calculate snow loads on the ground for other probability of exceedance (for example, MRI of 50 years for structures where a higher risk of exceedance is deemed acceptable). The above equation is shown graphically in Fig. 23.

NOTES

1 The characteristic ground snow load *versus* altitude relation as given above in equation and Fig. 21 is determined based on the snow depth observations at more than 70 field observatories of DGRE located in different parts of union territory of Jammu and Kashmir, union territory of Ladakh and states of Himachal Pradesh and Uttarakhand as well as High Asia Refined analysis (HAR10) dataset (HAR, Maussion et al., 2014).

2 Since adequate snow depth measurements are not available for Sikkim region, hence the characteristic ground snow load map for Sikkim given in Fig. 22 is generated based solely on high Asia refined analysis (HAR10) dataset (HAR, Maussion et al., 2014).

3 Annex H does not apply for sites at altitudes above 4000 m. For altitudes greater than 4000 m specialist advice should be sought from DGRE on the characteristic ground snow loads likely to occur at the site.

4 Unusual local effects have not been accounted for in the analysis undertaken to produce the characteristic ground snow load map given in Annex H. These include local shelter from the wind, which can result in increased local snow loads and local configurations in mountainous areas, which may funnel the snow and give increased local loading. Annex H is also not applicable for sites with permanent snow/ice cover throughout the year. If the designer suspects that there are unusual local conditions that need to be taken into account, DGRE may be consulted.

5 In special cases where more refined data is needed, the characteristic value of snow load on the ground (s_0) may be refined using an appropriate statistical analysis of long term records taken in a well sheltered area near the site.

6 Ground characteristics snow load at any place depends on the critical combination of maximum depth of undisturbed aggregate cumulative snowfall and its density. In due course the characteristics snow load for states of Arunachal Pradesh will be included based on studies. Till such time the users of this section are advised to consult DGRE for the specific information relating to the state of Arunachal Pradesh.

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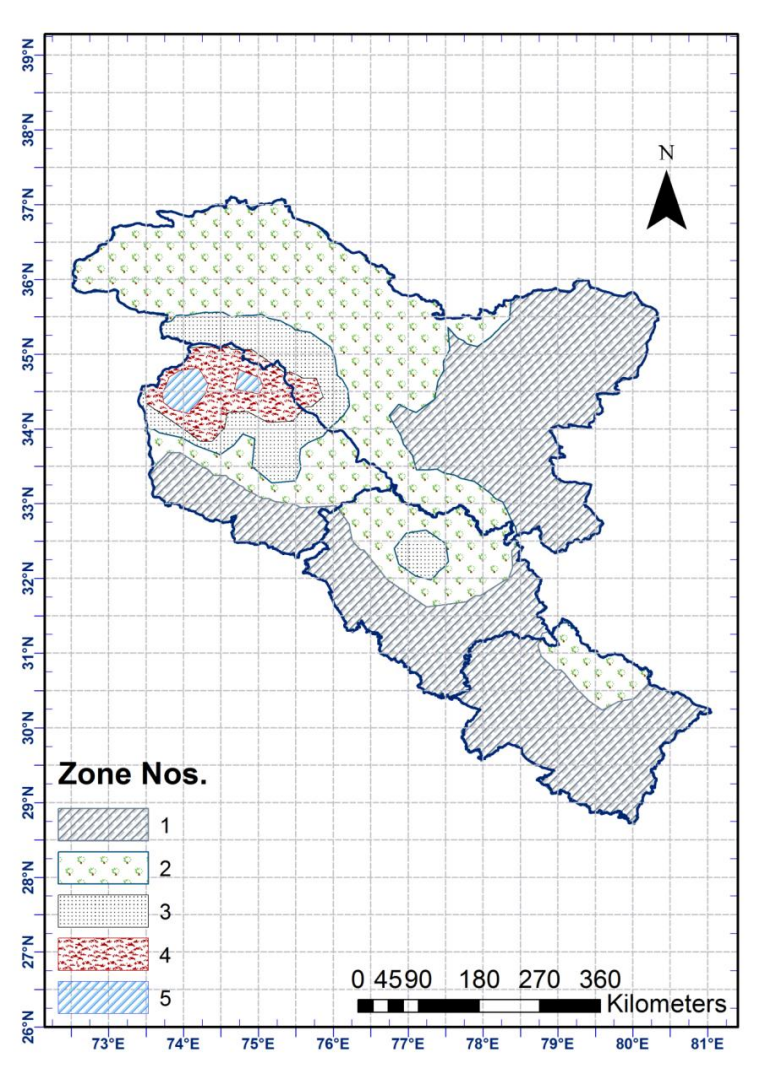

FIG. 21 ZONE MAP FOR J AND K, LADAKH, HP AND UK AT 10 KM RESOLUTION

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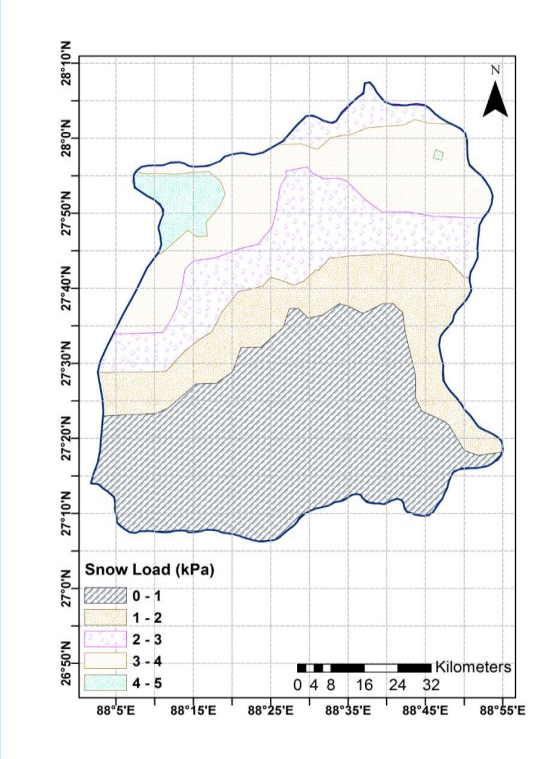

FIG. 22 CHARACTERISTIC GROUND SNOW LOAD MAP FOR SIKKIM AT 10 KM RESOLUTION

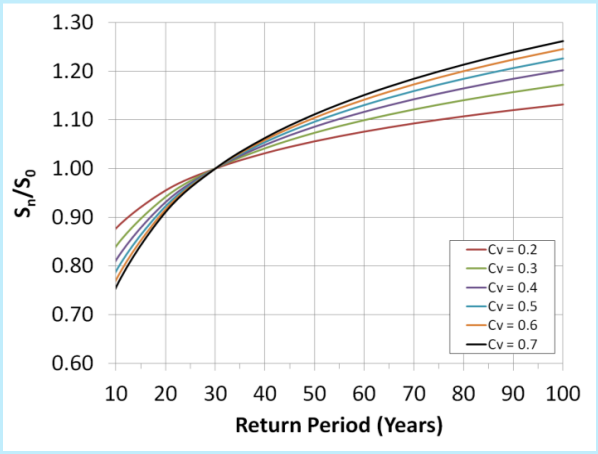

FIG. 23 ADJUSTMENT OF THE GROUND SNOW LOAD ACCORDING TO THE RETURN PERIOD

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ANNEX J

(*Clauses* 7.4.2.4 *and* 7.4.3)

SHAPE COEFFICIENTS FOR MULTILEVEL ROOFS

J-1 A more comprehensive formula for the shape coefficient for multilevel roofs than that given in **7.4.2** is as follows:

$$\mu_w = 1 + \frac{1}{h}(m_1 l_1 + m_2 l_2)(l_2 - 2h)$$

$$\mu_1 = 0.8$$

$$l_3 = 2h$$

(h and l in m)

Restriction:

$$\mu_w \leq \frac{kh}{S_o}$$

where

S_0 = kilopascals (kilonewtons per square metre)

k = newtons per cubic metre

l_3 = $\leq 15\ m$

Values of $m_1 (m_2)$ for the higher (lower) root depend on its profile and are taken as equal to:

0.5 for plane roofs with slopes $\beta \leq 20°$ and vaulted roofs with $\frac{f}{l} \leq \frac{1}{18}$

0.3 for plane roofs with slopes $\beta \leq 20°$ and vaulted roofs with $\frac{f}{l} \leq \frac{1}{18}$

The coefficient m_1 and m_2 may be adjusted to take into account conditions for transfer of snow on the roof surface (that is wind, temperature, etc).

NOTE — The other condition of loading shall also be tried.

ANNEX K

(*Clause* 8.7)

VIBRATIONS IN BUILDINGS

K-1 GENERAL

In order to design the buildings safe against vibrations, it is necessary to identify the source and nature of vibration. Vibrations may be included in the buildings due to various actions, such as:

a) human induced vibrations, for example, the walking or running or a single person or a number of persons or dancing or motions in stadia or concert halls;

b) machine induced vibrations;

c) wind induced vibrations;

d) blast induced vibrations;

e) traffic load, for example, due to rail, fork-lift, trucks, cars, or heavy vehicles;

f) airborne vibrations;

g) crane operations; and

h) other dynamic actions such as wave loads or earthquake actions.

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The dynamic response of buildings for the above mentioned causes of vibration of buildings may have to be evaluated by adopting standard mathematical models and procedures.

The severity or otherwise of these actions have to be assessed in terms of the limits set for dynamic response (frequencies and amplitude of motion) of the buildings related to (a) human comfort, (b) serviceability requirements such as deflections and drifts and separation distances to avoid damage due to pounding and (c) limits set on the frequencies and amplitude of motion for machines and other installations.

In order to verify that the set limits are not exceeded, the actions may be modelled in terms of force-time histories for which the structural responses may be determined as time histories of displacements or accelerations by using appropriate analytical/numerical methods.

K-2 SERVICEABILITY LIMIT STATE VERIFICATION OF STRUCTURE SUSCEPTIBLE TO VIBRATIONS

K-2.1 While giving guidance for serviceability limit state verification of structure susceptible to vibrations, here it is proposed to deal with the treatment of the action side, the determination of the structural response and the limits to be considered for the structural response to ensure that vibrations are not harmful or do not lead to discomfort.

K-2.2 Source of Vibrations

Vibrations may be included by the following sources:

a) by the movement of persons as in pedestrian bridges, floors where people walk, floors meant for sport or dancing activities and floors with fixed seating and spectator galleries;

b) by working of machines as in machine foundations and supports, vibrations transmitted through the ground and pile driving operations;

c) by wind blowing on buildings, towers, chimneys and masts, guyed masts, pylons, bridges, cantilevered roofs, airborne vibrations;

d) induced by traffic on rail or road bridges and car park structures and exhibition halls; and

e) by earthquakes.

K-2.3 Modelling of Actions and Structures

For serviceability limit states, the modelling of these actions and of the structure depends on how the serviceability limits are formulated. The serviceability limit states may refer to:

a) human comfort;

b) limits for the proper functioning of machines and other installations; and

c) maximum deformation limits to avoid damage or pounding.

In order to verify that these limits are not exceeded, the actions may be modeled in terms of force-time histories, for which the structural responses may be determined as time histories of displacements or accelerations by using appropriate analytical/numerical methods. Where the structural response may significantly influence the force-time histories to be applied, these interactions have to be considered either in modeling a combined load-structure vibration system or by appropriate modifications of the force time histories. In addition to the levels of vibration for which presently limits have been specified, the possible deformations of structural members and systems using different clauses in the relevant codes have to be evaluated by adopting standard mathematical models and procedures.

K-2.4 Force-Time Histories

The force-time histories used in the dynamic analysis should adequately represent the relevant loading situations for which the serviceability limits are to be verified. The force-time histories may model:

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a) human induced vibrations, for example the walking or running of a single person or a number of persons or dancing or motions in stadia or concert halls;

b) machine induced vibrations, for example by force vectors due to mass eccentricities and frequencies, that may be variable with time;

c) wind induced vibrations;

d) blast induced vibrations;

e) traffic load, for example rail, fork-lift, trucks, cars, or heavy vehicles;

f) airborne vibrations;

g) crane operations; and

h) other dynamic actions such as wave loads or earthquake actions.

ANNEX L

(*Foreword* and clause 8.8)

BLAST LOAD

L-1 BLAST LOAD

L-1.1 This annex prescribes the minimum criteria for analysis and design of new structures for blast effects of above-ground explosions of high-explosives under three different detonation scenarios: surface burst, free-air burst and air burst. The primary objective of this annex is to minimize human casualties and safeguard assets due to above ground explosions. The performance expectation of a structure designed as per the provisions of this annex would depend on the type of protection category selected for that structure and the blast loading determined through threat assessment.

L-1.2 This annex shall be used in conjunction with other applicable standards for analysis and design of structures. This annex does not supersede or replace other design standards. This annex shall be applicable if design against malevolent or accidental blast loading is determined to be a credible threat to a structure.

L-1.3 The owner of a structure is the competent authority who can determine the necessity of blast-resistant design. In the scenario of accidental and malevolent blast event which may pose public health and safety issue, competent authorities other than the owner of building that govern or regulate the construction, operation and maintenance of the structural facility, can specify the requirement of blast-resistant design.

L-1.4 The provisions of this annex do not address the following:

a) Estimation of structural loads due to nuclear detonations, pressure vessel explosions, dust and vapour cloud explosions;

b) Effects of cased explosives and shaped charges;

c) Threats emanating from fragmentation, induced fire or thermal effects, biological or chemical radiations, electromagnetic pulse (EMP) and deflagration events; and

d) Detonations in contact with structural elements.

L-2 TERMINOLOGY

For the purpose of this annex, the following definitions shall apply.

L-2.1 Arrival Time — Time taken by the shock after the explosion to reach the location of interest.

L-2.2 Charge Mass — Mass of the explosive material responsible for generating blast effects.

L-2.3 Clearance Time — Time in which the reflected pressure decays down to the sum of the side on overpressure and the dynamic pressure.

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L-2.4 Close-in Detonation — Detonation at a scaled distance (Z) below which the resulting blast pressure distribution is non-uniform over the reflecting surface of the element being considered. It corresponds to a value of Z less than 1.2 m/kg1/3.

L-2.5 Decay Parameter — The coefficient of the negative power of exponent e governing the fall of pressure with time in the pressure-time curves.

L-2.6 Detonation — Fast and sudden release of huge amount of energy through a stable chemical reaction in which reaction front propagates into unreacted substance at supersonic speed resulting in formation of shock waves.

L-2.7 Drag Force — Force on a structure or structural element due to the dynamic pressure of the blast wave. On any structural element, the drag force equals dynamic pressure multiplied by the drag coefficient and the area of the element.

L-2.8 Ductility Ratio — Ratio of the maximum deflection to the deflection corresponding to the elastic limit.

L-2.9 Dynamic Increase Factor (DIF) — A factor applied to material strength to account for high strain rate loading effects.

L-2.10 Dynamic Pressure — The pressure due to air mass movement behind the shock front.

L-2.11 Exponential Blast Pressure History — The variation of blast pressure with time at a point represented through an exponential decaying function, which considers positive and negative phase of blast overpressures.

L-2.12 Far-Field Detonation — Detonation at a scaled distance (*Z*) above which the resulting blast pressure distribution is uniform over the reflecting surface of element being considered. It corresponds to a value of *Z* greater than 1.2 m/kg$^{1/3}$.

L-2.13 High-Explosives — Chemical explosive that undergoes detonation to produce shock waves with high pressure and short duration.

L-2.14 Impulse — The integral of the pressure-time curve per unit projected area due to explosion (*see* specific impulse).

L-2.15 Mach Number — The ratio of the speed of the shock front propagation to the speed of sound in standard atmosphere at sea level.

L-2.16 Negative Phase — The duration of reflected overpressure when it is below the ambient atmospheric pressure.

L-2.17 Overpressure — The rise in pressure above atmospheric pressure due to the shock wave from an air blast.

L-2.18 Positive Phase —The duration of overpressure when it is above the ambient atmospheric pressure.

L-2.19 Protection Category — Category assigned to different structures that correlates its importance to the performance goals through qualitative estimate of damage.

L-2.20 Reflected Overpressure —The overpressure resulting due to reflection of a shock wave front striking any surface. If the shock front is parallel to the surface, the reflection is normal.

L-2.21 Regular Shaped Structure — A structure that does not have any spatial geometric irregularity.

L-2.22 Scaled Distance — The distance between the centre of explosion to the point of interest divided by the cube root of the charge mass expressed in equivalent TNT. The unit for scaled distance is m/kg$^{1/3}$.

L-2.23 Shape Function — The displaced shape of the component as a function of its length normalized to the maximum deflection.

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L-2.24 Shock Wave Front — The discontinuity between the blast wave and the surrounding atmosphere. It propagates away from the point of explosion in all directions at a speed greater than the speed of sound in the undisturbed atmosphere.

L-2.25 Side-on (Incident) Overpressure — Overpressure if it is not reflected by any surface.

L-2.26 Simplified Overpressure History — A triangular representation of the overpressure history obtained by equating the impulse and the peak overpressure of the exponential representation of the overpressure history.

L-2.27 Specific Impulse — The area under the pressure-time curve due to explosion. This is often referred to as impulse (*see* **L-2.13**).

L-2.28 Standoff Distance — The shortest distance between the centre of detonation and the point of interest at which the effects of explosion need to be considered.

L-2.29 Support Rotation — The angle formed by a flexural member in its deflected shape with respect to the chord adjoining the two support ends (*see* Fig. 45).

L-2.30 Transit Time — Time required for the shock front to travel across the structure or its element under consideration.

L-3 SYMBOLS

For the purpose of this annex, the following notations shall apply.

b	Decay parameter/coefficient
C_d	Drag coefficient
C_r	Velocity of sound in air
$C_{r\alpha}$	Reflection coefficient
E	Modulus of elasticity of material
H	Height of the structure
I	Moment of inertia of member
i_e	Equivalent specific impulse of a uniformly distributed load
i_{ro}	Peak reflected specific impulse
j	Number of concentrated load points
K_L	Load factor
K_{LM}	Load mass factor
K_M	Mass factor
k_E	Effective stiffness of equivalent single spring-mass system
L	Length of structure in the direction of motion of blast wave (*see* Fig. 39)
M	Charge mass/weight
M_p	Plastic moment of section
M_{pm}	M_p at mid-span
M_{ps}	M_p at support
M_{pfa}	Total positive ultimate moment capacity along mid-span section parallel to short edge
M_{pfb}	Total positive ultimate moment capacity along mid-span section parallel to long edge

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m	Distributed mass intensity per unit length
n	Number of concentrated masses
p_a	Ambient atmospheric pressure
P_r	Concentrated dynamic force at point *r*
p_{ro}	Peak reflected overpressure
p_s	Side-on overpressure
p_{so}	Peak side-on overpressure
P_t	Total dynamic load (at any instant of time)
$p(x)$	Intensity of distributed dynamic load per unit length
p_{o1}	Peak reflected overpressure on a surface including the effects of clearing
p_{o2}	Intercept of the reflected overpressure history on the y-axis corresponding to the clearing phase
q	Dynamic pressure
q_o	Peak dynamic pressure
R	Standoff distance
R_m	Peak resistance of a structural member against a given load distribution
S	W or *H* whichever is less
T	Elastic time-period of structural member
t_c	Clearance time
t_d	Duration of the equivalent triangular pulse
t_m	Time to the peak displacement of an SDOF system
t_o	Time for positive phase of side-on overpressure
t_t	Transit time
t_{d1}	Time at which clearing starts on a finite reflecting surface (*see* Fig. 47B)
t_{d2}	Total time duration of blast loading on a finite reflecting surface (*see* Fig. 47B)
U	Shock front velocity
u	Peak particle velocity
V_A	Total dynamic reaction along a short edge
V_B	Total dynamic reaction along a long edge
y_E	Displacement corresponding to the elastic limit of an equivalent bilinear representation of idealised resistance-deflection curve
y_{el}	Displacement corresponding to the first onset of yielding (elastic limit) of the idealized resistance-deflection curve of a structural member
y_m	Maximum deflection/deformation permitted in the design of the structure
y_o	Peak static displacement of an SDOF system without inertial effects
W	Span or width of structure across the direction of shock wave propagation
Z	Scaled distance

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μ Ductility ratio $= \frac{y_m}{y_E}$

φ_r Deflection at point *r* of an assumed deflected shape for concentrated loads

$\varphi_{(x)}$ Deflection at point *x* of an assumed deflected shape for distributed loads

$\varphi_{(x,y)}$ Deflected shape function of blast-loaded component

L-4 GENERAL PRINCIPLES

L-4.1 Blast Resistant Design Attributes

The design process of a building against blast should start with planning of building layout and arrangements. The distance between the explosive and the building shall be maximized through a combination of site-access controls, landforms, anti-ram structures, street and site landscaping and perimeter walls. The building should preferably have short-face oriented towards blast source. Also, it shall be located uphill and placed upstream of wind direction and away from congestion. The structures and their contents that need protection against an explosion event should be decided and all possible scenarios need to be considered which can cause damage or occupant injuries. The overall design of a structure shall aim to incorporate following attributes in the overall planning and design of a facility against explosion threats:

a) *Security* — Discourage potential terrorist attacks on a structure through visible security personnel deployment and defences to reduce chance of such attacks;

b) *Deception* — Disguise the critical area or content of structure so that the focus of the attack is misdirected. Such attacks, even if successfully completed, may not deliver intended impact and realize its goals;

c) *Shielding* — Shield the target structure through physical barriers, including but not limited to blast walls, bollards, anti-ram elements, vehicle and pedestrian entry controls and landforms. The objective of shielding is to increase the standoff distance between the charge and the target structure and components; and

d) *Design* — The attack must be blunted through appropriate design and detailing of the structure to absorb the energy of the attack and safeguard valuable assets.

L-4.2 Blast Source

L-4.2.1 *Charge Shape*

The provisions of this annex shall be applicable for:

a) explosives of spherical shape; and

b) non-spherical charges if the detonation effects are considered at scaled distances greater than 3 m/kg$^{1/3}$.

In case of non-spherical charges at scaled distances smaller than 3 m/kg$^{1/3}$, the provisions of **L-8.3.3** shall be applicable with explicit modelling of the detonation process.

L-4.2.2 *Detonation Scenarios*

The standard addresses three scenarios of unconfined explosions based on the location of the detonation with respect to the point of interest:

a) Spherical (free-air) burst (*see* Fig. 24);

b) Hemispherical (surface) burst (*see* Fig. 25); and

c) Air-burst (*see* Fig. 26).

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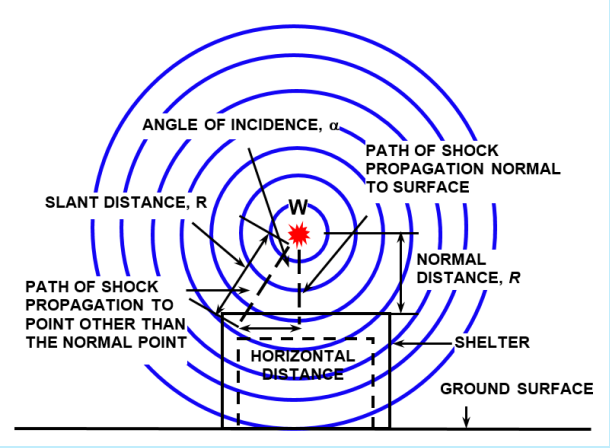

FIG. 24 SPHERICAL (FREE-AIR) BURST

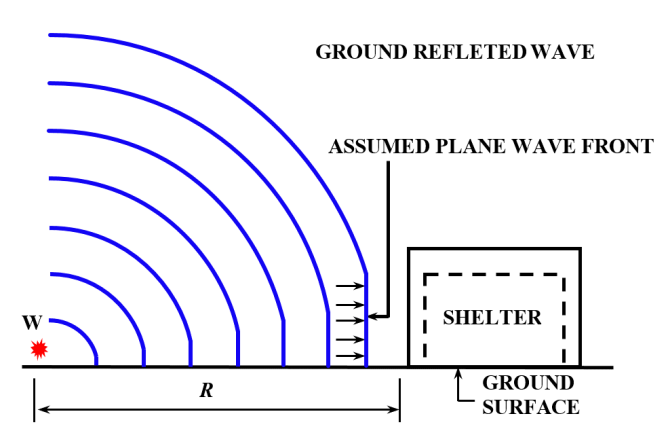

FIG. 25 HEMISPHERICAL (SURFACE) BURST

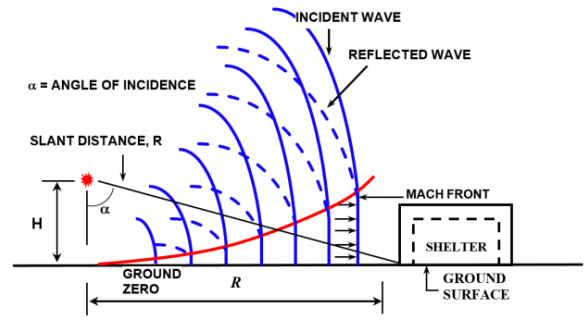

FIG. 26 AIR-BURST

L-4.2.3 *Standoff Distance*

The standoff distance (R) shall be estimated as the distance between the centre of detonation and the point of interest. If the location of interest is not directly in the line of the sight from the centre of detonation, then Fig. 27 shall be referred to estimate the effective standoff distance.

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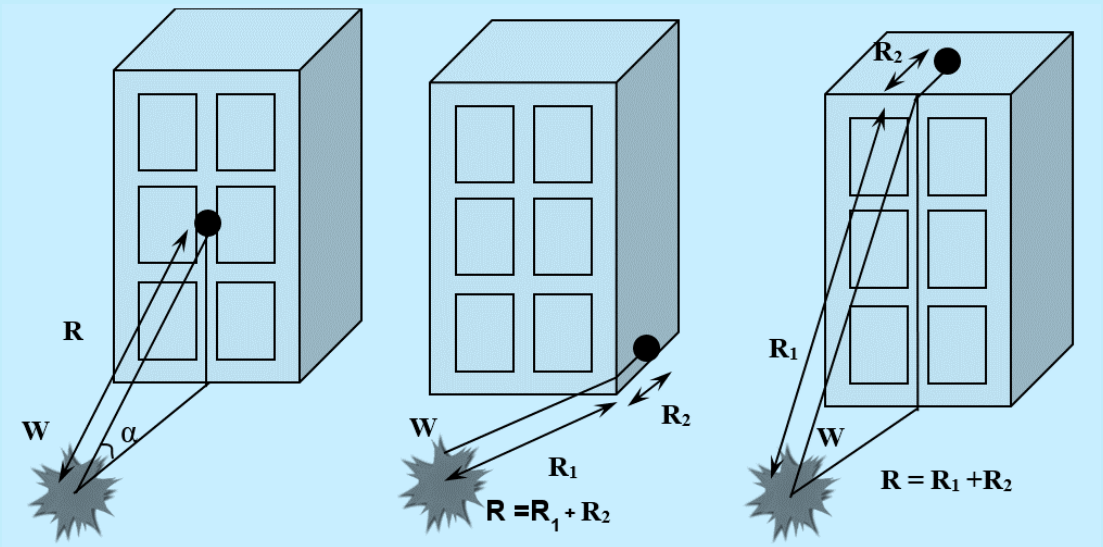

FIG. 27(A) FRONT FACE FIG. 27(B) SIDE FACE FIG. 27(C) TOP FACE

FIG. 27 CALCULATION OF STANDOFF DISTANCES AT DIFFERENCE FACES OF A BUILDING STRUCTURE

L-4.2.4 *Equivalent TNT Charge*

Trinitrotoluene (TNT) shall be used to benchmark all types of explosives. To quantify blast wave parameters (pressure, impulse) from explosives other than TNT, the actual mass of the explosive (M_{EXP}) shall be converted into a TNT-equivalent mass. The TNT equivalent weight (M_{TNT}) of any explosive shall be estimated as:

$$M_{TNT} = \left(\frac{\Delta E_{EXP}}{\Delta E_{TNT}}\right) M_{EXP}$$

where

ΔE_{EXP} and ΔE_{TNT} are the specific energies of the explosive and the TNT, respectively.

The factors presented in Table 42 shall be utilized to obtain equivalent mass of TNT corresponding to the listed explosives.

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Table 42 Equivalent TNT Mass of Different Explosives

(*Clause* L-4.2.4)

Sl No.	Explosive	Density Mg/m^3	Equivalent Mass for Pressure	Equivalent Mass for Impulse
(1)	(2)	(3)	(4)	(5)
i)	Ammonia dynamite (50 percent strength)	NA*	0.90	0.90 I
ii)	Ammonia dynamite (20 percent strength)	NA*	0.70	0.70 I
iii)	ANFO (94/6 ammonium nitrate/fuel oil)	NA*	0.87	0.87 I
iv)	Composition C-4	1.59	1.20	1.19
		1.59	1.37	1.19
v)	Cycloid (75/25 RDX/TNT) (70/30) (60/40)	1.71	1.11	1.26
		1.73	1.14	1.09
		1.74	1.04	1.16
vi)	Gelatine dynamite (50 percent strength)	NA*	0.80	0.80 I
vii)	Gelatine dynamite (20 percent strength)	NA*	0.70	0.70 I
viii)	HMX	NA*	1.25	1.25 I
ix)	Nitrocellulose	1.65 to 1.70	0.50	0.50 I
x)	Nitroglycerine dynamic (50 percent strength)	NA*	0.90	0.90 I
xi)	Nitroglycerine (NQ)	1.72	1.00	1.00 I
xii)	PETN	1.77	1.27	1.27 I
xiii)	Picrotol (52/48 Ex D/TNT)	1.63	0.90	0.93
xiv)	RDX	NA*	1.10	1.10 I
xv)	RDX/wax (98/2)	1.92	1.16	1.16 I
xvi)	RDX/AL/wax (74/21/5)	NA*	1.30	1.30 I
xvii)	Tetryl	1.73	1.07	1.07 I
xviii)	Tetrytol (75/25 tetryl/TNT)	1.59	1.06	1.06 I
xix)	TNT	1.63	1.00	1.00

NOTE
* NA — Data not available.
I Value is estimated.

L-4.2.5 *Cube Root Scaling*

Cube root scaling shall be used to relate blast parameters from different sources and standoff distances. The cube-root scaling shall be valid if the conditions specified in L-4.2.1 are satisfied.

The blast load parameters can be obtained in scaled form as function of scaled distance, Z, as:

$$Z = \frac{R}{M^{1/3}}$$

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where

R = standoff distance of the point of interest from the centre of charge mass; and

M = Mass in equivalent TNT.

The unit for scaled distance shall be $m/kg^{1/3}$, for standoff distance in m and charge mass in kg.

L-4.3 Performance Expectations

L-4.3.1 *Categorization of Structures*

Structures shall be categorized based on their importance as per the specification of Table 43.

The categorisation of structures as per Table 43, does not automatically require them to be designed for explosion threats. Only, if the threat assessment carried as per the provisions of **L-5.1** identifies blast loading due to explosion as a potential threat, the categorisation in Table 43 shall be used to define the importance of a structure.

Table 43 Categories of Structures

(*Clause* L-4.3.1)

Sl No.	Importance	Structure Type
(1)	(2)	(3)
i)	Low	Residential buildings with 50 or less occupants
ii)	Medium	a) Residential buildings with more than 50 occupants b) Buildings with large congregation, such as schools, public offices and cinemas c) Industrial buildings with continuous human occupancy
iii)	High	a) Lifeline structures (water, power, communication, transportation, hospital) b) Structures housing essential services that are required for disaster management (for example, emergency relief stores, data centres, governance continuity buildings) c) Financial and commercial establishments of strategic importance

L-4.3.2 *Structural Performance*

A protection category shall be decided by the building owner based on the importance of the building and required performance. The performance goals for each protection category are listed in Table 44.

Table 44 Protection Category and Associated Performance Goals

(*Clause* L-4.3.2)

Sl No.	Protection Category	Performance Goal
(1)	(2)	(3)
i)	1	Minimal damage, resumption after brief halt of operations
ii)	2	Limited damage, operational after repairs and retrofit
iii)	3	Extensive damage, safe replacement, or demolition

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Structures of high, medium and low importance shall be provided with protection categories 1, 2 and 3 respectively, as at least a minimum as specified in Table 45.

The facility owner may specify a protection category higher than the minimum protection category specified in Table 45 based on the realistic assessment of threat.

Table 45 Protection Category and Associated Category of Structures

(*Clause* L-4.3.2)

Sl No.	Building Category	Minimum Protection Category
(1)	(2)	(3)
i)	Low	3
ii)	Medium	2
iii)	High	1

The protection categories specified in shall comply with the deformation limits of structural elements provided in **L-8.1.4** when using simplified single degree of freedom (SDOF) and multi degree of freedom (MDOF) methods of analysis as per requirements of **L-8.3.1** and **L-8.3.2** respectively. Accordingly, all elements shall be designed and detailed to provide deformation limits specified for different protection categories.

If advanced methods of analysis are used as per **L-8.3.3**, the response and damage in structure and elements for the performance goals of each protection category mentioned in can be independently established. In such case, deformation limits specified in **L-8.3.1** and **L-8.3.2** need not to be used.

L-5 THREAT ASSESSMENT

L-5.1 Threat Assessment Parameters

A threat assessment shall be carried out to determine all potential sources of detonation.

The threat assessment shall identify two parameters for the purpose of calculation of blast load parameters, namely:

a) charge mass; and

b) standoff distance.

If threat assessment identifies multiple sources of detonation, the effect of blast on the structure shall be assessed considering individual, as well as combinations of all identified detonation threats.

L-5.2 Design Blast (Reference Explosion)

L-5.2.1 *Design TNT Equivalence*

If the threat scenario results in identification of one of the detonation sources listed in Table 46, then the associated charge mass may be used for determination of equivalent TNT capacity.

L-5.2.2 *Standoff Distance*

The standoff distance (R) is site specific and shall be estimated as per the actual geometry and access control of the target structure.

L-6 BLAST LOAD

L-6.1 Blast Wave Propagation

The blast loads for design of structures and components shall be determined from blast pressure history.

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Table 46 Detonation Source and the Associated Charge Mass
(*Clause* L-5.2.1 *and* Table 60)

Sl No.	Threat Description		Explosive Capacity (TNT Equivalent)
(1)	(2)	(3)	(4)
i)		Pipe bomb	2.5 kg
ii)		Suitcase	25 kg
iii)		Hatchback	250 kg
iv)		Sedans/SUV	500 kg
v)		Passenger van/small cargo truck	2 000 kg
vi)		Delivery truck	5 000 kg
vii)		Water tanker/diesel truck/gas truck	15 000 kg

The variation of pressure at a fixed location away from the centre of detonation shall be obtained using following assumptions:

a) the shock wave due to explosion moves away from the centre of detonation at a speed, U;

b) when the shock wave reaches a point at the arrival time t_a, the pressure rises instantaneously from the ambient pressure (p_a) to a peak pressure. This peak pressure, p_{so}, is known as side-on overpressure or peak incident overpressure;

c) the time needed for the pressure to reach its peak value is very small and for design purposes it is assumed to be zero;

d) the overpressure decreases exponentially from its peak value to the ambient pressure over the positive phase duration (t_o). After the positive phase of the pressure-time profile, the pressure becomes smaller than the ambient value (referred to as negative pressure) and finally returns to the ambient pressure, p_a;

e) the movement of air caused by the shock wave induces dynamic pressure, in addition to blast pressure. Air dynamic pressure q_o and peak particle velocity u shall be obtained using peak incident pressure, p_{so} (*see* Fig. 32 and Fig. 33); and

f) shock wave upon reflection from a rigid surface would apply reflected overpressure, p_r, to the reflecting surface. The variation of reflected overpressure with time for normal reflection follows the same trend as the incident overpressure but characterized by significantly higher peak overpressure.

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L-6.2 Decay of Blast Overpressure with Time

The blast overpressure is assumed to vary with time, t according to the following exponential relationship (*see* Fig. 28):

$$p_s(t) = p_o\left(1-\frac{t}{t_o}\right)e^{-b(t/t_o)}$$

where

p_o = peak blast overpressure, which is interpreted as the peak incident and reflected overpressures for the incident and the reflected blast wave, respectively; and

b = decay coefficient; and t_o is the positive phase duration. Special literature is to be referred for the calculation of the blast wave decay parameter, b.

L-6.3 Blast Overpressure History Parameters

The maximum values of the positive side-on overpressure p_{so}, incident impulse i_s, reflected over pressure p_r, reflected impulse i_r, positive phase duration t_o, shock wave front velocity U and wave length L_w, for varying scaled distances, are to be obtained for a spherical TNT explosion in free air and a hemispherical TNT explosion on the surface using Fig. 30 and Fig. 31, respectively. Further, air dynamic pressure q_o and peak particle velocity u, are to be obtained using Fig. 32 and Fig. 33, respectively.

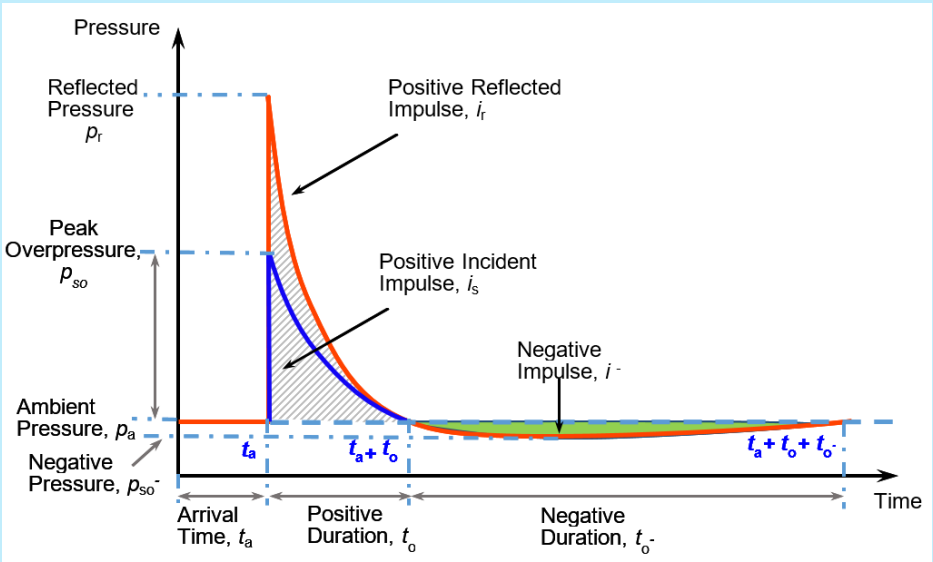

FIG. 28 INCIDENT AND REFLECTED BLAST PRESSURE HISTORY

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L-6.4 Oblique Reflection of Blast Wave

The clauses of this section are applicable to determine the peak reflected overpressure for scenarios where the angle between the wave front and the interacting surface is α (≤ 90°) as shown in Fig. 29. In such case, the peak reflected overpressure (p_{ro}) is to be determined as multiplication of peak incident overpressure (p_{so}) with a reflection coefficient ($C_{r\alpha}$). The reflection coefficient at any angle is to be determined from Fig. 34 for given incident overpressure.

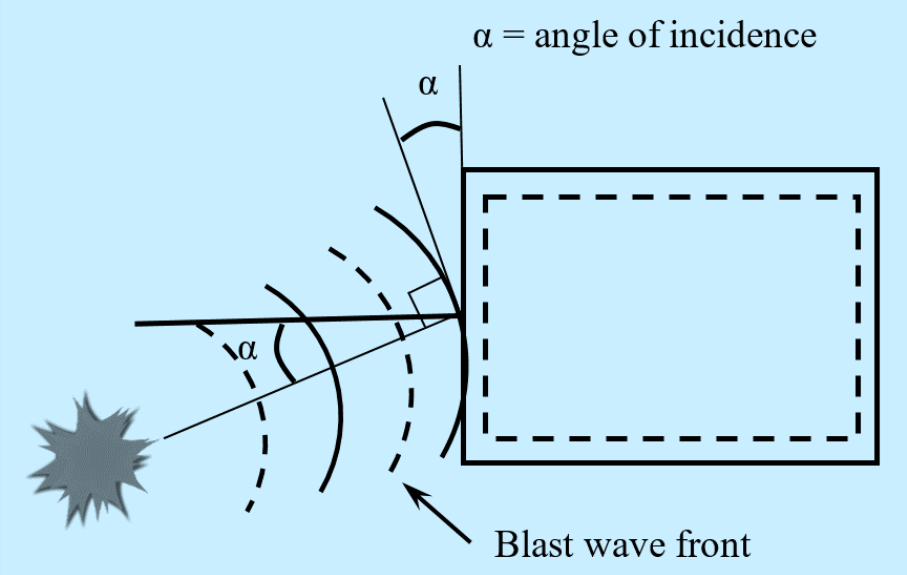

FIG. 29 OBLIQUE REFLECTION OF A BLAST WAVE

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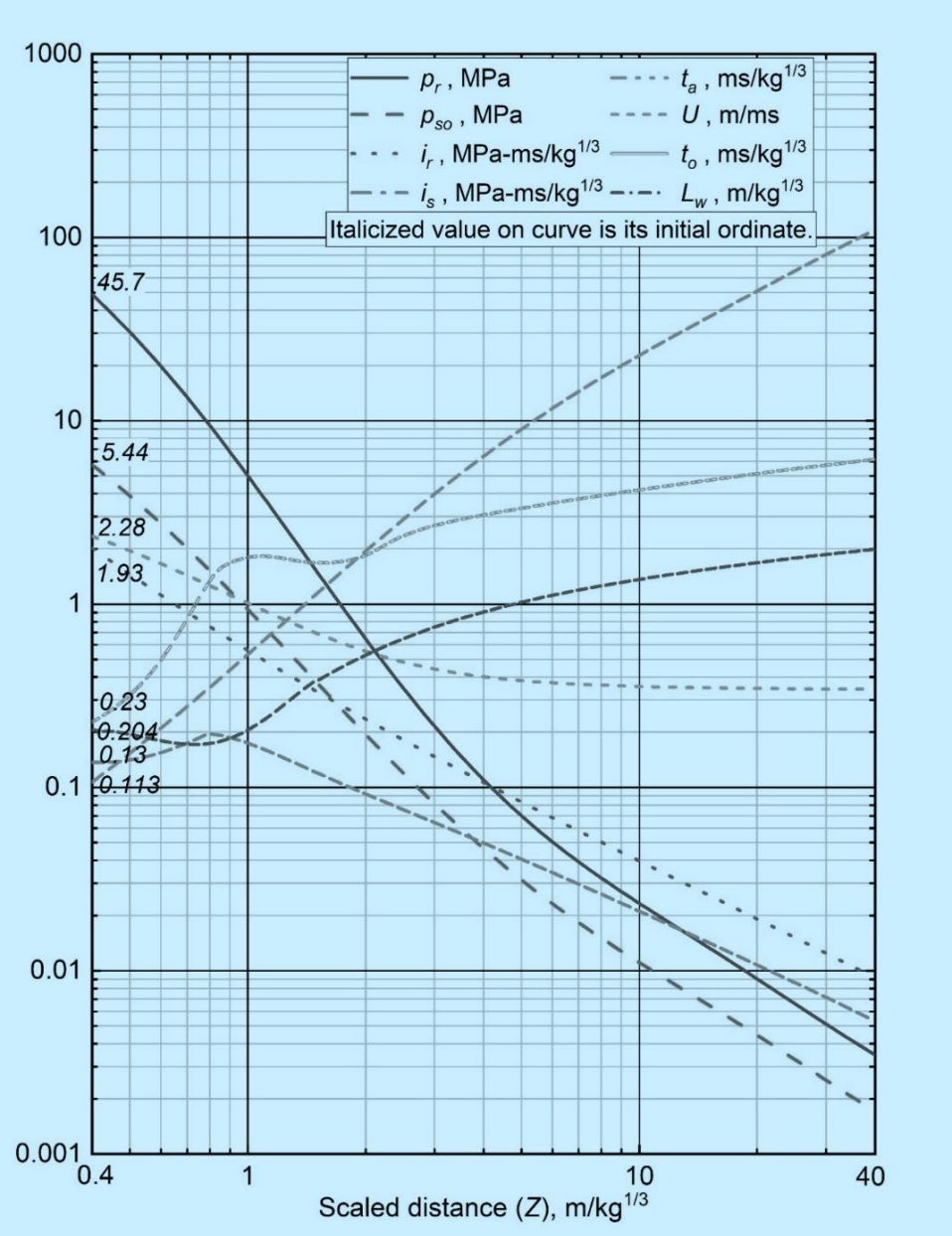

FIG. 30 POSITIVE PHASE SHOCK WAVE PARAMETERS FOR A SPHERICAL TNT EXPLOSION IN FREE AIR

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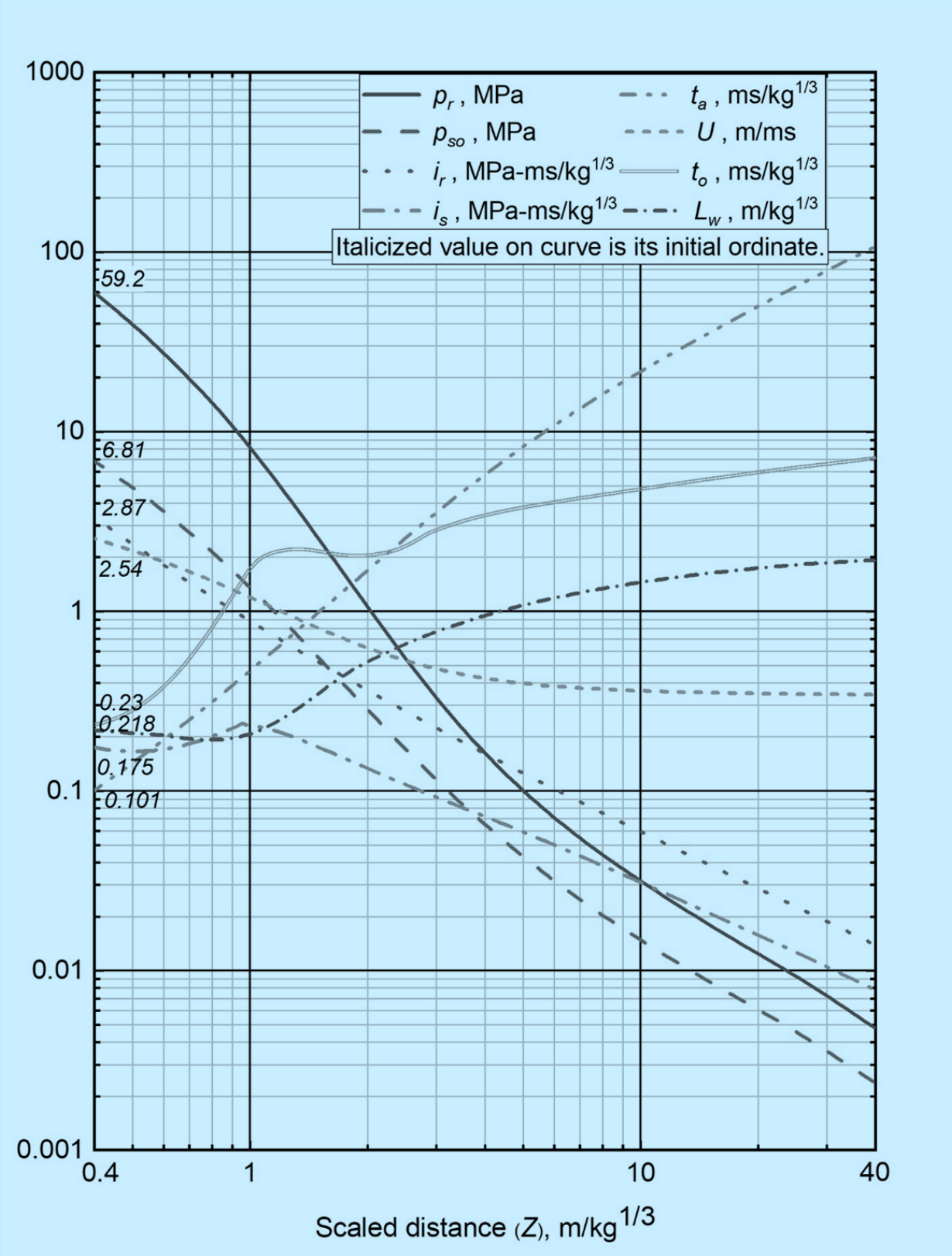

FIG. 31 POSITIVE PHASE SHOCK WAVE PARAMETERS FOR A HEMISPHERICAL TNT EXPLOSION ON SURFACE

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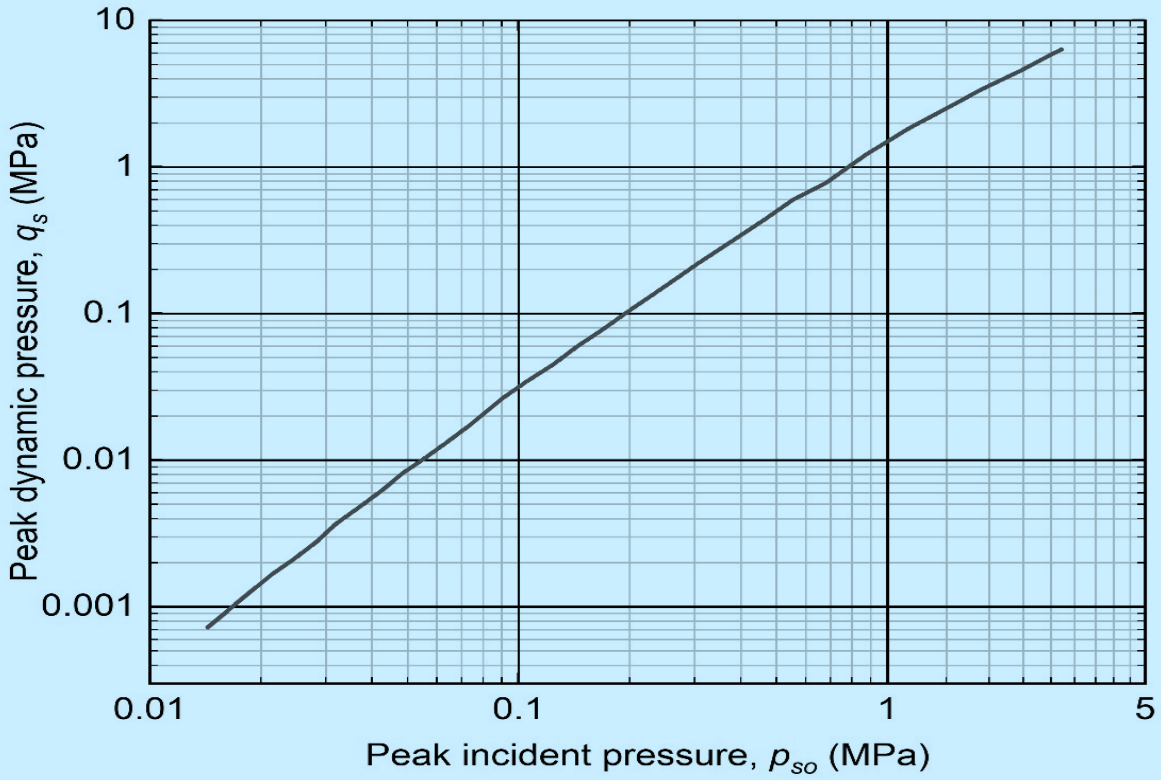

FIG. 32 PEAK DYNAMIC PRESSURE AND PEAK PARTICLE VELOCITY BEHIND A SHOCK FRONT

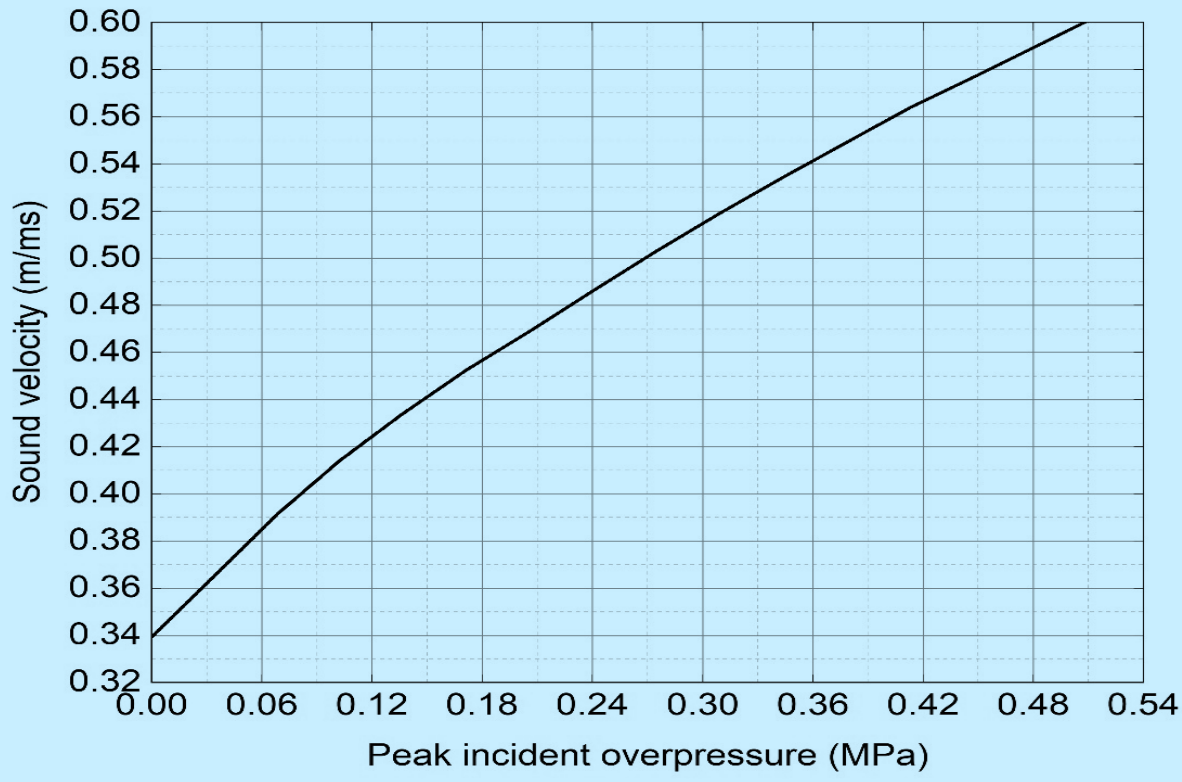

FIG. 33 SOUND VELOCITY IN THE REFLECTED OVERPRESSURE REGION

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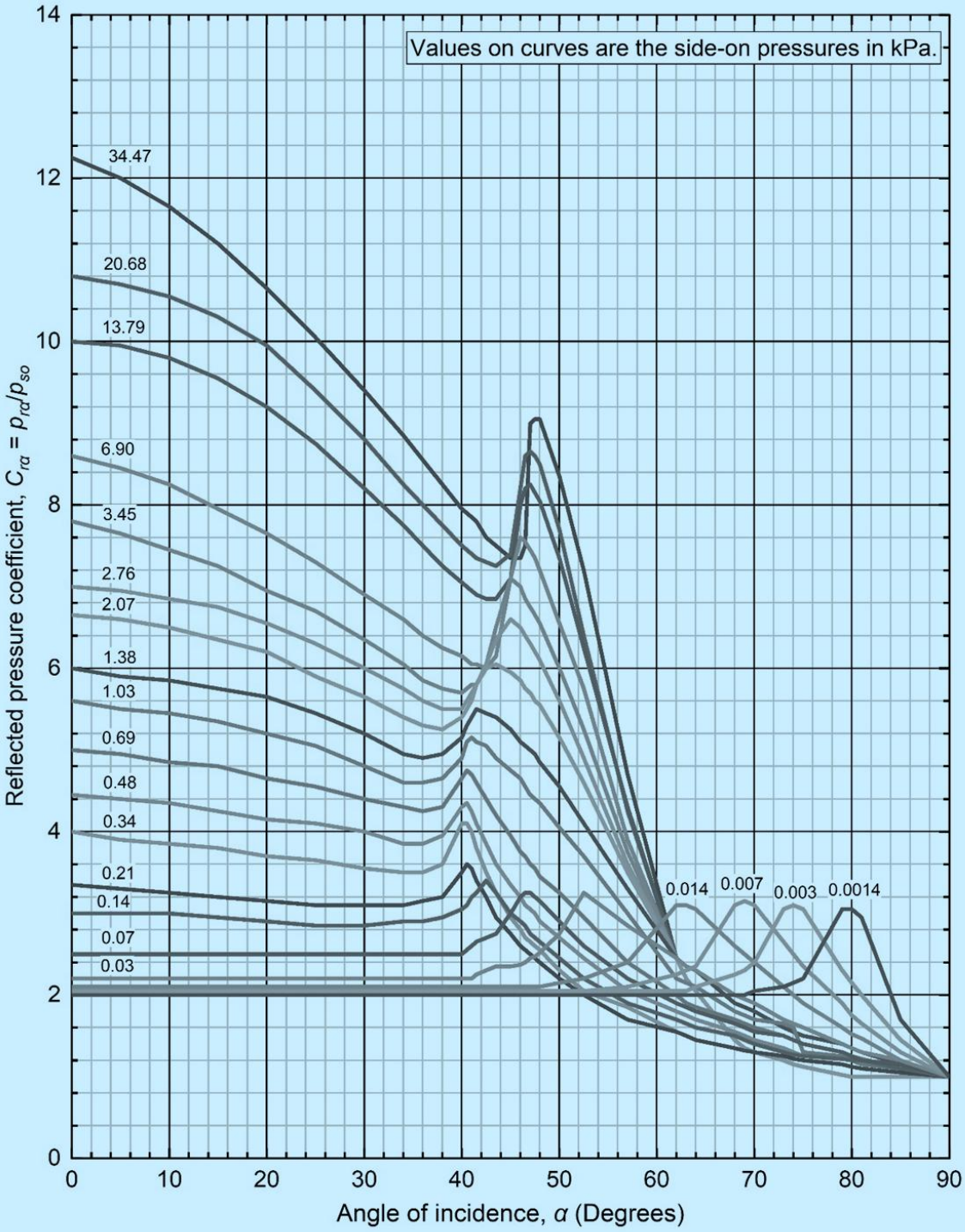

FIG. 34 INFLUENCE OF ANGLE OF INCIDENCE ON THE REFLECTED PRESSURE COEFFICIENT

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L-6.5 Simplified Blast Pressure History

For the purpose of simplified analysis and design, the exponential pressure time relation in the positive phase may be idealised to a triangular pulse e as shown in Fig. 35.

The triangular pulse shall be constructed to have the same peak pressure as in the original exponential blast pressure curve and the positive phase duration (t_d) so adjusted that the area under the curve remains unchanged.

The usage of triangular representation of pressure history shall be governed by relevant provisions in this annex.

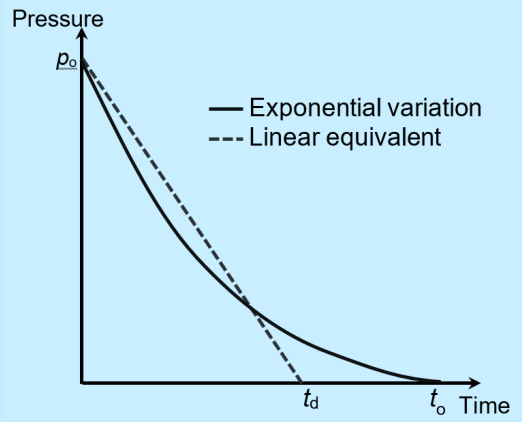

FIG. 35 SIMPLIFIED TRIANGULAR REPRESENTATION OF THE EXPONENTIAL OVERPRESSURE HISTORY

L-6.6 Ground Shocks

L-6.6.1 *Ground Shaking Intensity*

The scaled peak particle displacement, peak particle velocity and soil pressure generated by surface explosion shall be estimated using following equations:

Scaled peak particle displacement:

$$\frac{x}{M^{1/3}} = 60\frac{f}{c}\left(\frac{2.5R}{M^{1/3}}\right)^{1\text{-}n}$$

$$\text{Soil pressure, } p = 160 f \rho c \left(\frac{R}{M^{1/3}}\right)^{-n}$$

where

f = coupling factor;

n = attenuation coefficient;

M = explosive weight;

ρ = soil density;

c = soil seismic velocity; and

R = standoff distance.

Table 47 shall be used for taking the values of c, ρc and n.

The coupling factor f shall be taken as 0.14 for explosions above ground.

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Table 47 Soil Properties to Calculate Ground Shock Parameters

(*Clause* L-6.6.1)

Sl No.	Material Description	Seismic Velocity, c (m/s)	Acoustic Impedance, ρc (MPa/m/s)	Attenuation Coefficient, n
(1)	(2)	(3)	(4)	(5)
i)	Loose, dry sands and gravels with low relative density	182.9	0.27	3 to 3.25
ii)	Sandy loam, loess, dry sands and backfill	304.8	0.50	2.75
iii)	Dense sand with high relative density	487.7	1.00	2.5
iv)	Wet sandy clay with air voids (greater than 4 percent)	548.6	1.09	2.5
v)	Saturated sandy clays and sands with small amount of air voids (less than 1 percent)	1524	2.94	2.25 to 2.5
vi)	Heavy saturated clays and clay shales	<1524	3.40 to 4.07	1.5

L-6.6.2 *Safe Ground Particle Velocity*

For safety of buried and semi-buried structures, the peak particle velocity, u, as calculated using equation provided in **L-6.6.1** shall not exceed the values given in Table 48.

Table 48 Safe Ground Particle Velocity

(*Clause* L-6.6.2)

Sl No.	Type of Rock	Safe Ground Particle Velocity (mm/s)
(1)	(2)	(3)
i)	Soils, weathered or soft rock	50
ii)	Hard rock	70

For safety of unlined underground structures, the peak particle velocity, u, as calculated using equation provided in **L-6.6.1** shall not exceed the values listed in Table 49.

For safety of above ground structures, the peak particle velocity, u, measured at the foundation level, as calculated using equation provided in **L-6.6.1** shall not exceed the values listed in Table 50.

L-6.6.3 *Decay of Ground Shaking with Distance*

Decay of ground shaking with distance is to be estimated based on principle of mechanics and field measurement

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Table 49 PPV Damage Criterion for Unlined Underground Structures

(*Clause* L-6.6.2)

Sl No.	Rock Type	Unit Weight kg/m^3	Compressive Strength MPa	Tensile Strength MPa	Critical Peak Particle Velocity (m/s)			
					No Damage	*Slight Damage*	*Intermediate Damage*	*Serious Damage*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	Hard rock	2600 to 2700	75 to 110	2.1 to 3.4	0.27	0.54	0.82	1.53
		2700 to 2900	110 to 180	3.4 to 5.1	0.31	0.62	0.96	1.78
		2700 to 2900	180 to 200	5.1 to 5.7	0.36	0.72	1.11	2.09
ii)	Soft rock	2000 to 2500	40 to 100	1.1 to 3.1	0.29	0.58	0.90	1.67
		2000 to 2500	100 to 160	3.4 to 4.5	0.35	0.70	1.07	1.99

Table 50 Permissible Peak Particle Velocity at the Foundation Level of Structures, in mm/s

(*Clause* L-6.6.2)

Sl No.	Type of Structure	Dominant Excitation Frequency (Hz)		
		< 8 Hz	8 Hz to 25 Hz	> 25 Hz
(1)	(2)	(3)	(4)	(5)
i)	Buildings/structures not owned by owners:			
	a) Domestic houses/structure (*kuccha* brick and cement)	5	10	15
	b) Industrial buildings (RCC and framed structures)	10	20	25
	c) Objects of historical importance and sensitive structures	2	5	10
ii)	Buildings belonging to owner with limited span of life:			
	a) Domestic houses/structures (*kuccha* brick and cement)	10	15	25
	b) Industrial buildings (RCC and framed structures)	15	25	50

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L-6.7 Confined Explosions

The provisions given here are applicable for scenarios where detonation of explosives takes place inside closed spaces.

For the purpose of evaluating blast load effects, confined explosions are categorized into:

a) fully vented; and

b) partially vented.

The examples of fully and partially vented structures are shown in Fig. 36.

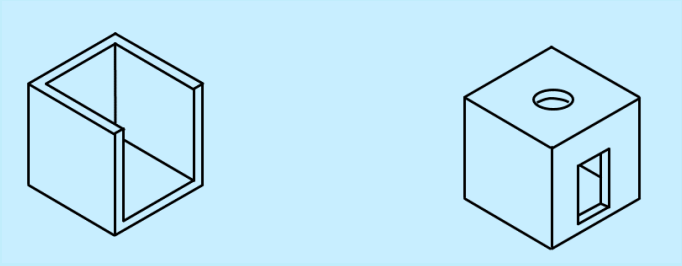

FIG. 36(A) FULLY VENTED FIG. 36(B) PARTIALLY VENTED

FIG. 36 CONFINED EXPLOSION SCENARIO

The variation of pressure at any location inside a closed space due to detonation inside the closed space is obtained using the following assumptions:

a) surfaces internal structural members are subjected to shock waves from multiple reflections characterized by multiple peaks; and

b) shock pressure is followed by development of gas pressure of small peak overpressures but significantly longer durations.

The provisions of **L-6.7** are only applicable when following conditions are satisfied:

a) structure is of regular geometric shape; and

b) the effect of confined detonation is being evaluated at scaled distances greater than 3 m/kg$^{1/3}$.

L-6.7.1 *Fully Vented Structures*

Fully vented structures are to be primarily subjected to shock pressures and the effect of gas pressures may be neglected. The reflected overpressure history at any point inside the structure is represented in Fig. 37. The peak pressure and impulse for the first pulse may be obtained using the procedure described in **L-6.3** as a case of direct reflection of the blast wave propagating from the centre of detonation to the point of interest. The second and the third triangular pulse shall be constructed from the characteristics of the first pulse.

Further simplification of pressure variation in Fig. 37 is allowed if the ratio of total load duration ($5t_a + t_d$) to the time-period of the structure is smaller than 0.1. In such cases, the three pulses of pressure history in Fig. 37 may be combined into a single pulse of peak pressure 1.75 p_{ro} and duration equal to t_d.

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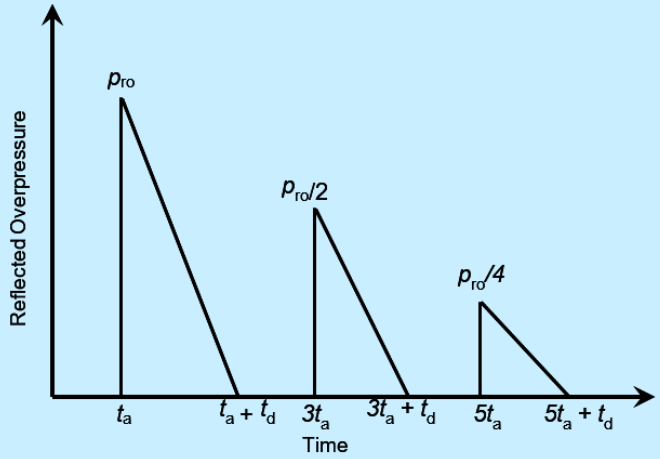

FIG. 37 REFLECTED OVERPRESSURE HISTORY FOR CONFINED DETONATION IN A FULL-VENTED SPACE

L-6.7.2 *Partially Vented Explosion*

Partially vented explosions are characterized by long duration gas pressures with minor effect of the shock pressures.

Wherever, required, the arrival time of the peak gas pressure is taken as the end time of the shock phase, that is, $5t_a + t_d$, as determined in **L-6.7.1**.

The blast load effects of confined explosion in partially vented structure is evaluated using advanced numerical technique as per the provisions of **L-8.3.3**.

The interior surface of a partially vented structure subject to confined explosion may conservatively be designed for the peak gas overpressure, p_g, which needs to be applied statically.

This peak gas overpressure is obtained from the chart in Fig. 38. The use of charts requires following condition to be satisfied:

$$0 \leq A / V_f^{2/3} \leq 0.022$$

where

A = structure's total vent area; and

V_f = free volume which shall be calculated as the total volume excluding the volume of all interior equipment and structural elements.

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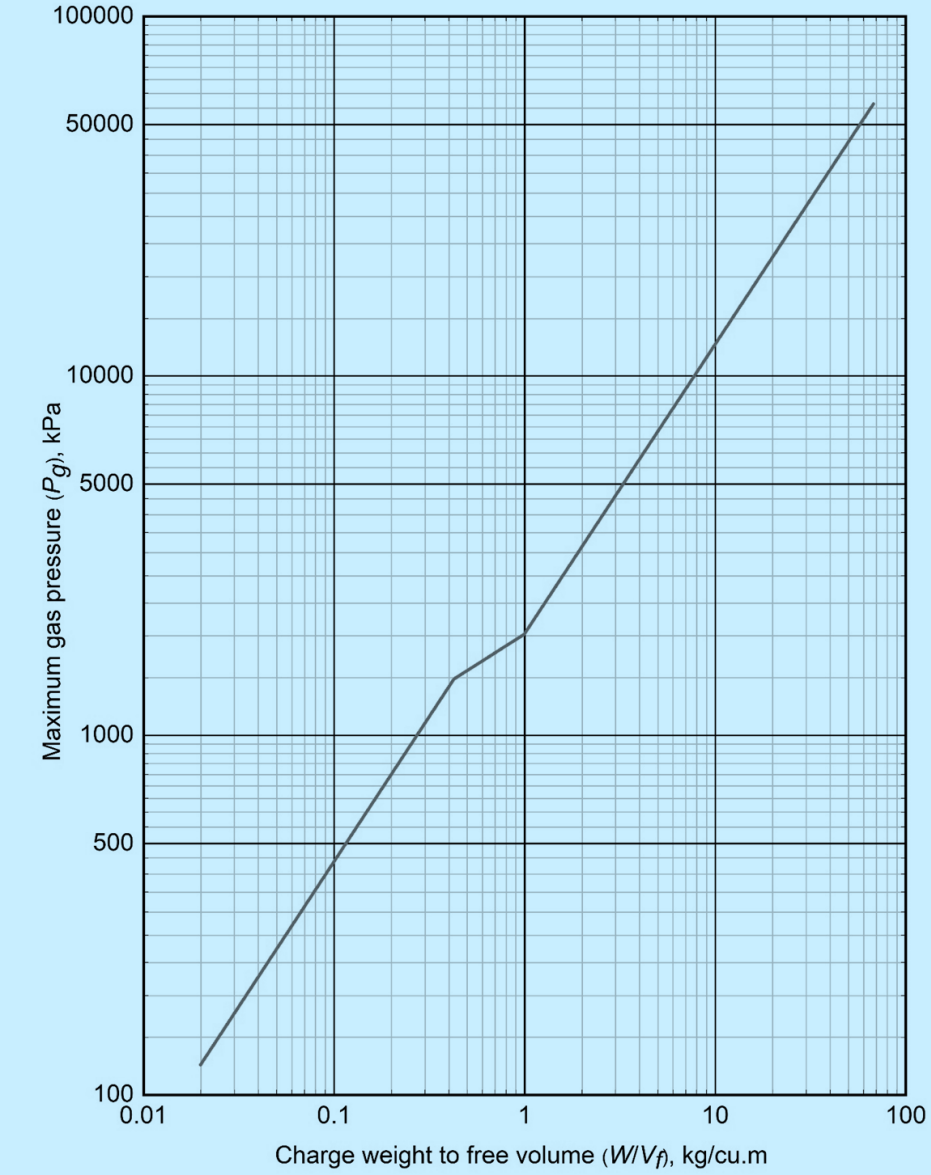

FIG. 38 PEAK GAS PRESSURE PRODUCED BY A TNT DETONATION IN A PARTIALLY VENTED STRUCTURE

L-7 EFFECTS OF BLAST ON STRUCTURES

L-7.1 Above Ground Structures

L-7.1.1 *Types of Structures*

Structures are categorized into following two types for the purpose of determining its interaction with the blast wave and calculating effective blast overpressure history:

a) *Diffraction type structures* — These are the closed structures without openings, with the total area opposing the blast. These are subjected to both the shock wave overpressure p_{so} and the dynamic pressures q caused by blast wind; and

b) *Drag type structures* — These are the open structures composed of elements like beams, columns, trusses, etc, which have small projected area opposing the shock wave. These are mainly subjected to dynamic pressures q.

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L-7.1.2 *Closed Rectangular Structures*

L-7.1.2.1 *Front face*

The peak reflected overpressure at any point on the front face of a closed rectangular structure shall be obtained using the following equation.

$$p_{r\alpha} = C_{r\alpha} p_{so}$$

where

p_{so} = peak side-on pressure that shall be obtained using Fig. 30 and Fig. 31 for spherical and hemispherical detonations, respectively, using the effective scaled distance; and

$C_{r\alpha}$ = reflection co-efficient, which shall be determined from Fig. 34 using the angle of reflection and the side-on pressure.

It is permitted to neglect spatial variation of pressure on the front face by assuming an equivalent uniform reflected overpressure which is equal to the peak reflected overpressure on the front face that is closest to the point of detonation. This, under most scenario, corresponds to the peak reflected overpressure, p_{ro}, at normal incidence ($\alpha = 0$ degrees).

The reflected overpressure, p_r, on the front face drops from the peak value p_{ro} to zero in time t_{rf}, whichever, is calculated as, $t_{rf} = 2i_{ro}/p_{ro}$, where p_{ro} and i_{ro} shall be obtained from using Fig. 30 and Fig. 31 for spherical and hemispherical detonations, respectively.

The stagnation overpressure, p_s, at the edges of the front face drops from the peak value, $p_{so} + C_d q_o$, to zero in time t_{of}, whichever, is calculated as, $t_{of} = 2i_{so}/p_{so}$, where p_{so} and i_{so} shall be obtained using Fig. 30 and Fig. 31 for spherical and hemispherical detonations, respectively. The drag coefficient, C_d, shall be taken as 1.0. The peak dynamic pressure, q_o, shall be determined from Fig. 32.

The reflected overpressure acting at a point on the front face drops from the peak value p_{ro} to overpressure ($p_{so} + C_d q$) in clearance time t_c or t_{of}, whichever, is less (*see* Fig. 40).

The drag coefficient, C_d, for different faces of closed rectangular structures located above ground shall be taken as per Table 51.

The clearing time, t_{cp}, for a point on the front wall is given by

$$t_{cp} = \frac{3S}{U}$$

where

$S = H$ or $W/2$ whichever, is less (*see* Fig. 39); and

U = shock front velocity to be obtained from using Fig. 30 and Fig. 31 for spherical and hemispherical detonations, respectively.

The average clearing time, t_c, required to clear the front face of reflection effects from the roof down and inwards from the sides shall be calculated using the following expression:

$$t_c = \frac{4HW}{C_r(W+2H)}$$

where

C_r = velocity of sound in the reflected overpressure region that shall be determined using Fig. 33.

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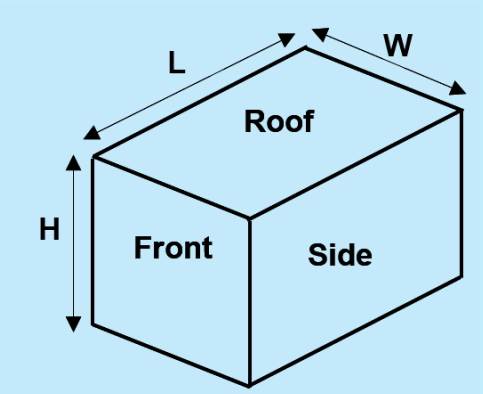

FIG. 39 ABOVE GROUND RECTANGULAR STRUCTURE

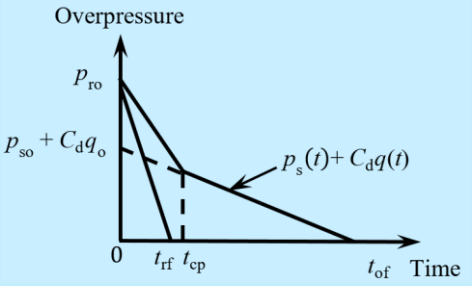

FIG. 40 REFLECTED OVERPRESSURE AT A POINT ON THE FRONT SURFACE OF FINITE REFLECTING SURFACE

The average net loading on the front face ($W \times H$) as a function of time is shown in Fig. 41 depending on whether t_c is smaller than or equal to t_{of}. The pressures p_{ro}, p_{so} and q, the impulses t_{ro} and I_{so} are for the actual explosion that are determined according to **L-6.3**.

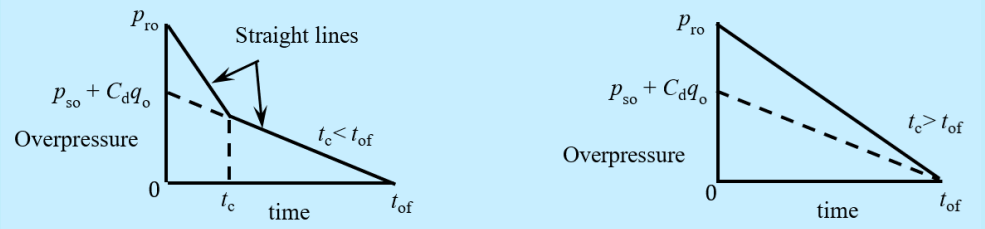

FIG. 41 REFLECTED OVERPRESSURE HISTORY ON THE FRONT FACE OF A CLOSED RECTANGULAR STRUCTURE

L-7.1.2.2 *Rear face*

Using the pressures for the actual explosion, the average loading on the rear face ($W \times H$) in Fig. 39 shall be taken as shown in Fig. 42, where the time has been taken from the instant the shock first strikes the front face. The time intervals of interest are the following:

$\frac{L}{U}$ = the travel time of shock from front to rear face; and

$\frac{4S}{U}$ =pressure rise time on the back face.

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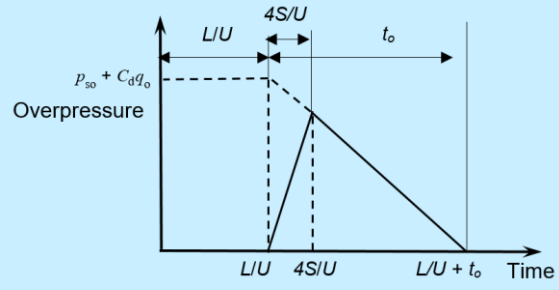

FIG. 42 OVERPRESSURE VERSUS TIME FOR REAR FACE

L-7.1.2.3 *Roof and side walls*

The average pressure versus time curve for roof and side walls is given in Fig. 43(A) when t_d is greater than the transit time $t_t = L/U$. When t_t is greater than t_d, the load on roof and side walls may be considered as a moving triangular pulse having the peak value of overpressure $p_{so}+C_d q_o$ and time t_o as shown in Fig. 43(B).

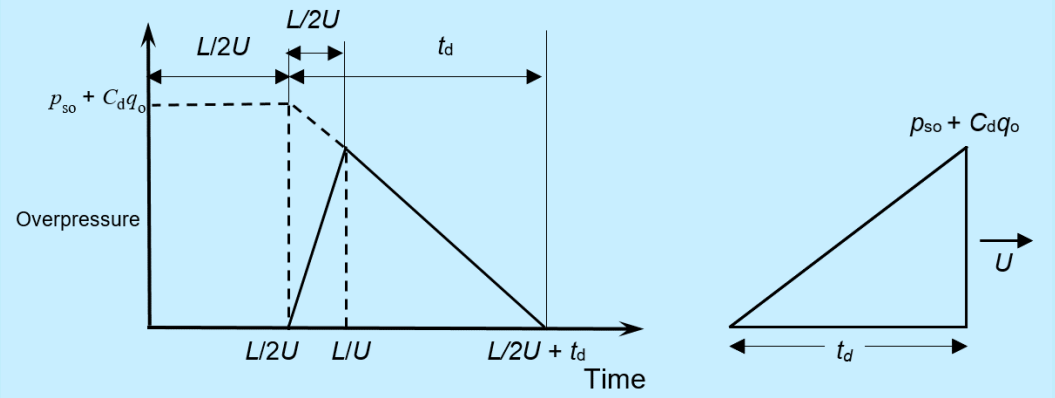

FIG. 43(A) AVERAGE PRESSURE DIAGRAM $t_d > t_t$ FIG. 43(B) MOVING PRESSURE PULSE FOR $t_t > t_d$

FIG. 43 OVERPRESSURE VERSUS TIME FOR ROOF AND SIDE WALLS

Table 51 Drag Coefficient C_d
(*Clause* L-7.1.2.1)

Sl No.	Shape of Element	Drag Coefficient C_d	Remarks
(1)	(2)	(3)	(4)
i)	Front vertical face	1.0	
ii)	Roof, near and side faces for:		For closed rectangular structures located above ground
	a) q_o = 0 kPa to 170 kPa	- 0.4	
	b) q_o = 170 kPa to 350 kPa	- 0.3	
	c) q_o = 350 kPa to 1 MPa	- 0.2	

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L-7.1.2.4 *Overturning of structure*

The average net load as a function of time which tends to cause sliding and overturning of the building is obtained by subtracting the loading on back face from that on the front face.

L-7.1.3 *Closed Cylindrical Arch-Shape Structures*

L-7.1.3.1 *Gable ends*

The loading may be taken same as for front and rear faces of above ground rectangular structure.

L-7.1.3.2 *Curved surface*

The direction of shock wave propagation is taken transverse to the ridge of the structure and since the usual arch spans are large so that the transit time t_t is greater than the positive phase time t_d, average loading condition cannot be assumed. Therefore, the loading on curved surface may be taken as a moving triangular pulse as shown in Fig. 43(B).

L-7.1.3.3 *Closed dome structures*

The loading on a domical structure may be taken as a moving triangular pressure pulse as shown in Fig. 43(B). The variation of pressure transverse to the direction of propagation of the pulse may be considered symmetrical varying according to cosine θ where the angle θ is measured from longitudinal vertical section of the dome, as shown in Fig. 44.

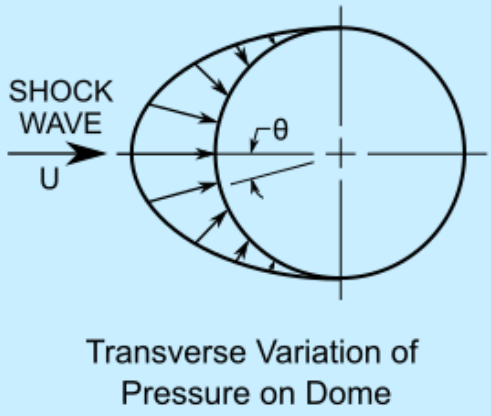

FIG. 44 TRANSVERSE VARIATION OF PRESSURE ON DOME

L-7.2 Blast Load on Below-Ground Structures

L-7.2.1 *Types of Structures*

The below-ground structures are classified into buried and semi-buried structures depending upon the earth cover and slopes of earth berms. The buried structures are subjected only to the general overpressure p_{so}, the reflected and dynamic pressures being neglected. The semi-buried structures are subjected to partial dynamic pressures besides the general overpressure. Both are acted upon by air-induced ground shock also.

For assessing the effects of above-ground blasts on below-ground structures, dynamic finite element analysis can be performed considering the flexibility of soil and interactions of blast induced ground waves with the structure.

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L-8 RESPONSE OF STRUCTURAL ELEMENTS

L-8.1 Properties of Materials

The nominal properties of materials are obtained as per relevant codes and guidelines. The change in material properties due to high strain rates associated with blast response are to be incorporated using the provisions in L-8.1.1 and L-8.1.2.

L-8.1.1 *Elastic Modulus and Poisson's Ratio*

The elastic modulus and Poisson's ratio of the constituent materials used in the structural elements shall be estimated based on relevant standards as applicable to that material.

No increase in elastic modulus and Poisson's ratio is to be considered for the calculation of blast response of structures.

L-8.1.2 *Dynamic Strength*

L-8.1.2.1 *Design strength of structural steel*

The average dynamic increase factors of strength for structural, carbon, mild, weldable or rivet steels and high strength alloy steels are listed in Table 52 and Table 53.

Table 52 Dynamic Increase Factors for Mild Steels

(*Clause* L-8.1.2.1)

Sl No.	Material	Dynamic Increase Factors
(1)	(2)	(3)
i)	Yield strength in bending and shear	1.25
ii)	Yield strength in tension and compression	1.20
iii)	Ultimate strength	1.00

Table 53 Dynamic Increase Factors for High Strength Steels

(*Clause* L-8.1.2.1)

Sl No.	Material	Dynamic Increase Factors
(1)	(2)	(3)
i)	Yield strength in bending and shear	1.10
ii)	Yield strength in tension and compression	1.05
iii)	Ultimate strength	1.00

L-8.1.2.2 *Design strength of reinforced concrete*

The average dynamic strength increase factors for concrete are listed in Table 54 and for reinforcing steel are listed in Table 55.

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Table 54 Dynamic Increase Factors for Reinforced Concrete

(*Clause* L-8.1.2.2)

Sl No.	Strength	Dynamic Increase Factors
(1)	(2)	(3)
i)	Flexure	1.15
ii)	Compression	1.10
iii)	Diagonal tension, direct shear and bond	1.00

Table 55 Dynamic Increase Factors for Reinforcing Steel

(*Clause* L-8.1.2.2)

Sl No.	Strength	Dynamic Increase Factors
(1)	(2)	(3)
i)	Yield strength in flexure	1.2
ii)	Yield strength in compression, diagonal tension, direct shear and bond	1.1
iii)	Ultimate strength in flexure	1.1
iv)	Ultimate strength in compression, diagonal tension, direct shear and bond	1.0

L-8.1.2.3 *Design strength for masonry or plain concrete*

The average dynamic strength increase factors for masonry and plain concrete are listed in Table 56.

Table 56 Dynamic Increase Factors for Masonry and Plain Concrete

(*Clause* L-8.1.2.3)

Sl No.	Strength	Dynamic Increase Factors
(1)	(2)	(3)
i)	Flexure	1.15
ii)	Compression	1.10
iii)	Diagonal tension and direct shear and bond	1.00

L-8.1.3 *Design Strength*

For obtaining design strength, the resistance of various elements is to be estimated using the strength increased by the dynamic increase factors mentioned in Q-8.1.2.

L-8.1.4 *Deformation Limits*

Structural elements subjected to blast loading can be designed subject to certain allowable dynamic ductility ratios and support rotations.

The blast resistance of a flexural member is to be expressed in terms of maximum permissible deflection. Support rotation and ductility is to be used to characterize the blast resistance of reinforced concrete and structural steel members, respectively.

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The support rotation shall be defined as per Fig. 45.

The maximum permissible deflection defines the energy absorption capacity of the element which is equal to the area under the resistance versus deflection curve. For a given blast scenario, greater the permissible deflection, lesser will be the maximum resistance required in the member.

The allowable deformation parameters are included in Table 57.

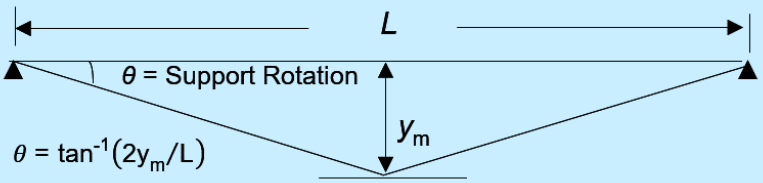

FIG. 45 SUPPORT ROTATION IN A FLEXURAL MEMBER

Table 57 Allowable Deformation Parameters

(*Clause* L-8.1.4)

Sl No.	Material	Low Damage	Medium Damage	High Damage
(1)	(2)	(3)	(4)	(5)
i)	Ductility ratio in structural steel members subjected to bending and direct stresses	3	10	20
ii)	Support rotation in reinforced concrete members subjected to bending and direct stresses (in degrees)	1	2	4

L-8.2 Modelling of Structural Members

L-8.2.1 *Geometry, Material and Boundary Conditions*

The methods of modelling are applicable to structural members of regular prismatic sections.

L-8.2.2 *Equivalent Single Degree of Freedom (SDOF) Model*

The structural element or system may be replaced by an equivalent single spring-mass system having effective stiffness k_E and effective mass equal to $K_{LM}M_t$, where M_t is the actual mass of the member under consideration and K_{LM} is a load-mass factor depending upon the stiffness and mass distribution in the member and its boundary conditions. The equivalent system is defined so that the deflection of the equivalent single mass is the same as that of some significant point in the given structure. The effective stiffness k_E is defined with respect to the deflection of this point.

The load-mass factor K_{LM} is equal to the ratio of mass factor K_M to the load factor K_L. The factors are evaluated on the basis of an assumed deflected shape of the structure as given below:

$$K_M = \frac{1}{M_t}\left\{\sum_{r=1}^{n} M_r \emptyset_r^2 + \int m\emptyset^2(x)\,dx\right\}$$

$$K_L = \frac{1}{P_t}\left\{\sum_{r=1}^{j} p_r \emptyset_r + \int p(x)\emptyset(x)dx\right\}$$

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where

n = number of concentrated masses;

M_r = concentrated mass at point r ;

M_t = total actual mass;

$\phi_r\phi_r$ = deflection at point r of an assumed deflected shape for concentrated loads;

m = distributed mass intensity per unit length;

$\phi_{(X)}$ = deflection at point x of an assumed deflected shape for distributed loads;

P_t = total dynamic load (at any instant of time);

j = number of concentrated load points;

P_r = concentrated dynamic force at point r; and

$p_{(X)}$ = intensity of distributed dynamic load per unit length.

L-8.2.2.1 *Effective single SDOF parameters*

Values of the factors k_E and K_{LM} for few structural members are given in Table 58 to Table 60. The effective SDOF parameters for other type of structural members shall be calculated by assuming an appropriate deflected shape as per **L-8.2.2.2**.

L-8.2.2.2 *Deflected shape*

The deflected shape is suitably chosen to resemble as far as possible the true deflected shape taking into consideration whether the structure or member remains elastic or goes into the plastic range. It may be taken the same as due to static application of the dynamic load on the structure. The deflected shape is normalized such that φ(x)=1 at the point with respect to which the effective stiffness k_E is defined.

L-8.2.2.3 *Effective time-period*

The effective time period T of the structural member may be calculated from the equation:

$$T = 2\pi \sqrt{\frac{K_{LM} M_t}{k_E}}$$

L-8.2.3 *Resistance Function*

The resistance versus deflection diagram of a structural element shall be idealized as elasto-plastic (*see* Fig. 46) by keeping the area under the actual and idealized curves about the same up to the maximum permissible deflection defined as per **L-8.1.4**.

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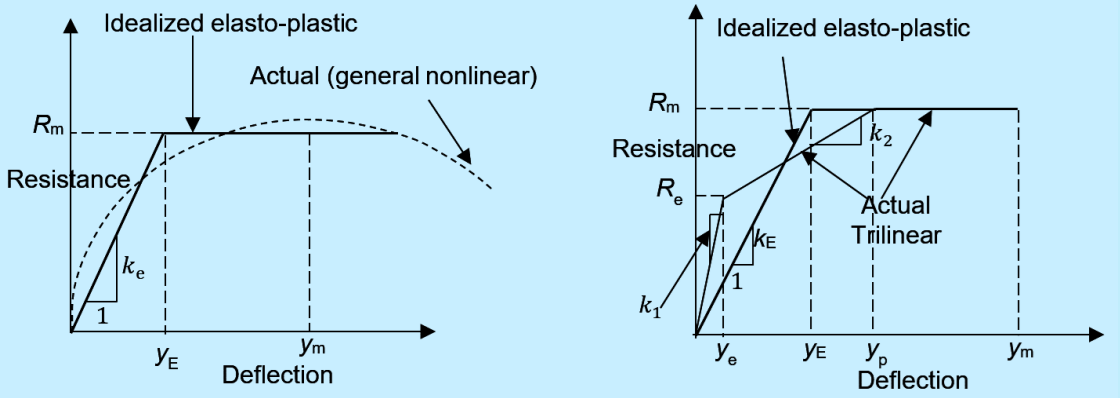

FIG. 46(A) GENERAL NONLINEAR FIG. 46(B) TRILINEAR

FIG. 46 IDEALIZATION OF RESISTANCE DEFLECTION DIAGRAM

A trilinear resistance versus deflection diagram may be simplified to its elasto-plastic (bilinear) equivalent of same peak resistance (R_m) and the elastic parameters calculated using following equations:

$$y_E = y_e + y_p\left(1 - \frac{R_m}{R_e}\right)$$

$$k_E = \frac{R_m}{y_E}$$

L-8.3 Methods of Analysis

These provisions apply to structures, components and systems subject to near-field and far-field detonations but not for contact or near-contact detonations.

L-8.3.1 *Analysis of Members*

Blast loaded structural components of a structure may be analysed independently provided it can be demonstrated that such independent member analysis would yield conservative performance of the member and the structure.

L-8.3.1.1 *Equivalent SDOF analysis*

The equivalent SDOF analysis may be used to determine response of structural members subject to blast loads when the following criteria are satisfied:

a) The equivalent mass, stiffness and damping parameters in the model must capture the dynamic response and failure mode of the structural member being analysed; and

b) A single flexural deformation mode controls the dynamic response of the structural system.

The equivalent SDOF analysis shall make use of following parameters:

a) Blast-pressure history obtained using L-6.3;

b) The effective time-period calculated using L-8.2.2.3;

c) Resistance function of the structural element established using provisions of L-8.2.3; and

d) Maximum permissible deflection provided in L-8.1.4.

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L-8.3.1.2 *Blast overpressure history*

The exponential blast pressure history of Fig. 28 shall be idealized to a triangular pulse [Fig. 47(A) as per provisions of **L-6.5** for the purpose of obtaining equivalent SDOF blast loads. Such idealization may neglect the effect of negative phase, unless otherwise it is expected to influence the flexure response of the structural member.

The positive phase duration of the triangular loading shall be calculated as:

$$t_d = \frac{2i_{so}}{p_{so}}$$

If the influence of clearing on the blast pressure history applied to the front face of the structural element is found to be significant, then the equivalent SDOF blast loads shall be idealized to Fig. 47(B) using **L-7.1.2**.

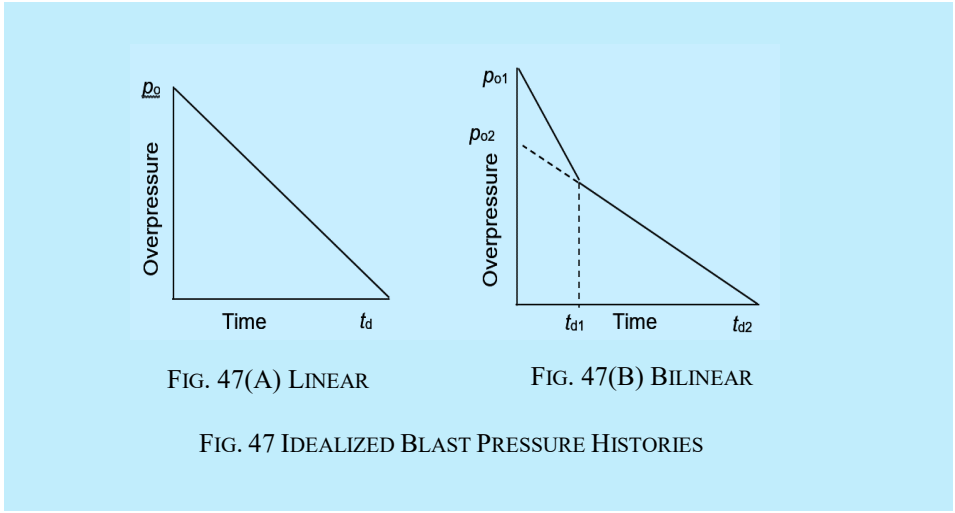

FIG. 47(A) LINEAR FIG. 47(B) BILINEAR

FIG. 47 IDEALIZED BLAST PRESSURE HISTORIES

When the ratio of time duration t_{d1}, or t_{d2}, to the natural period of the element is less than 0.1, the problem may be considered as an impulse problem taking the area under the pressure versus time curve as impulse per unit area. In such a case, the shape of pressure-time curve is not important.

L-8.3.1.3 *Uniform blast pressure distribution*

A uniform distribution of blast pressure over the length of the structural member may be considered provided one of the following conditions are satisfied:

a) The minimum scaled distance to the structural component is greater than 1.2 m/kg$^{1/3}$; and

b) The variation of peak reflected pressures over the middle two-third of the component span length is less than 25 percent.

For uniform distribution of blast pressure, the pressure value calculated at the closest point on the structure from the centre of detonation may be considered over the whole reflecting surface.

L-8.3.1.4 *Non-uniform blast pressure distribution*

For members subjected to spatially non-uniform blast loads, the effect of non-uniformity on the response shall be considered.

These provisions shall be applicable for non-uniform blast pressure distribution, provided the conditions outlined in **L-8.3.1.3** are not satisfied and the minimum scaled distance to the structural component is between 0.4 m/kg$^{1/3}$ and 1.2 m/kg$^{1/3}$.

An equivalent impulse weighted using deflected shape function shall be considered for the structural component as per following expression:

$$i_e = \frac{\int_0^L \int_0^H i(x,y)\phi(x,y)dxdy}{\int_0^L \int_0^H \phi(x,y)dxdy}$$

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where

i_e = equivalent impulse of uniformly distributed load;

L = length of loaded area;

H = width of loaded area;

$\emptyset(x,y)$ = deflected shape function of blast-loaded component; and

i(x,y) = non-uniform impulse applied to loaded area.

The simplified triangular representation of the pressure time history shall be constructed as per provisions of L-6.5 by calculating the positive loading phase duration using the equivalent uniform impulse (i_e) and the peak overpressure at the midspan or geometric centre of the reflecting surface.

A safety factor of 1.25 shall be used if a non-uniform blast load is represented as an equivalent uniform impulse in a blast analysis.

L-8.3.1.5 *Elastic SDOF response*

For elastic analysis (ductility ratio $\mu \leq 1.0$) of structures, the effective stiffness k_E and load mass factor K_{LM} shall be used as given in Table 58 to Table 60.

For elastic response of an SDOF system, the peak response (y_m) and the time to peak response (t_m) may be obtained using following expressions:

$$\frac{y_m}{y_0} = 2 - \frac{t_m}{t_d}; \qquad \text{when } t < t_d$$

$$t_m = \frac{2}{\omega}\tan^{-1}(\omega t_d) \qquad \text{when } t < t_d$$

$$\frac{y_m}{y_0} = \frac{1}{\omega t_d}\sqrt{2(1-\cos\omega t_d - \omega t_d \sin\omega t_d) + (\omega t_d)^2}; \qquad \text{when } t > t_d$$

$$t_m = \frac{1}{\omega}\tan^{-1}\left(\frac{1-\cos\omega t_d}{\sin\omega t_d - \omega t_d}\right) \quad \text{when } t > t_d$$

where

y_m = peak elastic displacement;

y_o = peak static displacement; and

t_m = time to the peak displacement.

L-8.3.1.6 *Inelastic SDOF response*

For elasto-plastic analysis, k_E shall be used as given in Table 58 to Table 60 but value of K_{LM} may be chosen in between the elastic and plastic cases depending upon the ductility factor.

Based on the elasto-plastic resistance-deflection curve shown in Fig. 46 and triangular pressure-time curve shown in Fig. 47(A), the ratio of resistance R_m, required to the peak dynamic load (F_1) is given in Fig. 48 for various values of t_d/T and μ.

The time to peak deflection (t_m)shall be obtained from Fig. 49 for various values of td/T and μ.

When the time ratio, t_d/T, is less than 0.1, the ratio k may be computed from the following equation:

$$k = \frac{\pi}{\sqrt{2\mu - 1}} \cdot \frac{t_d}{T}$$

When the pressure-time diagram is given by Fig. 47B, the following equation shall be satisfied:

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$$\frac{P_{01}}{R_m} k_1 + \frac{P_{02}}{R_m} k_2 = 1$$

where

R_m = the required resistance; and

k_1, k_2 = the values of ratios k for ductility ratio μ and time ratios t_{d1}/T and t_{d2}/T respectively.

For elasto-plastic design of fixed slabs, the modified value of k_E is to be worked out in accordance with Fig. 46(B) using the stiffness values of slab in elastic and elasto-plastic cases as given in Table 60 and K_{LM} is to be suitably chosen depending upon the ductility factor.

The value of modular ratio shall be taken the same as in static design for calculating EI .

For calculating moment of inertia I of reinforced concrete sections, the concept of effective transformed area shall be used.

For analysis of reinforced concrete sections when the simplified elasto-plastic resistance function is used, the elastic stiffness shall be based on effective moment of inertia that shall be calculated as the average of the gross moment of inertia of the transformed section and the cracked moment of inertia.

The gross and cracked moment of inertias shall be obtained from Fig. 50.

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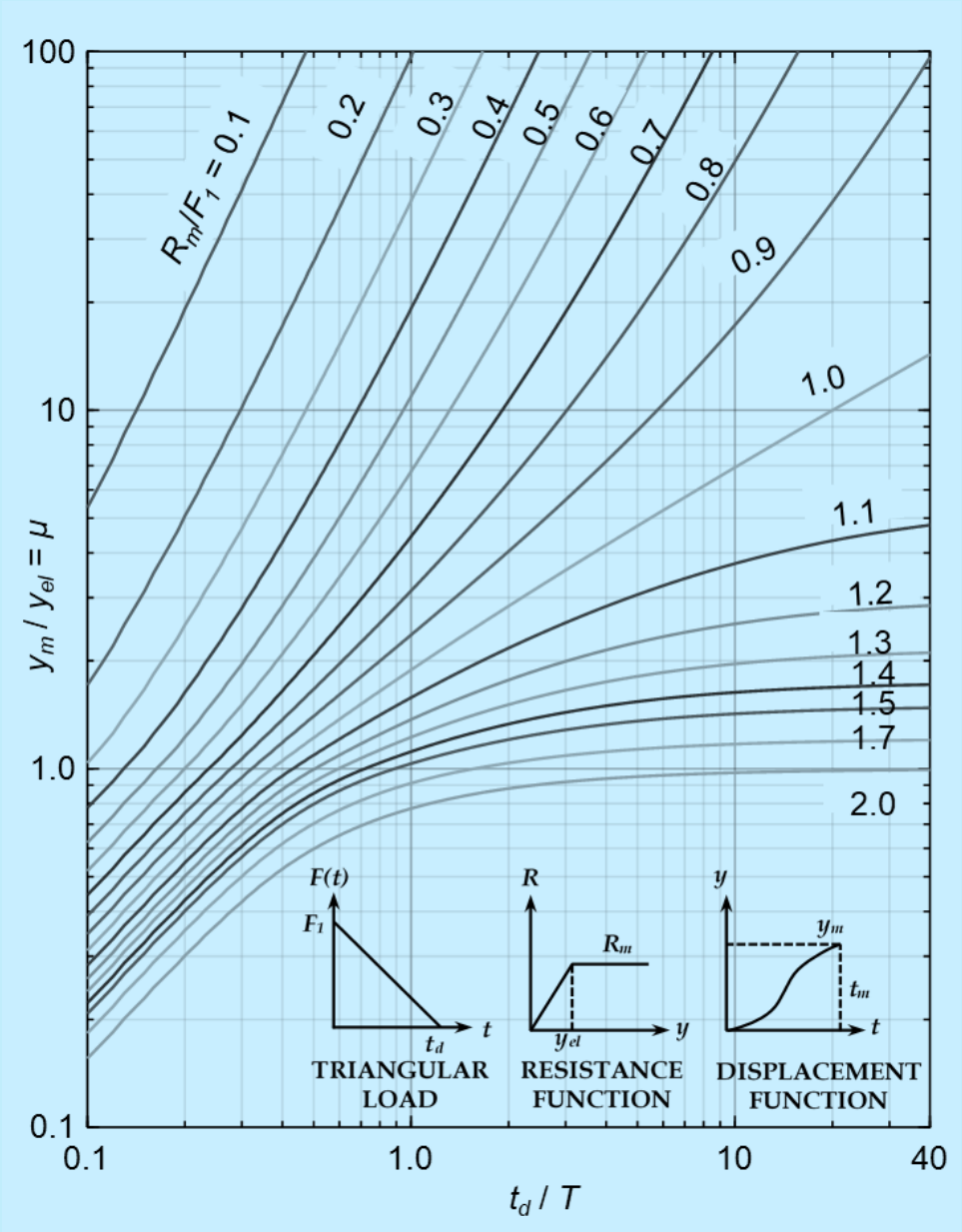

FIG. 48 PEAK DUCTILITY DEMAND OF SDOF SYSTEM SUBJECT TO TRIANGULAR PULSE LOADING

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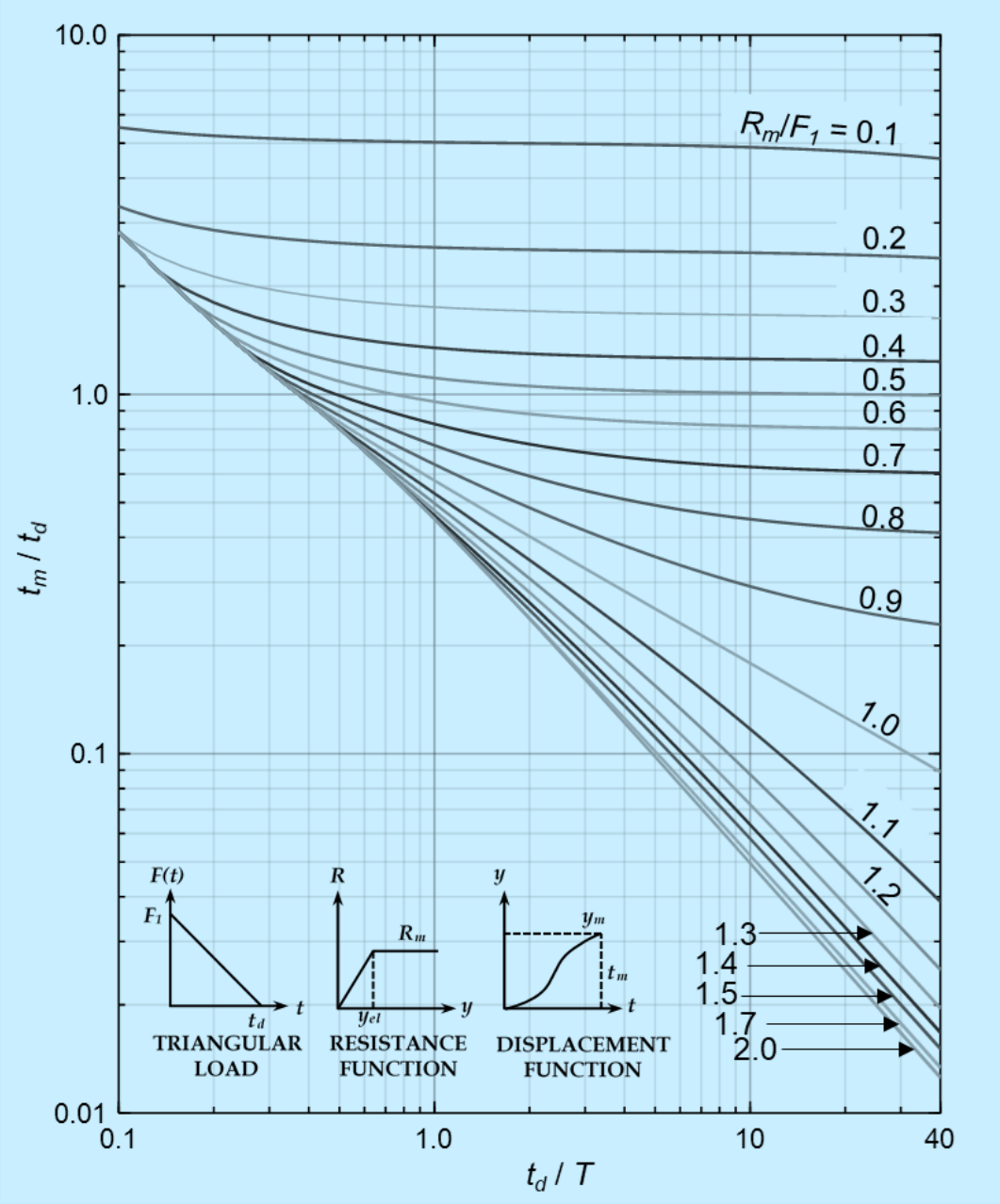

FIG. 49 TIME TO PEAK DUCTILITY DEMAND OF SDOF SYSTEM SUBJECT TO TRIANGULAR PULSE

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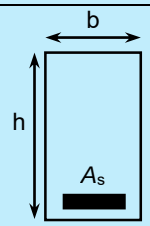
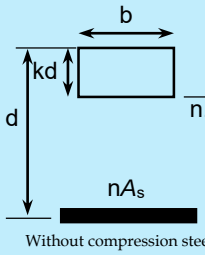
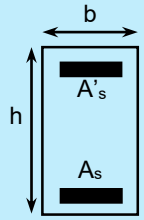
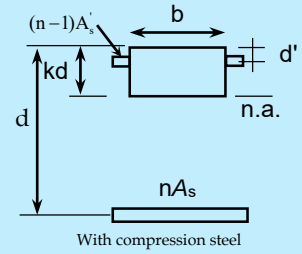
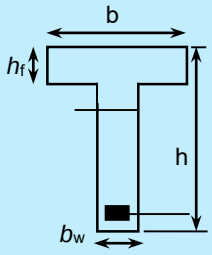
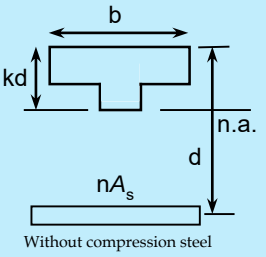
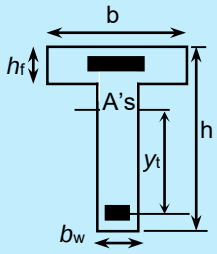
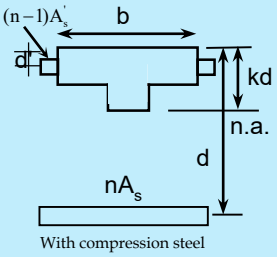
Gross Section	*Cracked Transformed Section*	Gross and Cracked Moment of Inertia
	 Without compression steel	$n = \frac{E_s}{E_c}$ $B = \frac{b}{(nA_s)}$ $I_g = \frac{bh^3}{12}$ Without compression steel: $kd = (\sqrt{2dB + 1} - 1)/B$ $I_{cr} = bk^3d^3/3 + nA_s(d - kd)^2$
	 With compression steel	With compression steel: $r = (n - 1)A'_s/(nA_s)$ $kd = \left[\sqrt{2dB(1 + rd'/d) + (1 + r)^2} - (1 + r)\right]/B$ $I_{cr} = bk^3d^3/3 + nA_s(d - kd)^2 + (n - 1)A'_s(kd - d')^2$
	 Without compression steel	$n = \frac{E_s}{E_c}$ $C = b_w/(nA_s),\ f = h_f(b - b_w)/(nA_s)$ $y_t = h\text{-}1/2\left[(b - b_w)h_f^2 + b_wh^2\right]/[(b - b_w)h_f + b_wh],$ $I_g = (b - b_w)h_f^3/12 + b_wh^3/12 + (b - b_w)h_f\,(h - h_f/2 - y_t)^2 + b_wh(y_t - h/2)^2$ Without compression steel: $kd = \left[\sqrt{C(2d + h_ff) + (1 + f)^2} - (1 + f)\right]/C$ $I_{cr} = (b - b_w)h_f^3/12 + b_wk^3d^3/3 + (b - b_w)h_f\,(kd - h_f/2)^2 + nA_s(d - kd)^2 + (n - 1)A_s(kd - d')2$
	 With compression steel	With compression steel: $kd = \left[\sqrt{C(2d + h_ff + 2rd') + (f + r + 1)^2} - (f + r + 1)\right]/C$ $I_{cr} = (b - b_w)h_f^3/12 + b_wk^3d^3/3 + (b - b_w)h_f\,(kd - h_f/2)^2 + nA_s(d - kd)^2 + (n - 1)\,A'_s(kd - d')2$

FIG. 50 CRACKED MOMENT OF INERTIA OF PRISMATIC SECTIONS

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L-8.3.2 *MDOF Analysis*

The multi degree of freedom (MDOF) analysis is permitted for structural systems which cannot be analysed using the simplified SDOF analysis, provided the expected failure mechanism under blast load can adequately be represented using the MDOF model representation.

L-8.3.2.1 The numerical model of MDOF system may comprise of distributed elements or combined lumped mass and stiffness elements.

L-8.3.2.2 The spring in the lumped-mass stiffness should adequately represent the member resistance-deflection behaviour by incorporating appropriate constitutive relationship.

L-8.3.2.3 The lumped-mass stiffness system should be able to properly account for the dynamic behaviour and failure modes at the structure and the element level.

L-8.3.2.4 The use of structural damping is permitted for MDOF system when the expected response is within elastic range. These structural damping values for elastic response shall conform to the typical values used for different type of structures.

L-8.3.2.5 The energy dissipation during inelastic response should be accounted by using appropriate nonlinear member resistance-deflection curves. Supplemental damping in form of structural damping is not permitted for nonlinear range of response.

L-8.3.2.6 The time-step for MDOF analysis shall be determined based on accuracy and convergence requirement. The time-step should be sufficiently small such that it captures the rise and decay of the applied blast load time history.

L-8.3.3 *Finite Element Analysis*

Advanced numerical analysis techniques, such as explicit dynamic finite element analysis (FEA), may be used to analyse structures subject to blast loading, if the methods described in **L-8.3.1** and **L-8.3.2** cannot be reliably used to obtain the response. Such methods shall be required if one or more of the following conditions are encountered:

a) detonation involves scaled distances smaller than 0.4 m/kg$^{1/3}$;

b) detonation of non-spherical charge masses at scaled distances smaller than 3 m/kg$^{1/3}$;

c) the blast wave propagation involves interaction with surrounding environment before it reaches the location of interest;

d) the variation in geometric and material properties of structural elements or the structure cannot appropriately be represented using the simplified methods; and

e) the interaction between the blast wave and structure is of significance and the assumption of rigid reflecting surface is not valid.

The FEA shall consider the modelling of non-ductile failure modes and failure surfaces that cannot be considered in the simplified SDOF and the MDOF methods of analysis.

The time-step of analysis need to be decided based on size of the smallest element of interest with appropriate consideration to the accuracy and convergence of the response.

The interaction of blast load with the surrounding shall be considered using computational fluid dynamics (CFD) methods.

Where the shape and size of the charge mass is expected to play a major role in the detonation process and the resulting blast load, the detonation process itself may be modelled.

The FEA methods shall not be subjected to the response limits typically used for structural members in the simplified SDOF and the MDOF analysis.

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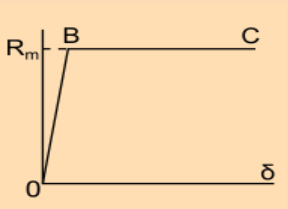
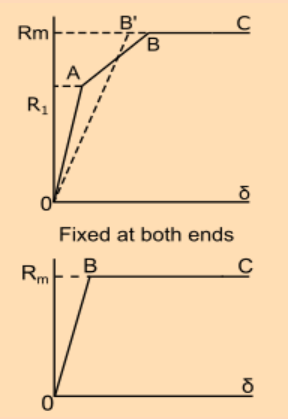
Table 58 Transformation Factors for Beams and One-way Slabs

(*Clauses* L-8.2.2.1, L-8.3.1.5 *and* L-8.3.1.6)

Sl No.	Resistance Curve, End-Conditions of Beams or One-way Slabs	Dynamic Loading	Strain Range	Load-Mass Factor K_{LM}		Maximum Resistance	Effective Spring Constant	Dynamic Reaction
				Concentrated Mass	Uniform Mass	R_m	k_E	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)		Uniform $P_t = pL$	Elastic	–	0.78	$8M_p/L$	$384EI/5L^3$	$0.39\ R_m + 0.11\ P_t$
			Plastic	–	0.66	$8M_p/L$	–	$0.38\ R_m + 0.12\ P_t$
		Concentrated at mid-span $P_t = P$	Elastic	1.0	0.49	$4M_p/L$	$48EI/L^3$	$0.78\ R_m - 0.28\ P_t$
			Plastic	1.0	0.33	$4M_p/L$	–	$0.75\ R_m - 0.25\ P_t$
		Concentrated at third points P/2 each $P_t = P$	Elastic	0.87	0.60	$6M_p/L$	$56.4EI/L^3$	$0.62\ R_m - 0.12\ P_t$
			Plastic	1.0	0.56	$6M_p/L$	–	$0.52\ R_m - 0.02\ P_t$
ii)		Uniform $P_t = pL$	Elastic (OA)	–	0.77	$12M_{ps}/L$	$384EI/L^3$	$0.36\ R_1 + 0.14\ P_t$
			Elastic (OB')	–	0.78	$(8/L)(M_{ps} + M_{pm})$	$307EI/L^3$	$0.39\ R_m + 0.11\ P_t$
			Plastic	–	0.66	$(8/L)(M_{ps} + M_{pm})$	–	$0.38\ R_m + 0.12\ P_t$
		Concentrated at mid-span $P_t = P$	Elastic (OB)	1.0	0.37	$(4/L)(M_{ps} + M_{pm})$	$192EI/L^3$	$0.71\ R_m - 0.21\ P_t$
			Plastic	1.0	0.33	$(4/L)(M_{ps} + M_{pm})$	–	$0.75\ R_m - 0.25\ P_t$
iii)	Fixed at one end and simply supported at the other	Uniform $P_t = pL$	Elastic (OA)	–	0.78	$(8/L)(M_{ps})$	$185EI/L^3$	$V_1 = 0.26R_1 + 0.12P_t$ $V_2 = 0.43R_1 + 0.19P_t$

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			Elastic (OB')	–	0.78	$(4/L)(M_{ps} + 2M_{pm})$	$160EI/L^3$	$0.39\ R_m + 0.11$
			Plastic	–	0.66	$(4/L)(M_{ps} + 2M_{pm})$	–	$0.38\ R_m + 0.12$
		Concentrated at mid-span $P_t = P$	Elastic (OA)	1.0	0.43	$(16/3L)M_{ps}$	$107EI/L^3$	$V_1 = 0.54R_1 + 0.14P_t$ $V_2 = 0.25R_1 + 0.07P_t$
			Elastic (OB')	1.0	0.49	$(2/L)(M_{ps} + 2M_{pm})$	$106EI/L^3$	$0.78\ R_m - 0.28\ P_t \pm M_{ps}/L$
			Plastic	1.0	0.33	$(2/L)(M_{ps} + 2M_{pm})$	–	$0.75\ R_m - 0.25\ P_t \pm M_{ps}/L$

L = span;
p = dynamic pressure;
P_t = dynamic load;
M_p = plastic moment of section;
M_{pm} = M_p at mid-span
M_{ps} = M_p at support;
R_m = maximum resistance; and
k_E = effective spring constant.

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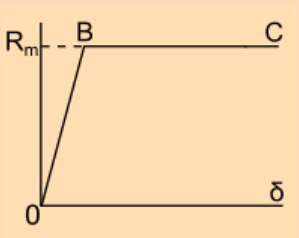
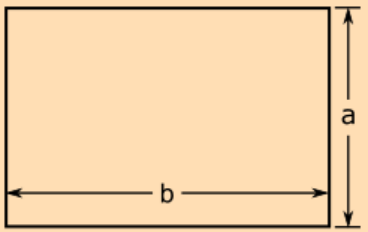
Table 59 Transformation Factors for Two-way Slabs — Four Sides, Simply Supported, Uniformly Loaded

(*Clauses* L-8.2.2.1, L-8.3.1.5 *and* L-8.3.1.6)





Sl No.	Strain Range	a/b	Load Mass Factor K_{LM}	Max Resistance R_m	Spring Constant k_E	Dynamic Reactions	
						V_A	V_B
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Elastic (OB)	1.0	0.68	$12(M_{pfa} + M_{pfb})/a$	$252EI/a^2$	$0.07P_r + 0.18R_m$	$0.07P_r + 0.18R_m$
		0.9	0.70	$(12M_{pfa} + 11M_{pfb})/a$	$230EI/a^2$	$0.06P_r + 0.16R_m$	$0.08P_r + 0.20R_m$
		0.8	0.71	$(12M_{pfa} + 10.3M_{pfb})/a$	$212EI/a^2$	$0.06P_r + 0.14R_m$	$0.08P_r + 0.22R_m$
		0.7	0.73	$(12M_{pfa}+9.8M_{pfb})/a$	$201EI/a^2$	$0.05P_r + 0.13R_m$	$0.08P_r + 0.24R_m$
		0.6	0.74	$(12M_{pfa} + 9.3M_{pfb})/a$	$197EI/a^2$	$0.04P_r + 0.11R_m$	$0.09P_r + 0.26R_m$
		0.5	0.75	$(12M_{pfa} + 9.0M_{pfb})/a$	$201EI/a^2$	$0.04P_r + 0.09R_m$	$0.09P_r + 0.28R_m$
ii)	Plastic (BC)	1.0	0.51	$12(M_{pfa} + M_{pfb})/a$	-	$0.09P_r + 0.16R_m$	$0.09P_r + 0.16R_m$
		0.9	0.51	$(12M_{pfa} + 11M_{pfb})/a$	-	$0.08P_r + 0.15R_m$	$0.09P_r + 0.18R_m$
		0.8	0.54	$(12M_{pfa} + 10.3M_{pfb})/a$	-	$0.07P_r + 0.13R_m$	$0.10P_r + 0.20R_m$
		0.7	0.58	$(12M_{pfa} + 9.8M_{pfb})/a$	-	$0.06P_r + 0.12R_m$	$0.10P_r + 0.22R_m$
		0.6	0.58	$(12M_{pfa} + 9.3M_{pfb})/a$	-	$0.05P_r + 0.10R_m$	$0.10P_r + 0.25R_m$
		0.5	0.59	$(12M_{pfa} + 9.0M_{pfb})/a$	-	$0.04P_r + 0.08R_m$	$0.11P_r + 0.27R_m$

M_{pfa} = total positive ultimate moment capacity along mid-span section parallel to short edge;

M_{pfb} = total positive ultimate moment capacity along mid-span section parallel to long edge;

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I = moment of inertia per unit width of slab;

P_t = dynamic load;

V_A = total dynamic reaction along a short edge; and

V_B = total dynamic reaction along a long edge.

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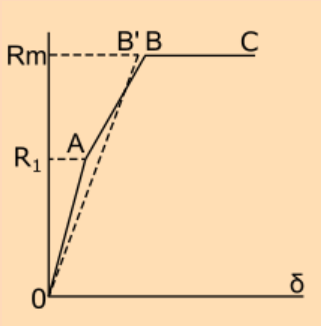
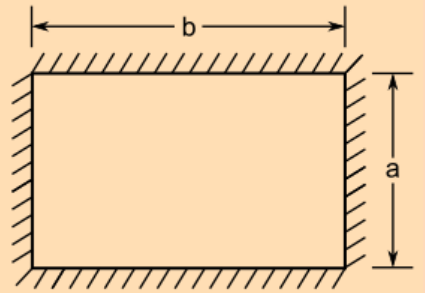
Table 60 Transformation Factors for Two-way Slabs — Fixed four sides, Uniform loaded

(*Clauses* L-8.2.2.1, L-8.3.1.5 *and* L-8.3.1.6)





Sl No.	Strain Range	a/b	Load Mass Factor K_{LM}	Max Resistance R_m	Strain Range	Spring Constant k_E	Dynamic Reactions	
							V_A	V_B
(1)	(2)	(3)	(4)	(5)	(6)	(7)	*(8)*	*(9)*
i)	Elastic (OA)	1.0	0.63	29.2 M^o_{psb}	810 EI/a^2	$0.10P_t + 0.15R_1$	$0.10P_t + 0.15R_1$	1.0
		0.9	0.68	27.4 M^o_{psb}	742 EI/a^2	$0.09P_t + 0.14R_1$	$0.10P_t + 0.17R_1$	0.9
		0.8	0.69	26.4 M^o_{psb}	705 EI/a^2	$0.08P_t + 0.12R_1$	$0.11P_t + 0.19R_1$	0.8
		0.7	0.71	26.2 M^o_{psb}	692EI/a^2	$0.07P_t + 0.11R_1$	$0.11P_t + 0.21R_1$	0.7
		0.6	0.71	27.3 M^o_{psb}	724 EI/a^2	$0.06P_t + 0.09R_1$	$0.12P_t + 0.23R_1$	0.6
		0.5	0.72	30.2 M^o_{psb}	806 EI/a^2	$0.05P_t + 0.08R_1$	$0.12P_t + 0.25R_1$	0.5
ii)	Elasto-plastic (OB')	1.0	0.67	$(1/a)[12(M_{pfa} + M_{psa}) + 12(M_{pfb} + M_{psb})]$	252 EI/a^2	$0.07P_t + 0.18R_m$	$0.07P_t + 0.18R_m$	1.0
		0.9	0.70	$(1/a)[12(M_{pfa} + M_{psa}) + 11(M_{pfb} + M_{psb})]$	230 EI/a^2	$0.06P_t + 0.16R_m$	$0.08P_t + 0.20R_m$	0.9
		0.8	0.71	$(1/a)[12(M_{pfa} + M_{psa}) + 10.3(M_{pfb} + M_{psb})]$	212 EI/a^2	$0.06P_t + 0.14R_m$	$0.08P_t + 0.22R_m$	0.8
		0.7	0.73	$(1/a)[12(M_{pfa} + M_{psa}) + 9.8(M_{pfb} + M_{psb})]$	201 EI/a^2	$0.05P_t + 0.13R_m$	$0.08P_t + 0.24R_m$	0.7
		0.6	0.74	$(1/a)[12(M_{pfa} + M_{psa}) + 9.3(M_{pfb} + M_{psb})]$	197 EI/a^2	$0.04P_t + 0.11R_m$	$0.09P_t + 0.26R_m$	0.6
		0.5	0.75	$(1/a)[12(M_{pfa} + M_{psa}) + 9.0(M_{pfb} + M_{psb})]$	201 EI/a^2	$0.04P_t + 0.09R_m$	$0.09P_t + 0.28R_m$	0.5

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iii)	Fully plastic (BC)	1.0	0.51	$(1/a)[12(M_{pfa}+M_{psa})+12(M_{pfb}+M_{psb})]$	–	$0.09P_t+0.16R_m$	$0.09P_t+0.16R_m$	1.0
		0.9	0.51	$(1/a)[12(M_{pfa}+M_{psa})+11(M_{pfb}+M_{psb})]$	–	$0.08P_t+0.15R_m$	$0.09P_t+0.18R_m$	0.9
		0.8	0.54	$(1/a)[12(M_{pfa}+M_{psa})+10.3(M_{pfb}+M_{psb})]$	–	$0.07P_t+0.13R_m$	$0.10P_t+0.20R_m$	0.8
		0.7	0.58	$(1/a)[12(M_{pfa}+M_{psa})+9.8(M_{pfb}+M_{psb})]$	–	$0.06P_t+0.12R_m$	$0.10P_t+0.22R_m$	0.7
		0.6	0.58	$(1/a)[12(M_{pfa}+M_{psa})+9.3(M_{pfb}+M_{psb})]$	–	$0.05P_t+0.10R_m$	$0.10P_t+0.25R_m$	0.6
		0.5	0.59	$(1/a)[12(M_{pfa}+M_{psa})+9.0(M_{pfb}+M_{psb})]$	–	$0.04P_t+0.08R_m$	$0.11P_t+0.27R_m$	0.5

Range (OA) = moment at centre of long edge just becomes plastic;

Range (OB') = moment at supports and mid-span sections just becomes plastic;

M^o_{psb} = negative ultimate moment capacity per unit width at centre of long edge;

M_{psa} = total negative ultimate moment capacity along a short edge support;

M_{psb} = total negative ultimate moment capacity along a long edge support;

M_{pfa}, M_{pfb}, I, P_t = (*see* Table 46);

V_A = total dynamic reaction along a short edge; and

V_B = total dynamic reaction along a long edge.

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L-9 LOAD COMBINATIONS FOR DESIGN

L-9.1 Partial Safety Factor

A partial safety factor of unity shall be assumed for blast loads.

L-9.2 Imposed loads

Imposed load on floors shall be considered as per 3. No Imposed load shall be considered on roof at the time of blast.

L-9.3 Load Combination

Wind or earthquake forces shall not be assumed to occur simultaneously with blast effects. Effects of temperature and shrinkage shall be neglected.

Following load combinations for limit state design shall be considered:

$$1.0DL + 0.5IL + 1.0B$$
$$0.9DL + 1.0B$$

where

DL = design dead load;

IL = design imposed load; and

B = design blast load.

L-10 For design of structures for blast effects of explosions above ground, *see also* good practice [C-1(16)].

For safety of structures during underground blasting, *see also* good practice [C-1(17)].

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ANNEX M

(*Clause* 10.2.2 *and Table* 61)

SUMMARY OF DISTRICTS HAVING SUBSTANTIAL MULTI-HAZARD RISK AREAS

State	*Name of Districts Having Substantial Multi-Hazard Prone Area*			
	E.Q. and Flood	Cyclone and Flood	E.Q., Cyclone and Flood	E.Q. and Cyclone
(1)	(2)	(3)	(4)	(5)
Andhra Pradesh	Adilabad, Karim Nagar, Khammam	Krishna, Nellore, Srikakulam, Visakhapatnam, Vizianagram	East Godavari, Guntur, Prakasam, West Godavari	—
Assam	All 22 districts listed in Table 61 could have M.S.K IX or more with flooding	No cyclone, but speed can be 50 m/s in districts of Table 61 causing local damage except Dhubri	—	—
Bihar	All 25 districts listed in Table 61	—	—	—
Goa	—	—	—	North and South Goa
Gujarat	Banaskantha, Danthe GS, Gandhinagar, Kheda, Mahesana, Panchmahals, Vadodara	—	Ahmedabad, Bharuch, Surat, Valsad	Amreli, Bhavnagar, Jamnagar, Rajkot, Junagad, Kachcha
Haryana	All 8 districts listed in Table 61	—	—	—
Kerala	Idduki, Kottayam, Palakkad, Pathanamthitta	—	Alappuzha, Ernakulum, Kannur, Kasargod, Kollam, Kozhikode, Malappuram, Thiruvanathapura, Thrissur	—
Maharashtra	—	—	—	Mumbai, Rayagad, Ratnagiri, Sindhudurg, Thane
Orissa	—	Ganjam	Baleshwar, Cuttack, Puri	Dhenkanal
Punjab	All 12 districts listed in Table 61	—	—	—
Uttar Pradesh and Uttarakhand	All 50 districts listed in Table 61	—	—	—
West Bengal	Birbhum, Darjeeling, Jalpaiguri, Cooch Behar, Malda, Murshidabad, West Dinajpur	—	Bardhaman, Kolkata, Hugli, Howrah, Midnapore, Nadia, North and South 24 Parganas	Bankura
Union Territories	Delhi	—	Yanam (Py)	Diu
India	**139 Districts**	**6 Districts**	**29 Districts**	**16 Districts**

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Table 61 Multi-Hazard Prone Districts

(*Annex* M)

Assam

Barpeta, Bongaigaon, Cachar[1)], Darrang, Dhemaji, Dhuburi, Dibrugarh, Goalpara, Golaghat, Hailaknadi[1)], Jorhat, Kamrup, Karbianglong, Karimganj[1)], Kokrajhar, Lakhimpur, Morigaon, Nagaon, Nalbari, Sibsagar, Sonitpur, Tinsukia

Bihar[2)]

Araria, Begusarai, Bhagalpur, Bhojpur, Darbhanga, Gopalganj, Katihar, Khagaria, Kishanganj, Madhepura, Madhubani, Munger, Muzaffarpur, Nalanda, Nawada, Paschim Champaran, Patna, Purbachamparan, Purnia, Samastipur, Saran, Saharsa, Sitamarhi, Siwan, Vaishali

Haryana[3)]

Ambala, Bhiwani, Faridabad, Gurgaon, Hisar, Jind, Kurukshetra, Rohtak

Punjab[4)]

Amritsar, Bathinda, Faridkot, Firozpur, Gurdaspur, Hoshiarpur, Jalandhar, Kapurthala, Ludhiana, Patiala, Rupnagar, Sangrur

Uttar Pradesh and Uttarakhand[5)]

Agra, Aligarh, Allahabad, Azamgarh, Bahraich, Ballia, Barabanki, Bareilly, Basti, Bijnor, Budaun, Bulandshahr, Deoria, Etah, Etawah, Faizabad, Farrukhabad, Fatehpur, Firozabad, Ghaziabad, Ghazipur, Gonda, Gorakhpur, Hardoi, Haridwar, Jaunpur, Kanpur (Dehat), Kanpur (Nagar), Kheri, Lucknow, Maharajganj, Mainpuri, Mathura, Mau, Meerut, Mirzapur, Mordabad, Muzaffarnagar, Nainital, Pilibhit, Partapgarh, Raebareli, Rampur, Saharanpur, Shahjahanpur, Siddarth Nagar, Sitapur, Sultanpur, Unnao, Varanasi

[1)] Districts liable to cyclonic storm but no storm surge
[2)] No cyclonic storm in Bihar
[3)] No cyclonic storm in Haryana
[4)] No cyclonic storm in Punjab
[5)] No cyclonic storm in Uttar Pradesh and Uttarakhand

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ANNEX N

[*Clause* 11.2]

STRUCTURAL DESIGN BASIS REPORT

N-1 This report shall accompany the application for Building Permit.

N-2 In case information on items (iii), (x), (xviii), (xix) and (xx) of Part 1 of SDBR not can be given at this time, it should be submitted at least one week before commencement of construction.

N-3 In case of reinforced concrete framed buildings, a certificate to the effect that the Part 3 of the SDBR will be completed and submitted at least one month before commencement of construction, shall be submitted with the application for building permit. In addition to the completed report, the following additional information shall be submitted, at the latest, one month before the commencement of construction:

a) Foundations:

1) In case raft foundation has been adopted, indicate stiffness (K) value used for analysis of the raft;

2) In case pile foundations have been used, give full particulars of the piles, type, diameter, length, capacity; and

3) In case of high water table, indicate system of countering water pressure and indicate the existing water table and that assumed to design foundations.

NOTE — The report/certificate of sub-surface investigation shall be duly considered and used in the preparation of foundation design and details.

b) Idealization for earthquake analysis:

1) In case of a composite system of shear walls and rigid frames, give distribution of base shear in the two systems on the basis of analysis and that used for design of each system; and

2) Indicate the idealization of frames and shear walls adopted in the analysis with the help of sketches.

c) Submit framing plans of each floor and in case of basements, indicate the system used to contain earth pressures.

N-4 The latest version of the Indian Standards with their amendments as indicated in the SDBR template given below, shall be referred for the preparation of the report.

N-5 The format of Part 1 of SDBR shall be as below:

PART 1 GENERAL DATA			
Sl No.	*Description*	*Information*	*Notes*
i)	Address of the building: a) Name of the building; b) Plot number; c) Subplot number, if any; d) Town Planning Scheme (if applicable): 1) Name; and 2) Number. e) Locality/Township; and f) District		
ii)	Name of owner		
iii)	Name of Building Constructor on record		

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Sl No.	*Description*	*Information*	*Notes*
iv)	Name of Architect/Engineer on record		
v)	Name of Structural engineer on record		
vi)	Use of the building		
vii)	Number of storeys above ground level (including storeys to be added later, if any)		
viii)	Number of basements below ground level		
ix)	Type of structure: a) Load bearing walls; b) R.C.C frame; c) R.C.C frame and shear walls; and d) Steel frame.		
x)	Soil data: a) Type of soil; and b) Design safe bearing capacity.		IS 1892 IS 1904
xi)	Dead loads (Unit weight adopted): a) Earth; b) Water; c) Brick masonry; d) Plain cement concrete; e) Reinforced cement concrete; f) Floor finish; g) Other fill materials; and h) Piazza floor fill and landscape.		IS 875 (Part 1)
xii)	Imposed (Live) loads: a) Piazza floor accessible to fire tender; b) Piazza floor not accessible to fire tender; c) Floor loads[1)]; and d) Roof loads[2)].		IS 875 (Part 2)
xiii)	Cyclone/Wind: a) Speed; and b) Design pressure intensity.		IS 875 (Part 3)
xiv)	Other loads, if any		Relevant parts of IS 875
xv)	Seismic zone		IS 1893 (Part 1)
xvi)	Importance factor (*I*)		IS 1893 (Part 1)
xvii)	Earthquake zone factor (Z)		IS 1893 (Part 1)
xviii)	Elastic force reduction factor (*R*)		IS 1893 (Part 1)
xix)	Fundamental natural period (approximate)		IS 1893 (Part 1)

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Sl No.	*Description*	*Information*	*Notes*
xx)	Horizontal site-specific acceleration PSA (A_{SS})		IS 1893 (Part 1)
xxi)	Expansion/Separation Joints[3)]		

1) Enclose small scale plans of each floor on A4 sheets
2) In case terrace garden is provided, indicate additional fill load and imposed (live) load
3) Indicate on a small scale plan on A4 sheet

Signature

(Registered Engineer/Structural Engineer)

N-6 The format of Part 2 of SDBR shall be as below:

PART 2 LOAD BEARING MASONRY BUILDINGS			
Sl No.	*Description*	*Information*	*Notes*
i)	Building category		IS 4326 IS 1893 (Part 1)
			Zone / Building: II III IV V
			Ordinary: B C D E
			Important: C D E E
ii)	Basement provided		
iii)	Number of floors including ground floor (all floors including stepped floors in hill slopes)		
iv)	Type of wall masonry		
v)	Type and mix of mortar		IS 4326
vi)	Size and position of openings (*see* Note 1): a) Minimum distance (b5); b) Ratio (b1+b2+b3)/l1 or (b6+b7)/l2; c) Minimum pier width between consequent opening (b4); d) Vertical distance (h3); e) Ratio of wall height to thickness; and		IS 4326

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Sl No.	*Description*	*Information*	*Notes*
	f) Ratio of wall length between cross wall to thickness;		
vii)	Horizontal seismic band:	**P** **TP** **NA**	(*see* Note 2)
	a) at plinth level;	☐ ☐ ☐	IS 4326
	b) at window sill level;	☐ ☐ ☐	IS 4326
	c) at lintel level;	☐ ☐ ☐	IS 4326
	d) at ceiling level;	☐ ☐ ☐	IS 4326
	e) at eave level of sloping roof;	☐ ☐ ☐	IS 4326
	f) at top of gable walls; and	☐ ☐ ☐	IS 4326
	g) at top of ridge walls.	☐ ☐ ☐	IS 4326
viii)	Vertical reinforcing bar:	**P** **TP** **NA**	
	a) at corners and T-junction of walls; and	☐ ☐ ☐	IS 4326
	b) at jambs of doors and window openings	☐ ☐ ☐	IS 4326
ix)	Integration of prefab roofing/ flooring elements through reinforced concrete screed	☐ ☐ ☐	IS 4326
x)	Horizontal bracings in pitched truss:		
	a) in horizontal plane at the level of ties; and	☐ ☐ ☐	
	b) in the slopes of pitched roofs	☐ ☐ ☐	

NOTES

1 Information in Sl No. (vi) of Part 2 should be given on separate A4 sheets for all walls with large number of openings.

2 P indicates "Information Provided"; TP indicates "Information to be Provided" and NA indicates "Not Applicable". Tick mark one box which is applicable.

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N-7 The format of Part 3 of SDBR shall be as below:

PART 3 REINFORCED CONCRETE FRAMED BUILDINGS			
Sl No.	*Description*	*Information*	*Notes*
i)	Type of Building: a) Regular frames; b) Regular frames with shear walls; c) Irregular frames; d) Irregular frames with shear walls; and e) Soft storey.		IS 1893 (Part 1)
ii)	Number of basements		
iii)	Number of floors including ground floor		
iv)	Horizontal floor system: a) Beams and slabs; b) Waffles; c) Ribbed Floor; d) Flat slab with drops; and e) Flat plate without drops.		
v)	Soil data: a) Type of soil; b) Recommended type of foundation: 1) Independent footings; 2) Raft; and 3) Piles. c) Recommended bearing capacity of soil; d) Recommended, type, length, diameter and load capacity of piles; e) Depth of water table; f) Chemical analysis of ground water; and g) Chemical analysis of soil.		
vi)	Foundations: a) Depth below ground level; and b) Type of independent interconnected raft piles.		
vii)	System of interconnecting foundations: a) Plinth beams; and b) Foundation beams.		
viii)	Grades of concrete used in different parts of building		

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Sl No.	*Description*	*Information*	*Notes*
ix)	Method of analysis used		
x)	Computer software used		
xi)	Torsion included		IS 1893 (Part 1)
xii)	Base shear: a) Based on approximate fundamental period; b) Based on dynamic analysis; and c) Ratio of a/b.		IS 1893 (Part 1)
xiii)	Distribution of seismic forces along the height of the building		IS 1893 (Part 1) (provide sketch)
xiv)	The column of soft ground storey specially designed		IS 1893 (Part 1)
xv)	Clear minimum cover provided in: a) Footing; b) Column; c) Beams; d) Slabs; and e) Walls.		IS 456
xvi)	Ductile detailing of RC frame: a) Type of reinforcement used; b) Minimum dimension of beams; c) Minimum dimension of columns; d) Minimum percentage of reinforcement of beams at any cross-section; e) Maximum percentage of reinforcement at any section of beam; f) Spacing of transverse reinforcement in 2-D length of beams near the ends; g) Ratio of capacity of beams in shear to capacity of beams in flexure; h) Maximum percentage of reinforcement in column; j) Confining stirrups near ends of columns and in beaL-column joints: 1) Diameter; and 2) Spacing. k) Ratio of shear capacity of columns to maximum seismic shear in the storey		IS 456 IS 13920 IS 13920 IS 456 IS 13920 IS 456 IS 13920 IS 13920 IS 456 IS 13920

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N-8 The format of Part 4 of SDBR shall be as below:

PART 4 STRUCTURAL STEEL/STEEL CONCRETE COMPOSITE BUILDINGS			
i)	Adopted method of design	a) Simple; b) Semi-rigid; and c) Rigid.	IS 800
ii)	Design based on	a) Elastic analysis; and b) Plastic analysis.	IS 800 SP 6 (6)
iii)	Floor construction	a) Composite; b) Non-composite; and c) Boarded.	
iv)	Roof construction	a) Composite; b) Non-composite; c) Metal; and d) Any other.	
v)	Horizontal force resisting system adopted	a) Frames; b) Braced frames; and c) Frames and shear walls.	NOTE — Seismic force as per IS 1893 (Part 1) would depend on the system.
vi)	Slenderness ratios maintained	Members defined in IS 800	IS 800
vii)	Member deflection limited to	a) Beams, rafters; b) Crane girders; c) Purlins; and d) Top of columns.	IS 800
viii)	Structural members	a) Encased in Concrete; and b) Not encased.	IS 800
ix)	Proposed material	a) General weldable; b) High strength; c) Cold formed; and d) Tubular.	IS 2062 (Part 1) IS 801, IS 811 IS 806
x)	Minimum metal thickness specified for corrosion	a) Hot rolled sections	IS 800

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	protection	b) Cold formed sections c) Tubes	
xi)	Structural connections	a) Rivets b) CT bolts c) HSFG bolts d) Black bolts e) Welding- Field/Shop (Specify welding type proposed) f) Composite	IS 800 IS 1929, IS 2155, IS 1148 IS 6639, IS 1367 IS 3757, IS 4000, IS 1363 IS 1367, IS 816, IS 814, IS 1395 IS 15977, IS 6419, IS 6560, IS 813, IS 9595 IS 11384
xii)	Minimum fire rating proposed, with method	a) Rating ___ hours; b) Method proposed: 1) Intumescent painting; 2) Spraying; 3) Quilting; and 4) Fire retardant boarding.	IS 1641, IS 1642, IS 1643

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LIST OF STANDARDS

The following list records those standards which are acceptable as 'good practice' and 'accepted standards' in the fulfillment of the requirements of this NBCS. The latest version of a standard shall be adopted at the time of enforcement of this NBCS. The standards listed may be used by the Authority as a guide in conformance with the requirements of the referred clauses in this NBCS.

In the following list, the number appearing in the first column within parentheses indicates the number of the reference in this Section.

	IS No.	*Title*
(1)	IS 8888 : 2020	Requirements of low income housing for urban areas — Guide (*second revision*)
(2)	IS 807 : 2006	Design, erection and testing (structural portion) of cranes and hoists — Code of practice (*second revision*)
	IS 3177 : 2020	Electric overhead travelling crane and gantry crane for all applications — Code of practice (*third revision*)
(3)	IS 17951 : 2023	Method for determination of performance relating to safety of external building envelope impacted by wind borne debris
(4)	IS 14732 : 2000/ISO 9867 : 1984	Guidelines for the evaluation of the response of occupants of fixed structures, especially buildings and off-shore structures, to low-frequency horizontal motion (0.063 to 1 Hz)
(5)	IS 18315 : 2023	Estimation of cyclonic factor (k_4) — Guidelines
(6)	IS 1893 (Part 1) : 2016	Criteria for earthquake-resistant design of structures: Part 1 General provisions and buildings (*sixth revision*)
(7)	IS 13827 : 1993	Improving earthquake resistance of earthen buildings — Guidelines
	IS 13828 : 1993	Improving earthquake resistance of Low strength masonry buildings — Guidelines
(8)	IS 13920 : 2016	Ductile design and detailing of reinforced concrete structures subjected to seismic forces — Code of practice (*first revision*)
(9)	IS 18168 : 2023	Earthquake resistant design and detailing of steel Buildings — Code of practice
(10)	IS 3414 : 1968	Code of practice for design and installation of joints in buildings
(11)	IS 875 (Part 5) : 1987	Code of practice for design loads (Other than earthquake) for building and structures: Part 5 Special loads and combinations (*second revision*)
(12)	IS 1642 : 2013	Fire safety of buildings (general) details of construction — Code of practice (*second revision*)
(13)	IS 14680 : 2024	Landslide control measures — Guidelines (*first revision*)
	IS 17162 : 2020	Preparation of landslide risk assessment maps in mountainous terrains — Guidelines
	IS 17163 : 2020	Site specific investigation and stability analysis of landslides — Guidelines
	IS 14496	Preparation of landslide hazard zonation maps in mountainous terrains — Guidelines:
	(Part 1) : 2020	Meso-zonation
	(Part 2) : 1998	Macro-zonation
	IS 18736 : 2024	Micropiles for slope stabilization for mitigation of landslides — Guidelines
	IS 14458 (Part 1) : 1998	Retaining wall for hill area — Guidelines: Part 1 Selection of type of wall
	IS 14458 (Part 2) : 1997	Retaining wall for hill area — Guidelines: Part 2 Design of

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	IS No.	*Title*
		retaining/breast walls
	IS 14458 (Part 3) : 1998	Retaining wall for hill area ― Guidelines: Part 3 Construction of dry stone walls
	IS 14458 (Part 4) : 2018	Retaining wall for hill area ― Guidelines: Part 4 Construction of banded dry stone masonry walls
	IS 14458 (Part 5) : 2018	Retaining wall for hill area ― Guidelines: Part 5 Construction of cement stone masonry walls
	IS 14458 (Part 6) : 2020	Retaining wall for hill area — Guidelines: Part 6 Construction of gabion walls
	IS 14458 (Part 7) : 2022	Retaining wall for hill area – Guidelines Part 7 Construction of peripheral reinforced gabion walls
	IS 14458 (Part 8) :2 025	Retaining wall for hill area – Guidelines Part 8 Design of reinforced cement concrete (RCC) Retaining Walls
	IS 14804 : 2000	Siting design and selection of materials for residential buildings in hilly areas ― Guidelines
	IS 14961 : 2024	Surface water management in hilly areas (including rainwater harvesting) – Guidelines (*first revision*)
(14)	IS 15498 : 2023	Improving the cyclonic resistance of low rise houses and other buildings — Guidelines (*first revision*)
	IS 15499:2004	Guidelines for survey of housing and building typology in cyclone prone areas for assessment of vulnerability of regions and post cyclone damage estimation
	IS 17164 : 2020	Design and construction of cyclone shelters ― Guidelines
	IS 17951 : 2023	Method for Determination of performance relating to safety of external building envelope impacted by wind borne debris
	IS 18153 : 2023	Improving the cyclonic resistance capacity of *Kutcha* houses — Guidelines
(15)	IS 212 : 2021	Crude coal tar for general use (*third revision*)
	IS 216 : 2006	Coal tar pitch — Specification (*second revision*)
	IS 269 : 2015	Ordinary portland cement‒ —Specification (*sixth revision*)
	IS 277 : 2018	Galvanized steel strips and sheets (Plain and Corrugated) - Specification (*seventh revision*)
	IS 303 : 2024	Plywood for general purposes — Specification (*fourth revision*)
	IS 383 : 2025	Coarse and fine aggregate for concrete — Specification (*fourth revision*) under revision Doc No: CED 2(27866)
	IS 399 : 1963	Classification of commercial timbers and their zonal distribution (*revised*)
	IS 404 (Part1) : 1993	Lead pipes — Specification: Part 1 for other than chemical purposes (*third revision*)
	IS 412 : 1975	Specification for expanded metal steel sheets for general purposes (*second revision*)
	IS 457 : 1957	Code of practice for general construction of plain and reinforced concrete for dams and other massive structures
	IS 458 : 2021	Precast concrete pipes (with and without reinforcement) (*fifth r*evision)
	IS 459 : 1992	Corrugated and semi — Corrugated asbestos cement sheets - Specification (*third revision*)
	IS 651 : 2007	Glazed stoneware pipes and fittings — Specification (*sixth revision*)
	IS 653 : 1992	Linoleum sheets and tiles — Specification (*third revision*)

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IS No.	*Title*
IS 654 : 2025	Clay roofing tiles mangalore pattern — Specification (*fifth revision*) Under Revision Doc No: CED 30 (27732)
IS 808 : 2021	Hot rolled steel beam, column, channel and angle sections — Dimensions and properties (*fourth revision*)
IS 809 : 1992	Rubber flooring materials for general purposes — Specification (*first revision*)
IS 811 : 1987	Specification for cold formed light gauge structural steel sections (*second revision*)
IS 1077 : 2025	Common burnt clay building bricks — Specification (*sixth revision*)
IS 1173 : 1978	Specification for hot rolled and slit steel tee bars (*second revision*)
IS 1322: 1993	Bitumen felts for water proofing and damp-proofing - Specification (*fourth revision*)
IS 1343 : 2012	Prestressed concrete — Code of practice (*second revision*)
IS 1478 : 2023	Clay flooring tiles specification (*third revision*)
IS 1536 : 2023	Centrifugally cast (Spun) iron pressure pipes for water, gas and sewage - Specification (*fifth revision*)
IS 1537 : 1976	Specification for vertically cast iron pressure pipes for water, gas and sewage (*first revision*)
IS 1592 : 2003	Asbestos cement pressure pipes and joints - Specification (*fourth revision*)
IS 1626	Asbestos cement building pipes and pipe fittings, gutters and gutter fittings and roofing fitting
(Part 1) : 1994	Pipe and pipe fittings (*second revision*)
(part 2) : 1994	Gutter and gutter fittings (*second revision*)
IS 1658 : 2006	Fibre hardboards — Specification (*third revision*)
IS 1725 : 2023	Stabilized soil blocks used in general building construction — Specification (*Third Revision*)
IS 1726 : 1991	Cast iron manhole covers and frames — Specification (*third revision*)
IS 1729 :2023	Sand cast iron spigot and socket pipes, fittings and accessories —Specification (*third revision*)
IS 2095	Gypsum plaster boards - Specification:
(Part 1) : 2011	Plain gypsum plaster boards (*fourth revision*)
(Part 2) : 2022	Coated laminated gypsum plaster boards (*third revision*)
(Part 3) : 2022	Reinforced gypsum plaster boards and ceiling tiles (*fourth revision*)
IS 2180 : 2025	Specification for heavy duty burnt clay building bricks (*fourth revision*) *Under Revision* CED 30(26671)
IS 2185	Concrete masonry units — Specification
(Part 1) : 2005	Hollow and solid concrete blocks (*third revision*)
(Part 3) : 1984	Autoclaved Cellular Aerated Concrete Blocks
IS 2222 : 2025	Specification for burnt clay perforated building bricks (*fourth revision*) *Under Revision* CED 30 (28174)
IS 2314	Steel sheet piling section — Specification
(Part 1) : 2023	Hot rolled sheet pile (*second revision*)
(Part 2) : 2023	Cold formed sheet pile (*second revision*)
IS 2394: 1984	Code of practice for application of lime plaster finish (*first revision*)

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IS No.	*Title*
IS 2508 : 2024	Polyethylene films and sheets — Specification (*fourth revision*)
IS 2526 : 1963	Code of practice for acoustical design of auditoriums and conference halls
IS 2541 : 1991	Preparation and use of lime concrete — Code of practice (*second revision*)
IS 2547	Specification for gypsum building plaster
(Part 1) : 1976	Excluding premixed lightweight plasters (*first revision*)
(Part 2) : 1976	Premixed lightweight plasters (*first revision*)
IS 2553 (Part 1) : 2018	Safety glass — Specification: Part 1 Architectural, building and general uses (*fourth revision*)
IS 2835 : 1987	Specification for flat transparent sheet glass (*third revision*)
IS 3087 : 2005	Particle boards of wood and other lingnocellulosic materials (medium density) for general purposes — Specification (*second revision*)
IS 3115 : 1992	Lime based blocks — Specification (*second revision*)
IS 3129 : 1985	Specification for low density particle boards
IS 3348 : 1965	Specification for fibre insulation boards
IS 3478 : 1966	Specification for high density wood particle boards
IS 4139 : 1989	Calcium silicate bricks — Specification (Second Revision)
IS 4253	Cork composition sheets — Specification
(Part 1) : 2008	Plain cork sheets (*second revision*)
(Part 2) : 2008	Cork and rubber (*second revision*)
IS 4671 : 2018	Expanded polystyrene for thermal insulation purposes (*second revision*)
IS 4860 : 1968	Specification for acid — Resistant bricks
IS 4923 : 2017	Hollow steel sections for structural use — Specification (*third revision*)
IS 6072 : 2023	Autoclaved reinforced cellular concrete wall slabs — Specifications (*first revision*)
IS 6598 : 2018	Cellular concrete for thermal insulation — Specification (*first revision*)
IS 8041 : 1990	Rapid hardening portland cement — Specification (*second revision*)
IS 9012 : 1978	Recommended practice for shotcreting
IS 12679 : 2023	By-product gypsum for construction — Specification (*second revision*)
IS 12823 : 2015	Prelaminated particle boards from wood and other Lignocellulosic material — Specification (*first revision*)
IS 12894 : 2002	Pulverized fuel ash — Lime bricks — Specification (*first revision*)
IS 14443 : 1997	Polycarbonate sheets — Specification
IS 14587 : 2023	Prelaminated medium density fibre board — Specification
IS 14635	Fluoropolymer dispersions and moulding and extrusion materials
(Part 2) : 2020	Preparation of test specimens and determination of properties (*first revision*)
IS 14753 : 1999	Polymethyl methacrylate (pmma) (acrylic) sheets
IS 14862 : 2000	Fibre cement flat sheets — Specification

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	IS No.	*Title*
	IS 15622 : 2017	Pressed ceramic tiles — Specification (*first revision*)
	IS 15658 : 2001	Concrete paving blocks — Specification (*first revision*)
	IS 16720 : 2018	Pulverized fuel ash-cement bricks — Specification
	IS 17452 : 2020	Use of alkali activated concrete for precast products — Guidelines
(16)	IS 4991 : 1968	Criteria for blast resistant design of structures for explosions above ground
(17)	IS 6992 : 1973	Criteria for safety and design of structures subject to underground blasts

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 2 SOILS AND FOUNDATIONS

BUREAU OF INDIAN STANDARDS

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National Building Code Sectional Committee, CED 46

FOREWORD

This NBCS (Part C/Sec 2) deals with the structural design aspects of foundations and mainly covers the design principles involved in different types of foundations.

This Section was published in 1970 and subsequently revised in 1983, 2005 and 2016 as a chapter of the National Building Code of India. In the first revision of 1983, design considerations in respect of shallow foundation were modified, provisions regarding pier foundation were added and provisions regarding raft foundation and pile foundation were revised and elaborated. In the second revision of 2005, design considerations in respect of shallow foundations were again modified, method for determining depth of fixity, lateral deflection and maximum moment of laterally loaded piles were modified and reference was made to ground improvement techniques.

In the third revision in 2016, the significant changes incorporated were: including the extension of the scope to cover the design of foundations on rock, modifications to definitions in line with prevailing engineering practices; addition of new terms related to pile foundations and ground improvement; inclusion of the clause on site investigation and number of modifications, such as, new methods of soil investigation; addition of depth of exploration for pile foundations; addition of new sub-clauses on vertical interval for field tests and site investigation report; inclusion of method for assessment of liquefaction potential of a site; modification of permissible differential settlements and tilt (angular distortion) for shallow foundations in soils; modification of procedures for calculation of bearing capacity, structural capacity, factor of safety, lateral load capacity, overloading, etc, in case of pile foundations; revision of design parameters with respect to adhesion factor, earth pressure coefficient, modulus of subgrade reaction, etc, for design of pile foundations to make them consistence with the outcome of modern research and construction practices; inclusion of provision for use of any established dynamic pile driving formulae, instead of recommending any specific formula, to control the pile driving at site, giving due consideration to limitations of various formulae; inclusion of a reference to spun piles, which are used in deep marshy soils where conventional pile installation beyond 50 m is difficult; elaboration of the clause on ground improvement techniques; and addition of a table on summary of soil improvement methods.

In the current revision, this publication has been renamed as National Building Construction Standards (NBCS).

As a result of technical developments in this subject, the experience gained since implementation of 2016 version of the NBCS and feedback received as well as the revision of related Indian Standards on the subject, a need was felt to revise this Section. The significant modifications made in this revision of the subsection include amongst others, the following:

a) Provisions on location and depth of investigation have been modified;

b) Provisions on various boring techniques have been updated and continuous soil samplers have been included;

c) Series of in-situ tests have been added, such as electric cone penetration test using piezocone with pore pressure measurement (CPTu), field California bearing ratio (CBR) test, cross hole seismic test (CHST), ground penetrating radar survey (GPR), spectral analysis of surface wave (SASW) and multi-channel analysis of surface wave (MASW), dilatometer test (DMT) and in-situ permeability test, etc;

d) Provisions on optical and acoustic tele-viewer methods have been added;

e) Provisions on pressuremeter test (PMT) have been detailed;

f) Detailed explanation for deep foundation has been added;

g) The provisions for the design and construction of Combined Piled Raft Foundations (CPRF) have been added. These provisions cover a composite foundation system that utilizes both raft and pile components to effectively transfer structural loads to the ground; and

h) Provision relating to geotechnical instrumentation has been introduced.

For detailed information regarding structural analysis and soil mechanics aspects of individual foundations, reference should be made to standard textbooks and available literature.

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The requirements for geotechnical engineering services, including roles and responsibilities, qualifications and experience of professionals and deliverables expected from various stakeholders, are covered in IS 19235 : 2025 'Geotechnical engineering services — Requirements'. While the standard primarily addresses pre-construction services, it also outlines interlinked construction stage services, thereby providing comprehensive guidance for geotechnical aspects throughout the project lifecycle.

The information contained in this section is mainly based on the following Indian Standards:

IS 1080 : 1985	Code of practice for design and construction of shallow foundations in soils (other than raft, ring and shell) (*second revision*)
IS 1892 : 2021	Subsurface investigation for foundations — Code of practice (*second revision*)
IS 1904 : 2021	General requirements for design and construction of foundation in soils — Code of practice (*fourth revision*)
IS 2911	Code of practice for design and construction of pile foundations
(Part 1/Sec 1) : 2010	Concrete piles, Section 1 Driven cast *in-situ* concrete piles (*second revision*)
(Part 1/Sec 2) : 2010	Concrete piles, Section 2 Bored cast *in-situ* piles (*second revision*)
(Part 1/Sec 3) 2010	Concrete piles, Section 3 Driven precast concrete piles (*second revision*)
(Part 1/Sec 4) : 2010	Concrete piles, Section 4 Precast concrete piles in prebored holes (*first revision*)
(Part 2) : 2021	Timber piles (*second revision*)
(Part 3) : 2021	Under-reamed piles (*second revision*)
IS 2950 (Part 1) : 1981	Code of practice for design and construction of raft foundations: Part 1 Design (*second revision*)
IS 19117 : 2025	Design and construction of combined piled raft foundations — Code of practice
IS 2131 : 2025	Standard penetration test of soil — Method of test

All standards, whether given herein above or cross-referred to in the main text of this Section, are subject to the revision. The parties to agreement based on this Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

As the regulation of land development and buildings is a state subject and is dealt by the states and local bodies such as urban local bodies within their jurisdiction through building regulatory documents like building regulations, building byelaws and fire regulations, this document is voluntary in nature and is non-binding. The implementation depends on adoption by concerned parties or stipulations in a contract or by appropriate adoption by the concerned authorities.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 2022 'Rules for rounding off numerical values (*second revision*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this Section.

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Important Explanatory Note for Users of the NBCS

In any Part/Section of this NBCS, where reference is made to **'good practice'** in relation to design, constructional procedures or other related information and where reference is made to **'accepted standard'** in relation to material specification, testing, or other related information, the Indian Standards listed at the end of the Part/Section shall be used as a guide to the interpretation.

At the time of publication, the editions indicated in the standards were valid. All standards are subject to revision and parties to agreements based on any Part/Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

In the list of standards given at the end of a Part/Section, the number appearing within parentheses in the first column indicates the number of the reference of the standard in the Part/Section. For example:

a) Good practices [C-2(1)] refers to the Indian Standard(s) given at serial number (1) of the list of standards given at the end of this Part/Section, that is, IS 1892 : 2021 'Code of practice for subsurface investigation for foundation (*second revision*)', IS 2131 : 2025 'Method of standard penetration test for soils (*second revision*)', IS 2132 : 1986 'Code of practice for thin walled tube sampling of soils (*second revision*)', IS 4434 : 1978 'Code of practice for in-situ vane shear test for soils (*first revision*)', IS 4968 (Part 1) : 1976 'Method for subsurface sounding for soils: Dynamic method using 50 mm cone without bentonite slurry (*first revision*)', IS 4968 (Part 2) : 1976 'Method for subsurface sounding for soils: Dynamic method using cone and bentonite slurry (*first revision*)', IS 4968 (Part 3) : 1976 'Method for subsurface sounding for soils: Static cone penetration test (*first revision*)', IS 8763 : 1978 'Guide for undisturbed sampling of sands and sandy soils' and IS 9214 : 1979 'Method for determination of modulus of subgrade reaction (k-value) of soils in the field'.

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 2 SOILS AND FOUNDATIONS

1 SCOPE

1.1 This NBCS (Part C/Section 2) covers geotechnical design (principles) of building foundations, such as shallow foundations, like, continuous strip footings, combined footings, raft foundations, deep foundations like pile foundations and other foundation systems to ensure safety and serviceability without exceeding the permissible stresses of the materials of foundations and the bearing capacity of the supporting soil/rock.

1.2 This Section also covers provisions relating to preliminary work required for construction of foundations and protection of excavation.

2 TERMINOLOGY

For the purpose of this Section, the following definitions shall apply.

2.1 General

2.1.1 *Clay* — An aggregate of microscopic and sub-microscopic particles derived from the chemical decomposition and disintegration of rock constituents. It is plastic within a moderate to wide range of water content. The particles are less than 0.002 mm in size.

2.1.2 *Clay*, *Firm* — A clay which at its natural water content can be moulded by substantial pressure with the fingers and can be excavated with a spade.

2.1.3 *Clay*, *Soft* — A clay which at its natural water content can be easily moulded with the fingers and readily excavated.

2.1.4 *Clay*, *Stiff* — A clay which at its natural water content cannot be moulded with the fingers and requires a pick or pneumatic spade for its removal.

2.1.5 *Foundation* — That part of the structure which is in direct contact with and transmits loads to the ground.

2.1.6 *Gravel* — Angular, rounded or semi-rounded particles of rock or soil of particle size between 4.75 mm and 75 mm.

2.1.7 *Peat* — A fibrous mass of organic matter in various stages of decomposition generally dark brown to black in colour and of spongy consistency.

2.1.8 *Sand* — Cohesion less aggregates of angular, sub-angular, sub-rounded, rounded, flaky or flat fragments of more or less unaltered rocks, or mineral of size between 4.75 mm and 0.075 mm IS Sieve.

2.1.9 *Sand*, *Coarse* — Sand of particle size between 2.0 mm and 4.75 mm IS Sieve.

2.1.10 *Sand*, *Fine* — Sand of particle size between 0.075 mm and 0.425 mm IS Sieve.

2.1.11 *Sand*, *Medium* — Sand of particle size between 0.425 mm and 2.0 mm IS Sieve.

2.1.12 *Silt* — Fine-grained soil or fine-grained portion of soil which exhibits a little or no plasticity and has a little or no strength when air dried. The size of particles ranges from 0.075 mm to 0.002 mm.

2.1.13 *Soft Rock* — A rocky cemented material which offers a high resistance to picking up with pick axes and sharp tools but which does not normally require blasting or chiselling for excavation.

2.1.14 *Soil*, *Black Cotton* — Inorganic clays of medium to high compressibility. They form a major soil group in India. They are predominately montmorillonitic in structure and yellowish black or blackish grey in colour. They are characterized by high shrinkage and swelling properties.

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2.1.15 *Soil*, *Coarse Grained* — Soils which include the coarse and largely siliceous and unaltered products of rock weathering. They possess no plasticity and tend to lack cohesion when in dry state. In these soils, more than half the total material by weight is larger than 75-micron IS Sieve size.

2.1.16 *Soil*, *Fine Grained* — Soils consisting of the fine and altered products of rock weathering, possessing cohesion and plasticity in their natural state, the former even when dry and both even when submerged. In these soils, more than half of the material by weight is smaller than 75-micron IS Sieve size.

2.1.17 *Soil*, *Highly Organic and Other Miscellaneous Soil Materials* — Soils consisting of large percentages of fibrous organic matter, such as peat and particles of decomposed vegetation. In addition, certain soils containing shells, concretions, cinders and other non-soil materials in sufficient quantities are also grouped in this division.

2.1.18 *Total Settlement* — The total downward movement of the foundation unit under load.

2.2 Ground Improvement

2.2.1 *Ground Improvement* — Enhancement of the in-place properties of the ground by controlled application of techniques suited to subsoil conditions.

NOTE — These techniques are used to improve the load bearing capacity and settlement potential of the loose or soft soil lying close to the surface or at depths.

2.2.2 *Injection* — Introduction of a chemical/cementitious material into a soil mass by application of pressure.

2.2.3 *Liquefaction* — It is a state primarily in saturated cohesionless soils wherein the effective shear strength is reduced to negligible value for all engineering purposes, when the pore pressure approaches the total confining pressure during earthquake shaking. In this condition, the soil tends to behave like a fluid mass.

2.2.4 *Preloading* — Application of loads to achieve improvement of soil properties prior to imposition of structural loads.

2.2.5 *Soil Densification* — A technique to densify cohesionless soils by imparting shocks or vibrations.

2.2.6 *Soil Reinforcement* — Rods, strips or geosynthetics (metallic or fabric) incorporated within soil mass to impart resistance to tensile, shear and compressive forces.

2.3 Shallow Foundation

2.3.1 *Back Fill* — Materials used or reused to fill an excavation.

2.3.2 *Bearing Capacity*, *Safe* — The maximum intensity of loading that the soil will safely carry with a factor of safety against shear failure of soil irrespective of any settlement that may occur.

2.3.3 *Bearing Capacity*, *Ultimate* — The intensity of loading at the base of a foundation which would cause shear failure of the supporting soil.

2.3.4 *Bearing Pressure*, *Allowable* (*Gross or Net*) — The intensity of loading which the foundation will carry without undergoing settlement in excess of the permissible value for the structure under consideration but not exceeding safe bearing capacity.

The net allowable bearing pressure is the gross allowable bearing pressure minus the surcharge intensity.

NOTE — The concept of ‘gross’ and ‘net’ used in defining the allowable bearing pressure could also be extended to safe bearing capacity, safe bearing pressure and ultimate bearing capacity.

2.3.5 *Factor of Safety* (*with Respect to Bearing Capacity*) — A factor by which the ultimate bearing capacity (net) shall be reduced to arrive at the value of safe bearing capacity (net).

2.3.6 *Footing* — A structure constructed in brick work, masonry or concrete under the base of a wall or column for the purpose of distributing the load over a larger area.

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2.3.7 *Foundation*, *Raft* — A substructure supporting an arrangement of columns or walls in a row or rows transmitting the loads to the soil by means of a continuous slab, with or without depressions or openings.

2.3.8 *Made-up Ground* — Refuse, excavated soil or rock deposited for the purpose of filling a depression or raising a site above the natural surface level of the ground.

2.3.9 *Offset* — The projection of the lower step from the vertical face of the upper step.

2.3.10 *Permanent Load* — Loads which remain on the structure for a period, or a number of periods, long enough to cause time dependent deformation/settlement of the soil.

2.3.11 *Shallow Foundation* — A foundation whose width is generally equal to or greater than its depth. The shearing resistance of the soil in the sides of the foundation is generally neglected.

2.3.12 *Spread* (*or Isolated or Pad*) *Foundation/Footing* — A foundation which transmits the load to the ground through one or more footings. A spread footing (or isolated or pad) is provided to support an individual column. A spread footing is circular, square or rectangular slab of uniform thickness. Sometimes, it is stepped or haunched to spread the load over a large area.

2.3.13 *Strip Foundation/Footing* — A type of shallow foundation which provides continuous and longitudinal bearing for loads carried by vertical elements, such as continuous wall foundation beams or the like. A strip footing is also provided for a row of columns which are so closely spaced that their spread footings overlap or nearly touch each other. A strip footing is also known as continuous footing.

2.4 Pile Foundation

2.4.1 *Allowable Load* — The load which may be applied to a pile after taking into account its ultimate load capacity, group effect, the allowable settlement, negative skin friction and other relevant loading conditions including reversal of loads, if any.

2.4.2 *Anchor Pile* — An anchor pile means a pile meant for resisting pull or uplift forces.

2.4.3 *Batter Pile* (*Raker Pile*) — The pile which is installed at an angle to the vertical using temporary casing or permanent liner.

2.4.4 *Bored Cast In-Situ Pile* — Piles formed by boring a hole in the ground by percussive or rotary method with the use of temporary/permanent casing or drilling mud and subsequently filling the hole with reinforced concrete.

2.4.5 *Bored Compaction Pile* — A bored cast *in-situ* pile with or without bulb(s) in which the reinforcement cage is driven by suitable method in the freshly filled concrete in the bore. If the pile is with bulb(s), it is known as under-reamed bored compaction pile.

2.4.6 *Bored Pile* — A pile formed with or without casing by excavating or boring a hole in the ground and subsequently filling it with plain or reinforced concrete.

2.4.7 *Cut-off Level* — It is the level where a pile is cut-off to support the pile caps or beams or any other structural components at that level.

2.4.8 *Driven Cast in-situ Pile* **—** The pile formed within the ground by driving a casing of uniform diameter, or a device to provide enlarged base and subsequently filling the hole with reinforced concrete. For displacing the subsoil the casing is driven with a plug or a shoe at the bottom. When the casing is left permanently in the ground, it is termed as cased pile and when the casing is taken out, it is termed as uncased pile. The steel casing tube is tamped during its extraction to ensure proper compaction of concrete.

2.4.9 *Efficiency of a Pile Group* — It is the ratio of the actual supporting value of a group of piles to the supporting value arrived at by multiplying the pile resistance of an isolated pile by their number in the group.

2.4.10 *Initial Load Test* — A test pile is tested to determine the load carrying capacity of the pile by loading either to its ultimate load or to twice the estimated safe load.

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2.4.11 *Negative Skin Friction* — Negative skin friction is the force developed through the friction between the pile and the soil in such a direction as to increase the loading on the pile, generally due to drag of a consolidating soft layer around the pile resting on a stiffer bearing stratum such that the surrounding soil settles more than the pile.

2.4.12 *Pile Spacing* **—** The spacing of piles means the centre to centre distance between adjacent piles.

2.4.13 *Precast Concrete Piles in Prebored Holes* — A pile constructed in reinforced concrete in a casting yard and subsequently lowered into prebored holes and the annular space around the pile grouted.

2.4.14 *Precast Driven Pile* — The pile constructed in concrete in a casting yard and subsequently driven into the ground when it has attained sufficient strength.

2.4.15 *Routine Test* — Test carried out on a working pile with a view to check whether pile is capable of taking the working load assigned to it without exceeding permissible settlement.

2.4.16 *Safe Load* **—** It is the load derived by applying a factor of safety on the ultimate load capacity of the pile/pile group or as determined from load test.

2.4.17 *Ultimate Load Capacity* — The maximum load which a pile can carry before failure, that is, when the founding strata fails by shear as evidenced from the load settlement curve or the pile fails as a structural member.

2.4.18 *Under-Reamed Pile* — A bored cast *in-situ* or bored compaction concrete pile with enlarged bulb(s) made by either cutting or scooping out the soil or by any other suitable process.

An under-reamed pile having more than one bulb is termed as multi under-reamed piles. The piles having two bulbs may be called double under-reamed piles.

2.4.19 *Working Load* — The load assigned to a pile as per design.

2.4.20 *Working Pile* — A pile forming part of the foundation system of a given structure.

2.5 Combined Piled-Raft Foundations

2.5.1 *Allowable Load* **—** The load which can be applied on the combined pile-raft foundation system by taking into account the load capacities of piles and raft, soil-pile-raft interaction effects and allowable displacement from serviceability criteria, negative skin friction on pile and other relevant loading types including reversal of loads, if any. In other words, it is the load at which the allowable settlement reduction ratio considering serviceability is achieved.

2.5.2 *Angular Distortion* (**θ**) **—** Vertical distance after deformation due to applied load between the points of maximum and minimum settlement divided by the horizontal distance between the two points (*see* Fig. 1).

2.5.3 *Combined Piled Raft Foundation* **—** A pile group in which the raft connecting all the pile heads positively contributes to the overall foundation behaviour. The presence of raft is recognized and its contribution in sharing the load with the pile group is taken into account. This composite foundation which combines load carrying capabilities of raft and pile foundations together, considers four interactions, namely, pile-pile interaction, pile-soil interaction, pile-raft interaction and raft-soil interaction [*see* Fig. 2(A)].

2.5.3.1 *Large CPRF* **—** CPRF in which the ratio of raft width (Br) to pile length (Lp) is greater than 1, that is, $\frac{B_r}{L_p} > 1$.

2.5.3.2 *Small CPRF* **—** CPRF in which the ratio of raft width (Br) to pile length (Lp) is less than or equal to 1, that is, $\frac{B_r}{L_p} \leq 1$.

2.5.4 *Cut-off Level* **—** The level where a pile is cut-off in order to make structural connection to the pile caps or beams or raft or any other structural component at that level.

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2.5.5 *Design Load* — The loads (compression, tension, lateral, moment or any combination thereof) considered to act on the CPRF during the life of the structure satisfying provisions mentioned in other relevant Indian Standards.

2.5.6 *Design Stress* — The stress (compressive, tensile or combination) imposed on the CPRF by the design load and calculated in accordance with engineering practice.

2.5.7 *Differential Settlement of CPRF* (*Spr*, *Diff*) — The difference between the maximum and the minimum settlement, across any section of the CPRF (*see* Fig. 1).

2.5.8 *Limiting Capacity* — The vertical load at which for a CPRF, bearing capacity failure occurs due to exceedance of the combined capacity of raft resistance and that of piles in terms of shaft resistance and end bearing resistance. The assessed value of the total resistance Rtot,k(s) of the CPRF depends on the settlements of the foundation and consists of the sum of the assessed pile resistances $\Sigma Rpile,k,j$ (s) and the assessed raft resistance, Rraft,k (s) [*see* Fig. 2(A)]. The assessed raft resistance results from the integration of the settlement dependent contact pressure, $\sigma(s, x, y)$ in the ground plan area of the raft and can be expressed as, $Rraft,k(s) = \iint \sigma(s, x, y)dxdy$. Similarly, due considerations for horizontal load capacity and moment capacity should be considered for the CPRF system.

2.5.9 *Load Eccentricity* (*eL or eB*) — Distance between the point of application of applied resultant load and the centre of gravity of the pile-raft system along length and width, respectively (*see* Fig. 1).

2.5.10 *Maximum CPRF Settlement* (*Spr*, *Max*) — The maximum value of settlement occurred at a particular point of the CPRF system (*see* Fig. 1).

2.5.11 *Pile-Pile Interaction* — The result of pile group effect, defined as the changes in the load-displacement response of a pile group and single piles due to superimposition of stress and displacement field of a single pile in a group [*see* Fig. 2(A)].

2.5.12 *Pile-Raft Interaction* — The changes in the load-displacement response of pile group when the raft is being rested to the soil surface [*see* Fig. 2(A)].

2.5.13 *Pile-Raft Coefficient or Load Sharing Ratio* (α_{CPRF}) — The ratio between the sum of the pile resistances and the value of the total resistance.

NOTE — $\alpha_{CPRF} = \frac{R_{piles}}{R_{total}}$, in which R_{piles} is the resistance provided by piles and R_{total} is the total resistance provided by the CPRF system under vertical load as shown in Fig. 2(B). Whenever, α_{CPRF} is equal to 1, all the vertical loads are taken up by the pile component which essentially makes the system to behave as a pile group foundation. Whereas, α_{CPRF} equals to zero indicates that the vertical loads are borne by raft component, hence, the foundation behaviour will be like spread raft foundation. To utilize both the foundation components effectively, the value of α_{CPRF} should be chosen carefully and the standard is not valid for $\alpha_{CPRF} \geq 0.9$.

2.5.14 *Pile-Enhanced Raft* — CPRF in which the piles are designed to mobilize their total ultimate capacity and the raft carries majority of the design load [*see* Fig. 3(A)].

2.5.15 *Raft-Pile Interaction* — The modification in the load carrying mechanism of the raft when piles are introduced beneath [*see* Fig. 2(A)]

2.5.16 *Raft-Enhanced Pile* — CPRFs in which both the piles and raft work within a pseudo-elastic range of behaviour. The pile group capacity is not fully mobilized at working load and the pile group carries majority of the design load [*see* Fig. 3(B)].

2.5.17 *Settlement Reduction Ratio* (*SRR*) — The ratio of the difference of settlement of the unpiled raft under the given loading, Sr and the settlement of the piled raft under the same loading considered in the case of the unpiled raft, S_{pr} to the settlement of the unpiled raft, that is, under any given loading conditions, SRR $= \frac{(S_r - S_{pr})}{S_r}$.

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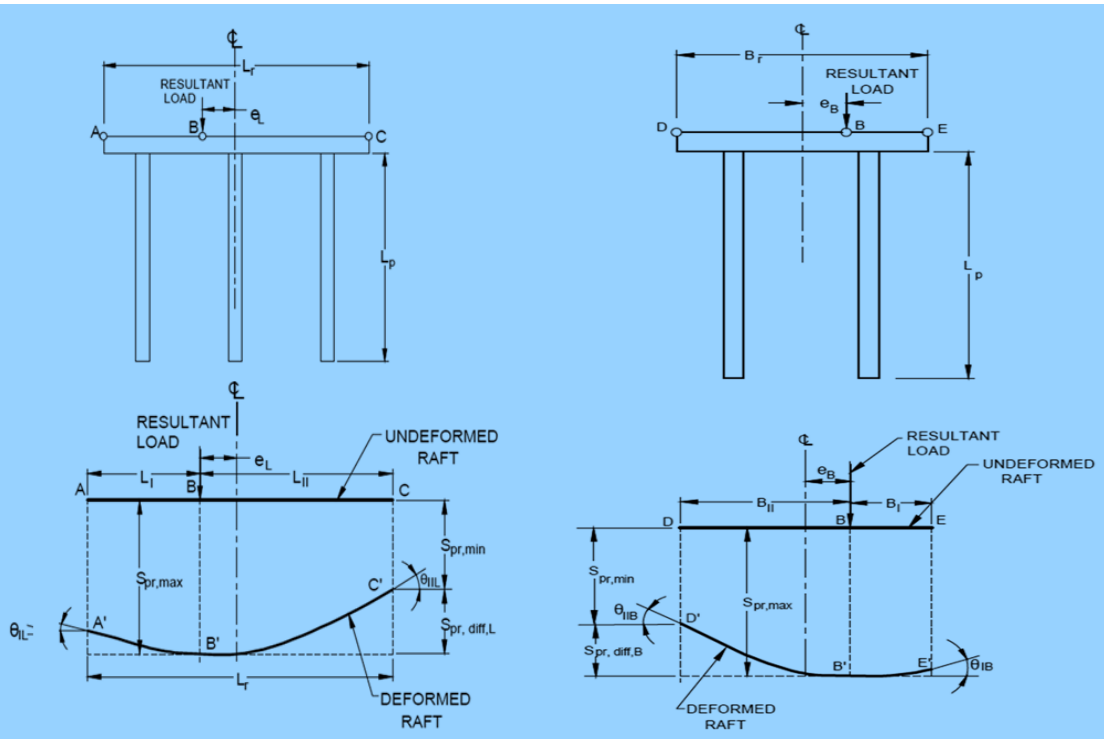

FIG. 1(A) ALONG RAFT LENGTH FIG.1(B) ALONG RAFT WIDTH

FIG.1 SCHEMATIC REPRESENTATION OF CPRF SYSTEM AND VARIOUS PARAMETERS FOR DESIGN CONSIDERATION BASED ON SERVICEABILITY LIMIT STATE

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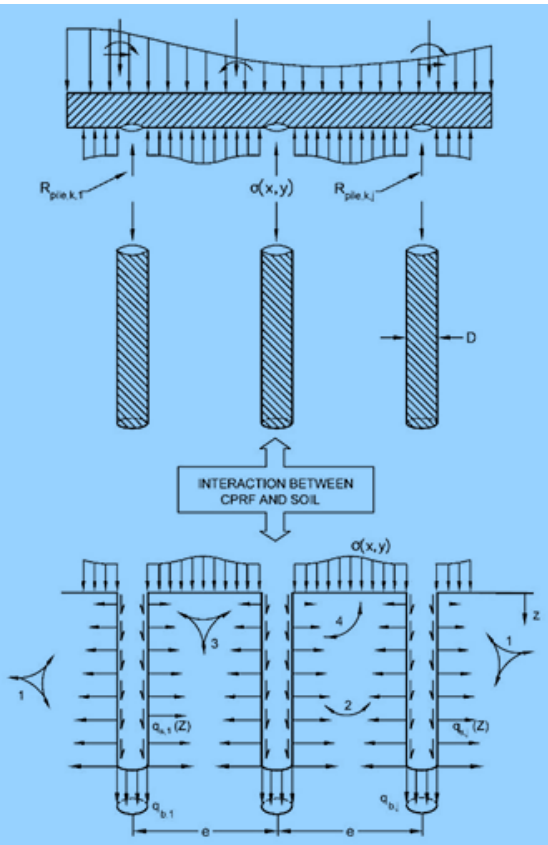
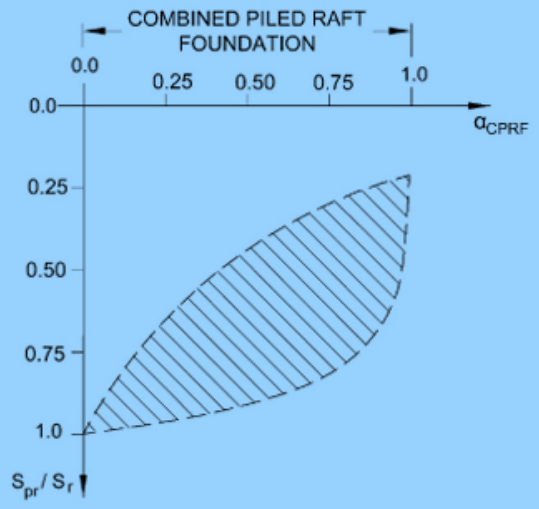


Key

1 Pile-soil interaction
2 Pile-pile interaction
3 Raft-soil interaction
4 Pile-raft interaction

FIG. 2A SOIL-STRUCTURE INTERACTION



Key

S_{pr} Settlement of combined piled raft foundation
S_r Settlement of raft foundation

FIG. 2B TYPICAL VARIATION OF α_{CPRF} WITH SETTLEMENT RATIO

FIG. 2 SOIL-STRUCTURE INTERACTION AND VARIATION OF α_{CPRF} FOR ACPRF

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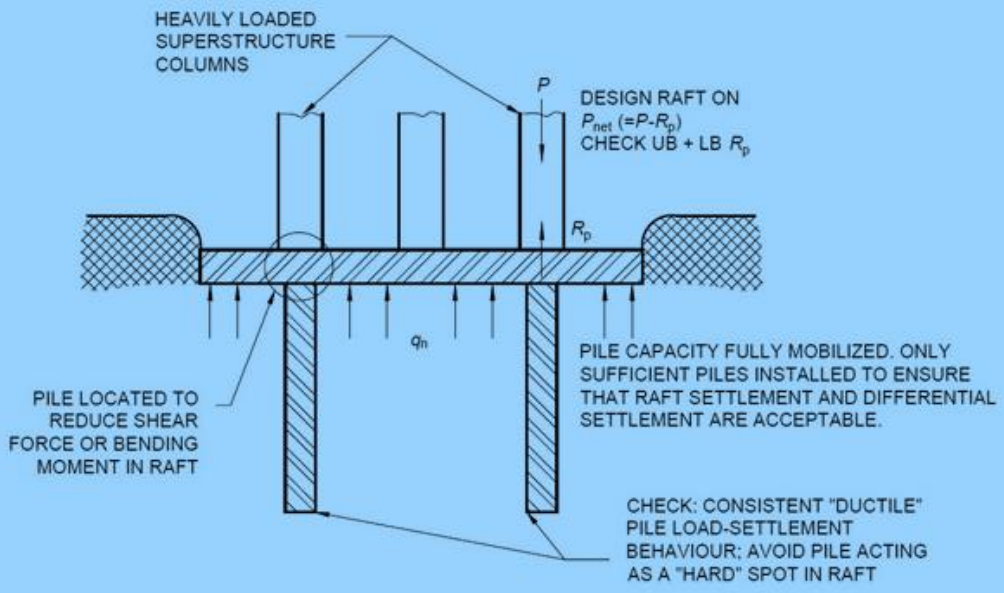


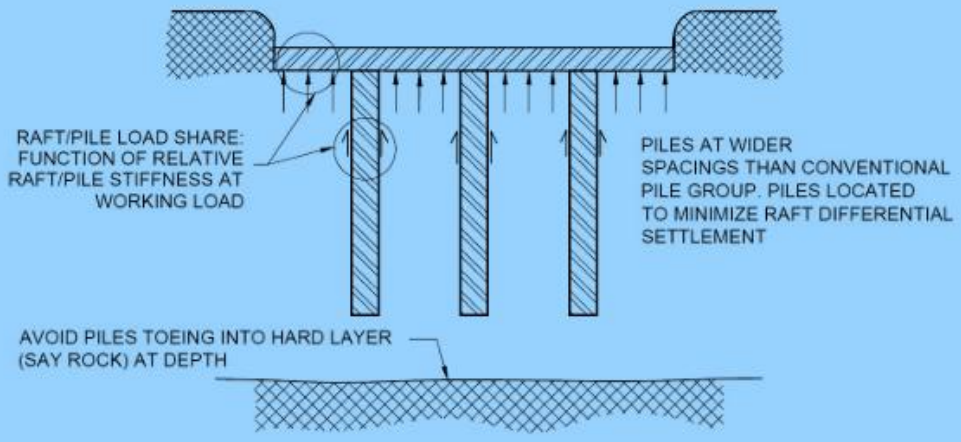
Key

q_n	Net contact stress on underside of raft
P	Column load
R_P	Pile capacity
UB	Upper bound
LB	Lower bound

Favourable ground conditions — Deep deposits of homogenous clays; typically stiff clays at raft level.

Typical design scenario — A raft which does not quite work, a small number of piles are added to resolve local non-compliances (for example, differential settlement, shear or bending moment).

FIG. 3A PILE-ENHANCED RAFT



Favourable ground conditions — Competent soils at raft level (stiff clays, dense sands) and depth; interbedded sands/clays.

Typical design scenario — A value-engineered pile group, often the raft was provided as a pile cap, but is then used to provide significant capacity.

FIG. 3B RAFT-ENHANCED PILE

FIG. 3 PILE-ENHANCED RAFT AND RAFT-ENHANCED PILE

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3 SITE INVESTIGATION

3.1 General

3.1.1 Site investigation is essential in determining the physical, chemical and engineering properties of subsoil to arrive at the required foundation system. However, in areas which have already been developed, information may be obtained regarding the existing local knowledge, records of trial pits, boreholes, etc, in the vicinity and the behaviour of the existing structures, particularly those of a similar nature to those proposed. This information may be made use of for design of foundation of lightly loaded structures of not more than two storeys and also for deciding the scope of further investigation for other structures.

3.1.2 If the existing information is not sufficient or is inconclusive, the proposed site should be explored in detail as per good practice [C-2(1)], so as to obtain a knowledge of the type, uniformity, consistency, thickness, sequence and dip of the strata, hydrology of the area and also the engineering properties. In the case of lightly loaded structures of not more than two storeys the tests required to obtain the above information are optional, mainly depending on site conditions.

3.1.3 *Geotechnical Engineering Services*

Geotechnical engineering services encompass the entire process of understanding ground conditions and providing reliable inputs for civil engineering works. They include site reconnaissance, planning and execution of field and laboratory investigations and interpretation of soil and rock behaviour for safe and economical design. The services also define the roles of the Appointing Authority (AA), Geotechnical Consultant (GC) and Geotechnical Investigation Agency (GIA), along with structured models of appointment — Model 1, where GC and GIA are appointed separately and Model 2, where GC is appointed and may additionally act as GIA. These provisions cover both pre-construction and interlinked construction stage activities, ensuring that the design and execution of works are based on sound geotechnical data and professional judgement.

A clear distinction is laid down between geotechnical factual reports (GFR), which document the results of investigations and geotechnical interpretive reports (GIR), which provide engineering analysis and design recommendations. The framework specifies minimum qualifications and experience requirements for team leaders to promote competence and accountability, while also incorporating provisions on safety, quality assurance, quality control and declarations against conflict of interest. Together, these measures strengthen transparency, consistency and technical integrity in geotechnical practice. Reference may be made to the good practice [C-2(2)] for more detail.

3.2 Stages of Subsurface Investigation

3.2.1 The stages of subsurface investigation are:

a) site reconnaissance;

b) preliminary investigation;

c) detailed investigation;

d) construction stage investigation; and

e) post-construction stage investigation.

3.2.2 *Site Reconnaissance*

3.2.2.1 Site reconnaissance would help in deciding future programme of field investigations, that is, to assess the need for preliminary and/or detailed investigations.

This would also help in determining scope of work, methods of exploration to be adopted, *in-situ* tests to be carried out and administrative arrangements required for the investigation. Where detailed information on the geotechnical conditions is not available, an inspection of site and study of topographical features are helpful in getting information about soil, rock and ground-water conditions. Site reconnaissance includes a study of local topography, excavations, ravines, quarries, escarpments; evidence of erosion or landslides, behaviour of existing structures at or near the site; water level in streams, water courses and wells; flood marks; nature of vegetation; drainage pattern, location of seeps, springs and swamps, evidence of expansive soils. Information on some of these may be obtained from topographical maps, geological maps, pedological and soil survey maps, vicinity

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maps, metrological data and digitized images.

3.2.2.2 Data regarding removal of overburden by excavation, erosion or landslides should be obtained. This gives an idea of the amount of pre-consolidation the soil strata have undergone. Similarly, data regarding recent fills is also important to study the consolidation characteristics of the fill as well as the original strata.

3.2.2.3 The type of flora affords at times some indication of the nature of the soil. The extent of swamp and superficial deposits and peats will usually be obvious. In general, such indications, while worth noting, require to be confirmed by actual exploration.

3.2.2.4 *Ground-water conditions*

The ground-water level fluctuates and will depend upon the permeability of the strata and the head causing the water to flow. The water level in streams and water courses, if any, in the neighbourhood, should be noted, but it may be misleading to take this as an indication of the depth of the water table in the ground. Wells at the site or in the vicinity give useful indications of the ground-water conditions. Flood marks of rivers may indicate former highest water levels. Tidal fluctuations may be of importance. There is also a possibility of there being several water tables at different levels, separated by impermeable strata and some of this water may be subject to artesian head. It is also important to collect water samples to check its suitability for construction purpose and any treatment of foundation required for corrosion protection.

3.2.2.5 *Enquiries regarding earlier use of the site*

In certain cases, the earlier uses of the site may have a very important bearing on proposed new works. This is particularly so in areas, where there have been underground workings, such as worked-out ballast pits, quarries, old brick fields, coal mines and mineral workings. Enquiries should be made regarding the location of shafts and workings, particularly shallow ones, where there may be danger of collapse, if heavy new structures are superimposed.

The possibility of damage to sewers, conduits and drainage systems by subsidence should also be investigated.

3.2.3 *Preliminary Investigation*

The purpose of preliminary investigation is to assess the feasibility of a project. The preliminary investigation includes determination of approximate depth, thickness, extent and composition of sub-strata, ground water table and also to obtain approximate information regarding strength and compressibility of the various strata. The preliminary investigation comprises boreholes with standard penetration test (SPT), collection of disturbed samples (DS), undisturbed samples (UDS), dynamic cone penetration test (DCPT), trial pits and laboratory testing. Geophysical investigation may also be carried out during the preliminary investigation to establish the sequence of layers, their approximate properties, depth of bedrock, etc.

3.2.4 *Detailed Investigation*

Planning of detailed investigation is generally based on the information obtained in the preliminary investigation. The objective is to find the sequence of various layers, thickness of the layers, ground water table and detailed engineering properties of each layer. Detailed investigation involves *in-situ* (field) testing and laboratory testing. The *in-situ* tests are described in **3.4**. Laboratory testing on soil/rock/water samples collected from the site is carried out to determine the index, physical, engineering and chemical properties as given in **3.5.7**. The type and number of tests shall depend upon the nature of the project, geology of the area and site conditions.

3.2.5 *Construction Stage Investigation*

During construction, there may be a requirement of additional investigation to overcome the difficulties/modifying the foundation system due to change in subsoil/ground water conditions. The type of test required shall be selected from those specified under detailed investigation.

3.2.6 *Post-construction Stage Investigation*

At times, in order to study the behaviour/strengthening of substructure/foundation, investigation may be required.

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3.3 Subsurface Exploration

The subsurface exploration includes the following:

a) Geophysical investigation;

b) Trial pits;

c) Subsurface soundings;

d) Boring in soil;

e) Core drilling in rock; and

f) Location and depth of investigation.

3.3.1 *Geophysical Investigation*

The geophysical investigation includes measurement of physical properties of the ground based on which the type of strata and depth of strata can be approximately evaluated. These tests provide a quick means of getting useful information about the stratification and underground utilities. This will enable to plan the depth/extent of the *in-situ* tests and type of test to be carried out *in-situ* and in the laboratory. The method provides only approximate parameters about the strata which need to be confirmed by conducting detailed investigation. These methods may be employed to get preliminary information on stratigraphy or complement a rationalized boring programme by correlation of stratigraphy between widely spaced boreholes. Commonly adopted geophysical tests are given in Table 1.

Table 1 Geophysical Tests

(*Clause* 3.3.1)

Sl No.	Type of Test	Property Measured
(1)	(2)	(3)
i)	Electrical resistivity test (ERT)	Electrical resistivity
ii)	Seismic refraction test (SRT)	Primary wave velocity
iii)	Cross hole seismic test (CHST) and down hole seismic test (DHST)	Shear wave velocity
iv)	Spectral analysis of surface wave (SASW) and multi-channel analysis of surface wave (MASW)	Primary/Shear wave velocity
v)	Ground penetrating radar survey (GPR survey)	Electromagnetic intensity (resistivity)
vi)	Thermal resistivity test	Thermal conductivity

3.3.2 *Trial Pit*

This method consists of excavating a pit at the site and thereby exposing the strata. This enables visual inspection of the strata, collection of disturbed/undisturbed samples and carrying out *in-situ* tests. Generally, trial pits are excavated up to 4 m depth below the ground level.

Minimum dimension of the pit shall be such that there is an adequate working space for excavation and carrying out sampling/testing. Generally, the pits have planned dimensions varying from 1 m × 1 m up to 3 m × 3 m depending on the depth required.

After the pit is excavated, the details of the strata shall be recorded with the description of each layer with thickness and ground water table, if any. After collecting the samples/in-situ testing, the pit shall be backfilled with the same excavated material. Arrangement shall be made for dewatering below ground water table. Precautions shall be taken to prevent surface water draining into the trial pit.

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3.3.3 *Subsurface Soundings*

Subsurface soundings provide quick means to get continuous resistance against penetration indicating compactness of the strata. The following sounding tests are generally carried out:

a) Dynamic cone penetration test;

b) Static cone penetration test; and

c) Static cone penetration test using piezocone with pore water pressure measurement.

The limitation of soundings is that samples cannot be collected. In addition, strata having stones, gravels, boulders, etc and stiff formations cannot be penetrated by sounding device.

3.3.4 *Boring in Soil*

Various techniques are available for boring in soil depending on the substrata and depth of investigation. Commonly used boring methods are given below and described in **3.3.4.1** to **3.3.4.4**.

a) Auger boring;

b) Shell and auger boring;

c) Rotary boring; and

d) Percussion drilling.

In general, the diameter of the borehole is about 100 mm to 150 mm. In boreholes, in-situ testing like standard penetration test along with collection of disturbed and undisturbed samples at regular depth intervals and at change of strata is carried out. Other *in-situ* tests may also be carried out in the boreholes as per requirements.

In order to prevent caving in of soil during boring, stabilizing fluid like bentonite slurry/temporary steel casing or combination of both should be used.

The bottom of the borehole shall be cleaned prior to sampling/testing at different levels.

3.3.4.1 *Auger boring*

An auger is either power or hand operated with periodic removal of cuttings. The bore profile is prepared based on physical observation of the cut material removed
from auger.

The process is slow in case of manual operation. Hand operated auger is generally used for boring up to a depth of 6 m. Casing is not used in this method. The method is convenient considering ease of mobilization. The auger is used in soil where borehole stands unsupported. Auger is penetrated into the ground for a depth of about 300 mm to 600 mm and the auger is withdrawn for removing the cuttings. The auger is then reintroduced to extend the borehole.

3.3.4.2 *Shell and auger boring*

Shell and auger boring is adopted for deep boreholes. The equipment consists of auger and a shell (bailer) for boring, connected with steel wire ropes/drilling rods which are lowered/raised with the help of tripod and mechanical winch. The casing is lowered in the borehole by rotating/pushing to the required depth.

The auger is generally used up to the ground water table level followed by shell.

The shell is raised/lowered repeatedly to cut the soil at the bottom and the cut material collected in the shell is removed. In stiff cohesive soil, it may be necessary to soak the borehole while boring above the ground water table. Where ground water is encountered, the water level in the borehole shall always be maintained at or above the water table.

The method is efficient to drill in hard strata but may disturb the soil at the bottom of the borehole. The method should be adopted with caution to minimize disturbances in the soil, prior to sampling/testing.

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3.3.4.3 *Percussion drilling*

This method should only be used for drilling boreholes in bouldery and gravelly strata where other boring methods become ineffective.

In this method, the formation is pulverized/broken by repeated blows using a bit or a chisel. Water should be added at the time of boring and the cuttings are bailed out at intervals. The bit/chisel is suspended by a cable or rods from a hoist.

As this method involves repeated dropping of a heavy chisel and can disturb the strata considerably, it is preferable that at least 0.5 m above the level of sampling/testing, the use of percussion method shall be discontinued and the borehole advanced/cleaned with shell or any other suitable method till the sampling depth has reached.

3.3.4.4 *Rotary boring*

In this method, boring is advanced by the cutting action of a rotating bit which should be kept in firm contact with the bottom of the hole. The bit is connected at the end of hollow, jointed drill rods which are rotated by a suitable chuck. Water/stabilizing fluid like bentonite slurry is pumped continuously down the hollow drill rods and the fluid returns to the surface through the annular space between the rods and the side of the hole and the casing may not be generally required. This method can be used in all type of strata.

3.3.5 *Core Drilling in Rock*

Core drilling in rock shall be carried out with rotary hydraulic rig using diamond-tipped bit (*see* Note) as per good practice [C-2(3)]. The core drilling equipment shall conform to accepted standard [C-2(4)]. Drilling shall be carried out with NX/HX size diamond-tipped drill bits or impregnated diamond bit depending on the type of rock formation. Double tube core barrel with core lifter shall be used. The diamond bit is attached to double tube core barrel with reaming shell. The bit cuts an annular groove into the formation and the inner core formed by the groove enters into the inner barrel as a core sample. The circulating fluid is injected through the annular gap between the outer barrel and inner barrel and hence the fluid does not come in contact with the core received in the barrel. This prevents damage to the core by washing out of the loose particles in the joints (if any) and improves both core recovery and rock quality designation (RQD). After advancing the core barrel, the barrel is withdrawn, the rock cores are removed by detaching the core cutter, reassembled and introduced back in the borehole for further coring. Indexing and storage of drill cores shall be in accordance with good practice [C-2(5)].

NOTE — Tungsten carbide bit may be used for rocks like, shale, siltstone and claystone.

The drilling rig shall have necessary facilities to regulate the spindle speed, bit pressure and water pressure during core drilling to get good core recovery.

The rotational speed of the bit (spindle speed), the amount of downward pressure applied on the bit (bit pressure) and water pressure shall be suitably adjusted and properly monitored so that the core is collected with least disturbance and avoid shearing of the core from its base.

Drilling run should be 0.75 m in length. This can be increased up to 1.5 m run in case core recovery is more than 50 percent in two successive 0.75 m drill runs. If the core recovery is nil, then SPT shall be performed before commencing the next drill run.

If at any time a blocking of the bit or grinding of the core is indicated, the core barrel shall be immediately withdrawn from the borehole regardless of the length of drill run completed.

3.3.6 *Location and Depth of Investigation*

3.3.6.1 The location and depth of investigation points shall depend upon the type of structure/building considering variation in substrata based on the preliminary subsurface investigation or available geotechnical data in an already developed site. The investigation points should be arranged in such a pattern that the stratification can be assessed across the site. The guidelines as specified in **3.3.6.2** and **3.3.6.3** may be adopted for deciding the location and depth of boreholes/trial pits and other *in-situ* tests.

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3.3.6.2 *Disposition of Boreholes/Trial Pits*

The guidelines for disposition of boreholes/trial pits are given in Table 2. In case significant variation in stratification is observed, additional boreholes/tests, as required may be carried out.

Table 2 Disposition of Borehole/Trial Pit

(*Clause* 3.3.6.2)

Sl No.	Type of Structure/Buildings	Location of Borehole/Trial Pit
(1)	(2)	(3)
i)	For lightly loaded residential building (such as single/double storeyed building)	At least one borehole/trial pit in the centre of the building (*see* Note)
ii)	For building(s) in a site covering an area of about 0.4 hectares	At least one borehole in each corner and one in the centre
iii)	a) For structure/building having any of the plan dimensions exceeding 50 m b) For multiple buildings in a site covering an area of more than 0.4 hectares c) For multistoried buildings and structure of height less than and equal to 50 m	Boreholes in a grid pattern with points at not more than 50 m distance, subject to a minimum of two boreholes. Boreholes should cover built-up areas.
iv)	For multistoried buildings and structure of height more than 50 m	Minimum three boreholes at each building/structure
v)	Linear structures (roads, railways, embankments, pipelines, boundary walls, tunnels, retaining walls, transmission lines, etc)	Boreholes at a spacing of 50 m to 500 m. In highly erratic substrata, spacing may be suitably reduced. In uniform substrata, the spacing may be suitably increased.
vi)	Solar power plant	One borehole for every 2 to 5 hectares subject to a minimum of 5 boreholes per site

NOTE — Boreholes are preferred over trial pits.

3.3.6.3 *Depth of Investigation*

3.3.6.3.1 The depth of investigation shall be adequate enough to provide necessary data for estimating safe bearing capacity and settlement. In general, the investigation shall be carried out to a minimum depth to which the increase in stress level due to the foundation load is lesser than 10 percent of the in-situ stress level.

The depth of investigation also depends on the risk induced by the presence of weaker formations below the influence zone to be stressed due to the proposed
construction.

3.3.6.3.2 The following guidelines may be followed for adopting the depth of investigation:

a) *Shallow foundation* (*other than raft foundations*) *in soil* —The depth of investigation shall be up to 2 to 3 times the estimated width of the largest foundation below the expected founding level;

b) *Raft foundation* (*minimum foundation width equal to 6 m for the purpose of investigation*) *in soil* — In case of raft foundation, the depth of investigation shall be up to 1.0 to 2.0 times the width of the raft foundation below the expected founding level;

c) *Pile foundation in soil* — The depth of investigation shall be sufficiently below the expected founding level of piles but this should not be less than 5 m or 5 times the diameter of piles, whichever is more

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beyond the estimated pile termination level;

d) *Embankments* — The depth of investigation shall be 1.0 to 2.0 times the height of the embankment;

e) *Well foundation in soil* — The depth of investigation in the case of well foundation shall be 1.5 to 2.0 times the width/diameter of the well from the estimated termination level of the well; and

f) *Penetration into rock* — In case rock formation is encountered prior to the termination depth of boreholes as stipulated in (a) to (e) above, the boreholes may be extended by minimum 5 m in rock strata depending upon rock characteristics and project requirements.

NOTE — The depth of investigation may be suitably arrived as per the specific requirements for any other foundation/substructure.

3.3.6.4 *Other in-situ tests*

The location and depth of investigation for other *in-situ* tests shall be as per the requirements.

3.4 *In-Situ* Testing

In-situ tests provide information about the strength and deformability characteristics of the strata. Following are the *in-situ* tests which may be conducted depending on the nature of the strata and project requirements:

a) Standard penetration test (SPT);
b) Vane shear test (VST);
c) Pressuremeter test (PMT);
d) Plate load test (PLT);
e) Cyclic plate load test (CPLT);
f) Block vibration test (BVT);
g) Dynamic cone penetration test (DCPT);
h) Static cone penetration test (SCPT);
j) Piezocone with pore pressure measurement (CPTu);
k) Flat dilatometer test (DMT);
m) Electrical resistivity test (ERT);
n) Seismic refraction test (SRT);
p) Down hole, up-hole and cross hole seismic test (CHST);
q) Spectral analysis of surface wave (SASW) and multi-channel analysis of surface wave (MASW) test;
r) Ground penetrating radar (GPR) survey;
s) Thermal resistivity test;
t) *In-situ* permeability test;
u) Modulus of subgrade reaction;
v) Field california bearing ratio test (CBR); and
w) Optical and acoustic tele-viewer methods.

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3.4.1 *Standard Penetration Test*

Standard penetration test (SPT) shall be conducted in all type of soil deposits to determine the resistance of soil at any depth in a bore hole and to recover disturbed soil sample for laboratory testing/identification purpose.

The basis of the test consists of driving a split spoon sampler by dropping a hammer of 63.50 kg ± 0.50 kg mass onto an anvil or drive head from a height of 750 mm ± 10 mm. The number of blows necessary to achieve 300 mm penetration of the sampler after the seating penetration of 150 mm is the penetration resistance. The test shall be conducted in accordance to good practice [C-2(6)]. In this test, the drive weight assemblies namely:

a) safety hammer with inner guide rod assembly and concealed anvil over which the hammer strikes;

b) do-nut hammer or safety hammer equipped with auto trip mechanism; and

c) automatic hammer system with the housing are preferred for consistent transfer of energy in each strike.

Hammer drop system used for lifting and dropping during the test shall be either mechanical auto-trip system or fully automated system in order to minimize human error. Rope and cat-head mechanism shall be used only when the other systems namely. mechanical auto trip system/fully automated system are not practicable. The depth interval between the bottom of the test and next undisturbed sampling/testing shall not be less than 500 mm.

3.4.2 *Vane Shear Test*

Field vane shear test shall be performed inside the borehole to determine the shear strength of cohesive soils, especially in soft clays which are highly susceptible to sampling disturbance, in accordance with good practice [C-2(7)].

3.4.3 *Pressuremeter Test*

The test is carried out to determine the *in-situ* stress-strain characteristics of the substrata. The test involves positioning of a radially expandable probe at the desired depth in a borehole and applying increments of pressure to inflate the probe thus applying increasing radial stresses on the substrata in the walls of the borehole and measuring the volumetric deformations at different pressure increments. Applied pressure *versus* volumetric deformation is plotted and analyzed. Further details regarding the test are given in good practice [C-2(8)].

3.4.4 *Plate Load Test*

Plate load test shall be conducted to determine the load/settlement characteristics of the substrata. The specifications for the equipment and accessories required for conducting the test, the test procedure, field observations and reporting of results shall conform to accepted standard [C-2(9)].

3.4.4.1 *Test for determination of modulus of subgrade reaction*

Modulus of subgrade reaction shall be determined in accordance with accepted standard [C-2(10)].

3.4.5 *Cyclic Plate Load Test*

This test shall be carried out to determine the dynamic soil properties required for the analysis of foundation subjected to dynamic loads. This test shall be conducted on similar lines as the ordinary plate load test. In addition, unloading shall also be done before each stage of loading. Test set up, load increment, maximum load intensity, recording of field data, etc shall be as per accepted standards [C-2(11)].

After each stage of loading, the load shall be removed in a minimum of two stages. After each stage of reduction of load, dial gauge readings and settlements shall be taken for at least one hour until the readings stabilize. There after the next loading stage shall commence. Analysis of test data shall be reported in accordance with accepted standard [C-2(12)].

3.4.6 *Block Vibration Test*

This test shall be carried out in accordance with accepted standard [C-2(12)] to determine the dynamic soil properties required for the analysis of foundation subjected to dynamic loads.

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A test pit of suitable size depending upon size of block should be made. The size of the pit may be 3 m × 6 m at the bottom and a depth preferably equal to proposed depth of foundation. The test should be conducted above the ground water table. In case of rock, the test may be performed on the surface of rock bed itself. The bottom of the pit should be levelled. Side slopes of the pit should be stable. Analysis of test data shall be reported as per accepted standard [C-2(12)].

3.4.7 *Dynamic Cone Penetration Test* (*DCPT*)

Dynamic cone penetration test is a method of sub surface sounding, wherein a continuous record of soil resistance is obtained from of the ground to the depth below by driving a cone. Dynamic cone penetration test shall be conducted either as per accepted standard [C-2(13)] or as per [C-2(14)], as required. The test shall be conducted up to the required depth; however, in order to avoid damage to the equipment, driving may be stopped when the number of blows exceeds 35 mm for 100 mm penetration when the cone is driven dry {*see* accepted standard [C-2(13)]} and 20 mm for 100 mm penetration when the cone is penetrated by circulating slurry {*see* accepted standard [C-2(14)]}. The specification for the equipment and accessories required for performing this test, procedure, field observations and reporting of results shall be in accordance with accepted standard [C-2(13)] or as per [C-2(14)], as applicable. For gravelly deposits (gravel size 100 mm to 120 mm), DCPT may be conducted as per good practice [C-2(15)].

3.4.8 *Static Cone Penetration Test* (*SCPT*)

Static cone penetration test is conducted to record the continuous *in-situ* penetration resistance of soils below the ground level. The specification for the equipment and accessories required for performing the test procedure, field observations and reporting of results shall be in accordance with accepted standard [C-2(16)].

3.4.9 *Static Cone Penetration Test using Piezocone with Pore Water Pressure Measurement* (*CPTu*)

The test is used to record the continuous *in-situ* penetration resistance, frictional resistance along with pore water pressure during penetration at the level of the base of the cone. For CPTu, all measurements shall be made by sensors contained in penetrometer.

The cone shall be pushed into the soil at a constant rate of penetration.

Test performed using this test method provide a detailed record of cone penetration which is useful for evaluation of site stratigraphy, homogeneity, voids or cavities and other discontinuities. The use of friction sleeve and porewater pressure element can provide an estimate of preliminary soil classification and correlations with engineering properties of soils.

NOTE — The test procedure, field observation and reporting of result may be referred from international standards.

3.4.10 *Flat Dilatometer Test* (*DMT*)

The test is used to determine *in-situ* strength and deformation properties of soils. The test consists of inserting a blade-shaped steel probe vertically into the soil with a thin expandable circular steel membrane mounted flush on one face and determining, at selected depths or in a semi-continuous manner, the contact pressure exerted by the soil against the membrane when the membrane is flush with the blade and subsequently the pressure exerted when the central displacement of the membrane reaches 1.10 mm. Further details regarding the test are given in good practice [C-2(8)].

NOTE — The test procedure, field observation and reporting of results may be referred from international standards.

3.4.11 *Electrical Resistivity Test*

This test is carried out to assess the substrata profiles (that is, soil and rock profiles) over large areas at a relatively lower cost and can thus supplement the information obtained from detailed site investigations.

This test involves placing four electrodes in line along the ground surface and passing an alternating current through the outer two current electrodes and measuring the resulting potential drop between the two inner potential electrodes. From the resistance, calculated as the ratio of the potential drop to the current applied, the apparent resistivity of the substrata (that is, soil or rock strata) is determined by multiplying with applicable factors which

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are dependent on the geometry and spacings of the four electrodes. The electrode spacings are varied and the apparent resistivity of the substrata is determined for various electrode spacings.

Detailed analysis/interpretation of the plot of apparent resistivity *versus* electrode spacings is carried out to assess the number and type of substrata and their thicknesses.

The equipment and accessories required for carrying out the test, the test procedure, recording of observations, presentation and analysis of results shall be in accordance with good practice [C-2(17)].

Electrical resistivity tests are also carried out to determine the earth resistivity of the substrata for use in the design of earthing systems. The equipment and accessories required for carrying out the test for earth resistivity determination, the test procedure, recording of observations, presentation and analysis of results shall be in accordance with good practice [C-2(18)].

3.4.12 *Seismic Refraction Test*

This test is carried out to assess the substrata profiles (that is, soil and rock profiles) over large areas to supplement the information obtained from detailed site investigations.

This test involves the generation of seismic waves in the ground using a suitable seismic source and recording these seismic waves through geophones (that is, instruments which convert ground vibrations to electrical signals) located in a line along the ground surface at regular distance intervals. From the recorded waveforms, the arrival times of the seismic waves at different geophone locations are determined and a plot is then drawn of the geophone distances *versus* the corresponding arrival times. The refraction/reflection of the seismic waves at the interfaces of the various substrata leads to variations in the arrival times of the seismic waves at larger distances from the seismic source. The slopes of the lines connecting the data points on such a plot give the seismic wave velocities of the various substrata. The thicknesses of the various substrata so identified are determined by detailed analysis/interpretation of the plot which are based on the principles of refraction/reflection of the seismic waves.

The equipment and accessories required for carrying out the test, the test procedure, recording of observations, presentation and analysis of results shall be in accordance with good practice [C-2(19)].

3.4.13 *Downhole*, *Uphole and Crosshole Seismic Tests*

These tests are carried out to determine the seismic wave velocity profiles with depth and therefrom the profiles of dynamic modulus of rigidity (that is, shear modulus) and dynamic Young's modulus (that is, elastic modulus) with depth.

In the downhole seismic test, seismic waves are generated in the ground by a suitable seismic source at the ground surface near a borehole and the seismic waves so generated are recorded using geophones located at different depths within the borehole. On the other hand, in the uphole seismic test, seismic waves are generated at different depths within a borehole by a suitable seismic source and the seismic waves so generated are recorded using a geophone located at the ground surface near the borehole.

In the crosshole seismic test, seismic waves are generated at different depths within a borehole termed the source borehole, using a suitable seismic source and the seismic waves so generated are recorded using geophones installed at the corresponding depths in one or two other boreholes termed the receiver boreholes, located in a straight line with the source borehole at predefined borehole spacings.

In these tests, from the recorded waveforms, the seismic wave velocities are determined and therefrom the dynamic shear and elastic moduli are determined.

The equipment and accessories required for carrying out the downhole or uphole seismic test, the test procedure, recording of observations, presentation and analysis of results shall be in accordance with good practice [C-2(20)].

The equipment and accessories required for carrying out the crosshole seismic test, the test procedure, recording of observations, presentation and analysis of results shall be in accordance with good practice [C-2(21)].

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3.4.14 *Spectral Analysis of Surface Wave* (*SASW*) *and Multi-Channel Analysis of Surface Wave* (*MASW*) *Test*

The tests are used for evaluation of substrata properties based on the generation of wave propagation phenomena like reflection, refraction and dispersion. Compression and surface wave velocities are measured. Further details regarding the test are given in the good practice [C-2(8)].

NOTE — The test procedure, field observation and reporting of results may be referred from international standards.

P-wave and S-wave velocities as obtained from above tests can also be used to get seismic tomography.

3.4.15 *Ground Penetrating Radar Survey*

Ground penetrating radar (GPR) is a tool to assess the presence of soil/rock strata and to identify presence of buried objects such as pipeline, conduit, concrete, etc. This works on the principle of reflection of the electromagnetic waves at the interface of two different materials. The depth of penetration below the ground surface can be increased by reducing the frequency.

3.4.16 *Thermal Resistivity Test*

Thermal resistivity test is conducted to determine the thermal conductivity of soil and soft rock in field using thermal needle probe procedure. Further details regarding the test are given in good practice [C-2(8)].

NOTE — The test procedure, field observation and reporting of results may be referred from international standards.

3.4.17 *In-situ Permeability Test (in Soil)*

In-situ permeability test is conducted to determine the water percolation capacity of the sub strata. This test is performed inside a borehole/trial pit at specified depths. The type of test shall be either pump-in or pump-out test depending on the subsoil and ground water conditions. Pump-in test is suitable for sub-strata with or without ground water; wherein pump-out test is conducted for sub-strata with ground water.

3.4.17.1 *Pump-in test*

Pump-in test is conducted in a borehole/trial pit by allowing water to percolate into the soil.

a) *Constant head method* (*in borehole*) — This test is preferred for soils with high permeability. The specification for the equipment and the procedure of testing shall be in accordance with good practice [C-2(22)];

b) *Falling head method* (*in borehole*) — This method is preferred for soils with low permeability. The specification for the equipment and the procedure of testing shall be in accordance with good practice [C-2(22)]; and

c) *Percolation test* (*in trial pit*) — Percolation test in trial pit shall be conducted as per good practice [C-2(23)].

3.4.17.2 *Pump-out test*

Pump-out test is conducted in a borehole by pumping out of water from sub-soil. The specification for the equipment and procedure for testing shall be in accordance with good practice [C-2(22)].

3.4.18 *In-situ Permeability Test* (*in Rock*)

In-situ permeability test is conducted inside the drilled hole by pumping in water under pressure to determine the percolation capacity of the rock stratum. The test may be conducted by single packer method or by double packer method. The specification for the equipment and the procedure of testing shall be in accordance with accepted standard [C-2(24)].

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3.4.18.1 *Single packer method*

This method shall be adopted when the bottom elevation of the test section is the same as the bottom of the drill hole and where it is considered necessary to know the permeability value during drilling itself. This test shall be useful where the full length of the hole cannot stand encased or ungrouted. The packer shall be fixed at the top level of the test section, such that only the test section lies below the packer. Water shall then be pumped through a pipe into the test section under a required pressure and maintaining it till a constant quantity of water intake is observed. The amount of water percolating through the hole shall be recorded at every 5 min intervals. The test shall be repeated by increasing the pressure at regular intervals up to a pressure limit as specified in accordance to accepted standard [C-2(24)]. The details and observations during the test shall be suitably recorded in a proforma recommended in the accepted standard [C-2(24)].

3.4.18.2 *Double packer method*

This method shall be used when the permeability of an isolated section inside a drill hole is to be determined. Packers shall be fixed both at the top and bottom of the test section such that their spacing is exactly equal to the length of the test section.

3.4.19 *Field California Bearing Ratio* (*CBR*) *Test*

This test is carried out to determine the *in-situ* CBR value of the compacted subgrade on which the road pavement will be constructed so as to confirm that the *in-situ* CBR value is equal to or greater than the design CBR value for the road pavement. The test involves applying static loading to cause penetration of a cylindrical plunger of standard diameter into the compacted subgrade material and recording the applied static load to cause the required increments of penetration till the required maximum penetration of the plunger is achieved. Based on the plot of the recorded applied load and corresponding penetrations of the plunger, the CBR value is determined as the ratio of the applied test load to the predefined standard load for different standard penetrations of the plunger. The equipment and accessories required for carrying out the test, the test procedure, recording of observations and presentation of results shall be in accordance with accepted standard [C-2(25)].

3.4.20 *Optical and Acoustic Tele-Viewer Methods*

The optical and acoustic tele-viewers provide a continuous, 360°view of the borehole wall that allows rock mass discontinuities to be identified and characterized. Both devices can be oriented within the borehole so that the absolute orientation of features like bedding planes can be measured. An optical tele-viewer (OTV) uses a camera to record high-resolution images of the borehole wall and includes lights for illumination. An OTV is best suited for dry boreholes or boreholes filled with clear water.

Any conditions that produce cloudy or murky water or coatings on the borehole wall, limit the usefulness of the OTV. If good images are obtained, it is possible to identify locations and orientations of joints, bedding planes, foliations, faults, shears and other naturally occurring rock mass discontinuities.

The acoustic tele-viewer (ATV) uses ultrasound pulses from a rotating sensor in an open, fluid-filled borehole to record the amplitude and travel time of the signals reflected at the (high-impedance) interface between fluids and the borehole wall. Because an ATV uses ultrasound rather than visible light, the borehole fluid is not required to be clear. Rock mass discontinuities in the wall of the borehole will change the amplitude of the reflected acoustic wave compared to the surrounding material. The method does not work well in soil because of the lack of a high-impedance boundary between the fluid and soil. ATV surveys are used to provide information regarding locations and orientations of joints, bedding planes, faults, shears and other naturally occurring rock mass discontinuities.

3.5 Sampling and Testing

3.5.1 In order to identify the type of soil/rock and to determine the physical, chemical and engineering properties of sub strata, samples of soil, rock and water, as applicable are collected during the investigation. The samples are of the following two types:

a) *Disturbed sample* — The sample in which natural structure of the material is disturbed; and

b) *Undisturbed sample* — The sample in which the natural structure of the material is not disturbed.

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3.5.2 The type of sample, quantity of sample and number of samples to be collected depends on the purpose of investigation and the nature of the strata condition. The interval at which the samples are collected depends on the type of strata variation of the profiles within the strata. Generally, undisturbed samples (UDS) are collected at depth interval of 3.0 m or change of strata, whichever is lower. The depth between UDS and SPT are generally staggered at 1.5 m so that at every 1.5 m depth either UDS is collected or SPT is conducted. In case of variations of profiles within a stratum, the samples shall be collected at a frequency of at least every 1.5 m to 2 m depth interval in a borehole. Wherever undisturbed sampling is not possible, such as in cohensionless soil or some utility service is encountered during the investigation, the same may be replaced by standard penetration test. In cohesive soils, when N value is above 32, undisturbed samples may be replaced by standard penetration test.

3.5.3 The following are the methods that are usually employed for sampling:

Nature of stratum	*Type of sample*	*Method of sampling*
Soil	Disturbed	Shovel, pick axe, etc
		Auger and shell boring
		Split spoon sampler
	Undisturbed	Block sample
		Tube sampler
		Piston sampler
		Core cutter sampler
		Continuous soil sampler
Rock	Disturbed	Blasting
		Chiseling
	Undisturbed	Coring

3.5.4 In cohesive soils, it is possible to collect undisturbed samples for examination and testing. In cohesionless soils below water table, undisturbed sampling is difficult with the samplers other than piston sampler.

3.5.5 Disturbed soil sample and undisturbed soil sample are to be obtained as per 3.5.5.1 and 3.5.5.2, respectively.

3.5.5.1 *Disturbed soil sample*

Disturbed sample of soil is obtained during the course of excavation of trial pit, boring and after conducting standard penetration test. This sample can be used for testing of physical and chemical properties. The disturbed sample of clay is also useful for determining shear strength of remoulded sample.

3.5.5.2 *Undisturbed soil sample*

Undisturbed sample is obtained from pits and boreholes in such a manner that moisture content and structure do not get altered. This may be attained by careful protection, packing and by the use of a correctly designed sampler. Various methods of undisturbed sampling are as given in good practice [C-2(8)].

3.5.6 *Protection, Handling and Labelling of Samples*

Care should be taken in protecting, handling and subsequent transport of samples and in their full labelling, so that samples can be received in a fit state for examination and testing and can be correctly recognized as coming from a specified trial pit or boring.

3.5.7 *Examination and Testing of Samples*

3.5.7.1 The following tests shall be carried out in accordance with accepted standard [C-2(26)]:

a) *Test on disturbed/undisturbed soil samples*
   1) Visual and engineering classification;
   2) Sieve analysis and hydrometer analysis;

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3) Atterberg limits (liquid limit, plastic limit and shrinkage limit);
4) Specific gravity;
5) Free swell index;
6) Light/Heavy compaction test (standard/modified proctor compaction test);
7) California bearing ratio;
8) Relative density (for sand); and
9) Laboratory permeability test (constant head/falling head).

b) *Test on undisturbed soil samples*
   1) Bulk density and moisture content;
   2) Unconfined compression test (on remoulded or undisturbed sample);
   3) Swelling pressure (on undisturbed and remoulded sample);
   4) Direct shear test (drained/undrained) (on remoulded or undisturbed sample);
   5) Triaxial compression tests (on undisturbed or remoulded sample):
      i) Unconsolidated undrained;
      ii) Consolidated undrained test with/without the measurement of pore water pressure; and
      iii) Consolidated drained;
   6) Consolidation test; and
   7) Vane shear test.

c) *Test on rock samples*
   1) Visual classification;
   2) Water absorption;
   3) Porosity;
   4) Density;
   5) Specific gravity;
   6) Hardness;
   7) Slake durability;
   8) Uniaxial compression test (both saturated and unsaturated);
   9) Point load strength index;
   10) Deformability test (both saturated and unsaturated);
   11) Tensile strength (Brazilian); and
   12) Triaxial compression test.

d) *Chemical analysis on subsoil and ground water* — Tests are conducted to determine the concentration of following chemicals:
   1) *p*H value;
   2) Sulphates (SO_3 and/or SO_4);
   3) Chlorides;
   4) Organic matter; and
   5) Any other chemicals harmful to the foundation material.

In case of ground water, colour, odour, turbidity and specific conductivity should also be determined.

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3.6 Soil Investigation Report

3.6.1 *General*

On completion of the *in-situ* and laboratory tests, subsurface investigation report shall be prepared which should include, geological information, procedure adopted for investigation, field observations, laboratory test data, analysis, interpretation, recommendation, etc.

3.6.2 *Information to be Provided*

3.6.2.1 The report shall include but not be limited to the following:

a) Plan showing location and depth of *in-situ* tests;

b) Geological information of the area;

c) General topography of the site;

d) Past usage and historical data of the site, if available;

e) Borelog/trial pit log (*see* good practice [C-2(8)] for format of borelog);

f) *In-situ* test results;

g) Plot of standard penetration test (N values both uncorrected and corrected) with depth for identified areas;

h) Laboratory test results (*see* good practice [C-2(8)] for format for laboratory test results);

j) Summary of strata profile; and

k) In case of pressuremeter tests, the following should be furnished:

   1) Calibration record including, description of membrane and sheath on probes, dimension of thick walled cylinder, length of flexible tubing, calibration curves/chart, temperature, etc;

   2) Drilling record including, borehole number, method of making borehole, log of soil type and condition, depth of water in borehole, weather and temperature; and

   3) *Test record* — Type of test, date and time, depth of centre point of probe, volume readings at 30 s and 60 s elapsed time and corresponding pressure readings, notes on any deviation from standard test procedure.

3.6.2.2 Reporting of *in-situ* and laboratory test results should be in accordance with relevant Indian Standards.

3.6.3 *Analysis and Interpretation of Results*

The following shall be included in the report:

a) Analysis of test results;

b) Discussion and interpretation of results analyzed including basis of adopting design parameters for the substrata;

c) For shallow foundations:

   1) Net allowable bearing pressure considering shear as well as settlement criteria for isolated footing, continuous strip footing and raft foundation; and

   2) Modulus of subgrade reaction and modulus of elasticity of substrata.

d) For pile foundations: Safe load carrying capacity;

e) Dynamic soil properties such as shear wave velocity, dynamic shear modulus, Poisson's ratio, damping ratio, as applicable;

f) Suitability of the soil for backfilling around foundations;

g) Susceptibility of sub-soil strata to liquefaction, if any, in the event of dynamic loading; and

h) Remedial measures against the presence of harmful chemicals in the sub-soil/ground water.

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4 CLASSIFICATION AND IDENTIFICATION OF SOILS

The classification and identification of soils for engineering purposes shall be in accordance with accepted standard [C-2(27)].

5 MATERIALS

5.1 Cement, coarse aggregate, fine aggregate, lime, *Surkhi*, steel, timber and other materials go into the construction of foundations shall conform to the requirements of Part B 'Building Materials' of this NBCS.

5.2 Protection Against Deterioration of Materials

Where a foundation is to be in contact with soil, water or air, that is, in a condition conducive to the deterioration of the materials of the foundation, protective measures shall be taken to minimize the deterioration of the materials.

5.2.1 *Concrete*

The concrete used for construction shall be in accordance with Part C 'Structural Design/ Section 5 Structural Concrete' of this NBCS.

5.2.2 *Timber*

Where timber is exposed to soil, it shall be treated in accordance with good practice [C-2(28)].

6 GENERAL REQUIREMENTS FOR FOUNDATIONS/SUB-STRUCTURES FOR BUILDINGS

6.1 Types of Foundations

Types of foundations for buildings covered in this Section are:

a) *Shallow foundations* (*see* 7)
   1) Pad or spread and strip foundations (*see* 7.3);
   2) Raft foundations (*see* 7.4);
   3) Ring foundations (*see* 7.5); and
   4) Shell foundations (*see* 7.6).

b) *Deep foundations*
   1) Pile Foundations
      i) Driven cast *in-situ* concrete piles (*see* 8.2);
      ii) Bored cast *in-situ* concrete piles (*see* 8.2);
      iii) Driven precast concrete piles (*see* 8.3);
      iv) Precast concrete piles in prebored holes (*see* 8.4);
      v) Under-reamed concrete piles (*see* 8.5);
      vi) Timber piles (*see* 8.6); and
      vii) Spun piles.

      NOTE — Spun piles are used in deep marshy soils as conventional pile installation beyond 50 m in such soils is very difficult.

c) *Other foundations/Sub-structures/Foundations for special structure*
   1) Pier foundations (*see* 10.1);
   2) Combined piled-raft foundation (*see* 9);
   3) Diaphragm walls (*see* 10.2); and
   4) Machine foundations (*see* 10.2).

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6.2 Depth of Foundations

6.2.1 The depth to which foundations should be carried depends upon the following principal factors:

a) Getting adequate allowable bearing capacity;

b) In the case of clayey soils, penetration below the zone where shrinkage and swelling due to seasonal weather changes and due to trees and shrubs are likely to cause appreciable movements;

c) In fine sands and silts, penetration below the depth of frost penetration;

d) Maximum depth of scour, wherever relevant, should also be considered and the foundation should be located sufficiently below this depth; and

e) Other factors such as ground movements and heat transmitted from the building to the supporting ground may be important.

6.2.2 All foundations shall extend to a depth of at least 500 mm below natural ground level. On rock or such other weather resisting natural ground, removal of the top soil may be all that is required. In such cases, the surface shall be cleaned and, if necessary, stepped or otherwise prepared so as to provide a suitable bearing and thus prevent slipping or other unwanted movements.

6.2.3 Where there is excavation, ditch, pond, water course, filled-up ground or similar condition of the subsoil on which the structure is to be erected, the foundation of such structure shall either be carried down to a depth beyond the detrimental influence of such conditions, or retaining walls shall be provided for the purpose of shielding from the effects of such conditions.

6.2.4 A foundation in any type of soil shall be below the zone significantly weakened by root holes or cavities produced by burrowing animals or works. The depth shall also be enough to prevent the rainwater scouring below the footings.

6.2.5 Clayey soils, like black cotton soils, are seasonally affected by drying, shrinkage and cracking in dry and hot weather and by swelling in the following wet weather to a depth which will vary according to the nature of the clay and the climatic condition of the region. It is necessary in these soils, either to place the foundation bearing at such a depth where the effects of seasonal changes are not important or to make the foundation capable of eliminating the undesirable effects due to relative movement by providing flexible type of construction or rigid foundations. Adequate load counteracting against swelling pressures also provide satisfactory foundations.

6.3 Foundation at Different Levels

6.3.1 Where footings are adjacent to sloping ground or where the bottoms of the footings of a structure are at different levels or at levels different from those of the footings of adjoining structures, the depth of the footings shall be such that the difference in footing elevations shall be subject to the following limitations:

a) When the ground surface slopes downward adjacent to a footing, the sloping surface shall not intersect a frustum of bearing material under the footing having sides which make an angle of 30° with the horizontal for soil and horizontal distance from the lower edge of the footing to the sloping surface shall be at least 600 mm for rock and 900 mm for soil (*see* Fig. 4);

b) In the case of footings in granular soil, a line drawn between the lower adjacent edges of adjacent footings shall not have a steeper slope than one vertical to two horizontal (*see* Fig. 5); and

c) In case of footing in clayey soils a line drawn between the lower adjacent edge of the upper footing and the upper adjacent edge of lower footing shall not have a steeper slope than one vertical to two horizontal (*see* Fig. 6).

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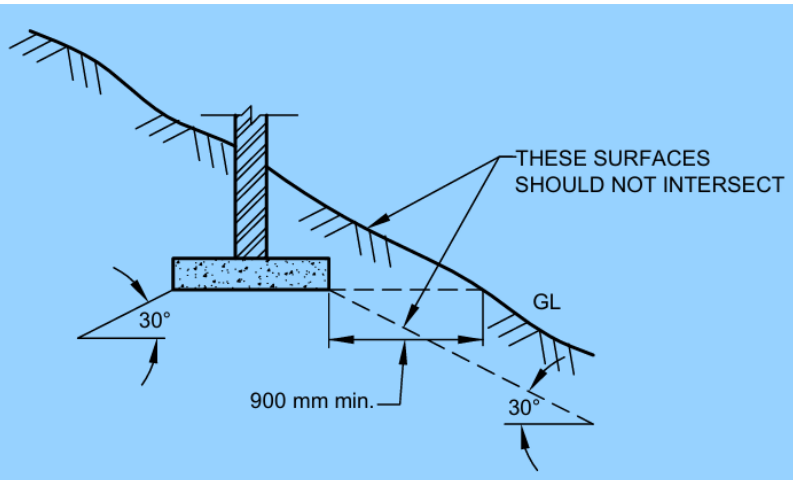

FIG. 4 FOOTING IN SLOPING GROUND

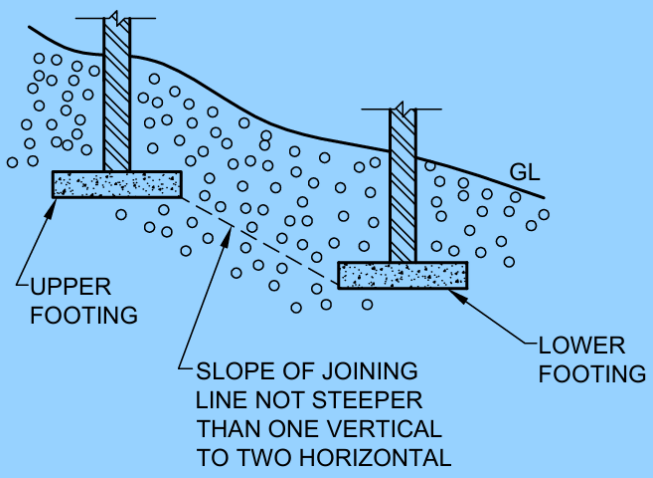

FIG.5 FOOTING IN GRANULAR SOIL

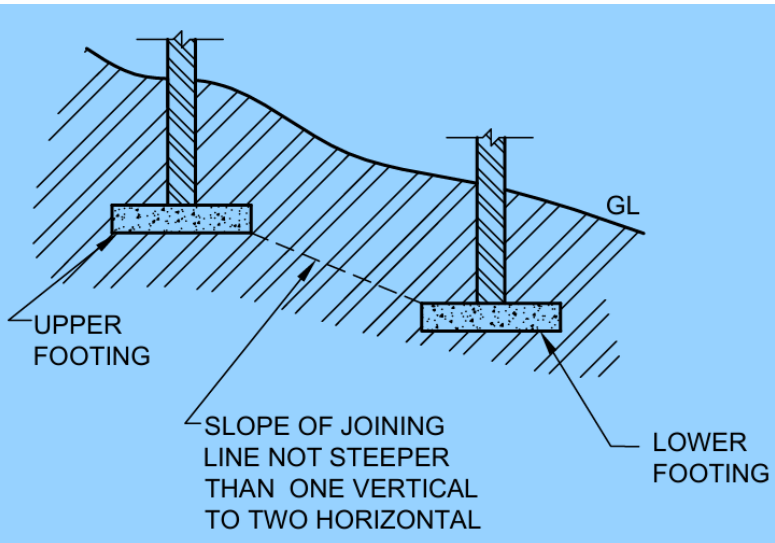

FIG.6 FOOTING IN CLAYEY SOIL

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6.3.2 The requirement given in 6.3.1 shall not apply in the condition where adequate provision is made for the lateral support (such as with retaining walls) of the material supporting the higher footing.

6.4 Effect of Seasonal Weather Changes

During periods of hot, dry weather a deficiency of water develops near the ground surface and in clay soils, that is associated with a decrease of volume or ground shrinkage and the development of cracks. The shrinkage of clay will be increased by drying effect produced by fast growing and water seeking trees. The range of influence depends on size and number of trees and it increase during dry periods. In general, it is desirable that there shall be a distance of at least 6 m between such trees. Boiler installations, furnaces, kilns, underground cables and refrigeration installations and other artificial sources of heat may also cause increased volume changes of clay by drying out the ground beneath them, the drying out can be to a substantial depth. Special precautions either in the form of insulation or otherwise should be taken. In periods of wet weather, clay soils swell and the cracks lend to close, the water deficiency developed in the previous dry periods may be partially replenished and a subsurface zone or zones deficient in water may persist for many years. Leakage from water mains and underground sewers may also result in large volume changes. Therefore, special care shall be taken to prevent such leakages.

6.5 Effect of Mass Movements of Ground in Unstable Areas

In certain areas mass movements of the ground are liable to occur from causes independent of the loads applied by the foundations of structures. These include mining subsidence, landslides on unstable slopes and creep on clay slopes.

6.5.1 *Mining Subsidence*

In mining areas, subsidence of the ground beneath a building or any other structure is liable to occur. The magnitude of the movement and its distribution over the area are likely to be uncertain and attention shall, therefore, be directed to make the foundations and structures sufficiently rigid and strong to withstand the probable worst loading condition and probable ground movements. Where future subsidence is likely, care should be taken to design the superstructure and foundation sufficiently strong to cater for probable ground movements. Long continuous buildings should be avoided in such areas and large building in such area should be divided into independent sections of suitable size, each with its own foundations. Expert advice from appropriate mining authority should be sought.

NOTE — For prediction of subsidence in coal mines, guidelines as given in the accepted standard [C-2(29)] may be referred.

6.5.2 *Landslide Prone Areas*

The construction of structures on slopes which are suspected of being unstable and are subject to landslip shall be avoided.

On sloping ground on clay soils, there is always a tendency for the upper layers of soil to move downhill, depending on type of soil, the angle of slope, climatic conditions, etc. In some cases, the uneven surface of the slope on a virgin ground will indicate that the area is subject to small land slips and, therefore, if used for foundation, will obviously necessitate special design consideration as specified in accepted standard [C-2(30)].

Where there may be creep of the surface layer of the soil, protection against creep may be obtained by following special design considerations.

On sloping sites, spread foundations shall be on a horizontal bearing and stepped. At all changes of levels, they shall be lapped at the steps for a distance at least equal to the thickness of the foundation or twice the height of the step, whichever is greater. The steps shall not be of greater height than the thickness of the foundation, unless special precautions are taken.

Cuttings, excavations or sloping ground near and below foundation level may increase the possibility of shear failure of the soil. The foundation shall be well beyond the zone of such shear failure.

If the probable failure surface intersects a retaining wall or other revetment, the latter shall be made strong enough to resist any unbalanced thrust. In case of doubt as to the suitability of the natural slopes or cuttings, the structure shall be kept well away from the top of the slopes, or the slopes shall be stabilized.

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Cuttings and excavations adjoining foundations reduce stability and increase the likelihood of differential settlement. Their effect should be investigated not only when they exist but also when there is possibility that they are made subsequently.

Where a structure is to be placed on sloping ground, additional complications are introduced. The ground itself, particularly if of clay, may be subject to creep or other forms of instability, which may be enhanced if the strata dip in the same direction as the ground surface. If the slope of the ground is large, the overall stability of the slope and substructure may be affected. These aspects should be carefully investigated.

6.6 Precautions for Foundations on Inclined Strata

In the case of inclined strata, if they dip towards a cutting of basement, it may be necessary to carry foundation below the possible slip planes, land drainage also requires special consideration, particularly on the uphill side of a structure to divert the natural flow of water away from the foundations.

6.7 Strata of Varying Thickness

Strata of varying thickness, even at appreciable depth, may increase differential settlement. Where necessary, calculations should be made of the estimated settlement from different thickness of strata and the structure should be designed accordingly. When there is large change of thickness of soft strata, the stability of foundation may be affected. Site investigations should, therefore, ensure detection of significant variations in strata thickness.

6.8 Layers of Softer Material

Some soils and rocks have thin layers of softer material between layers of harder material, which may not be detected unless thorough investigation is carried out. The softer layers may undergo marked changes in properties if the loading on them is increased or decreased by the proposed construction or affected by related changes in ground water conditions. These should be taken into account.

6.9 Spacing Between Existing and New Foundation

The deeper the new foundation and the nearer to the existing it is located, the greater the damage is likely to be. Preferably, the minimum horizontal spacing between existing and new footings shall be equal to the width of the wider clear footing. While the adoption of such provision shall help minimizing damage to adjacent foundation, an analysis of bearing capacity and settlement shall be carried out to have an appreciation of the effect on the adjacent existing foundation. Suitable site-specific protection measures shall be adopted for the existing foundation, as required.

6.10 Preliminary Work for Construction

6.10.1 The construction of access roads, main sewers and drains should preferably be completed before commencing the work of foundations; alternatively, sufficient precautions shall be taken to protect the already constructed foundations during subsequent work.

6.10.2 *Clearance of Site*

Any obstacles, including the stump of trees, likely to interfere with the work shall be removed. Holes left by digging, such as those due to removal of old foundation, uprooted trees, burrowing by animals, etc, shall be back-filled with soil and well compacted.

6.10.3 *Drainage*

If the site of a structure is such that surface water drain towards it, land may be dressed or drains laid to divert the water away from the site.

6.10.4 *Setting Out*

Generally, the site shall be levelled before the layout of foundations are set out. In case of sloping terrain, care shall be taken to ensure that the dimensions on plans are set out correctly in one or more horizontal planes.

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6.10.5 The layout of foundations shall be set out in accordance of good practice [C-2(31)]. The setting out of walls shall be facilitated by permanent row of pillars, parallel to and at a suitable distance beyond the periphery of the building. The pillars shall be located at junctions of cross walls with the peripheral line of pillars. The centre lines of the cross walls shall be extended to and permanently erected on the plastered tops of the corresponding sets of pillars. The datum lines parallel to and at the known fixed distance from the centre lines of the external walls also be permanently worked on the corresponding rows of pillars to pillars to serve as checks on the accuracy of the work as it proceeds. The tops of these pillars shall be at the same level and preferably at the plinth or floor level. The pillars shall be of sizes not less than 250 mm wide and shall be bedded sufficiently deep into ground, so that they are not disturbed.

6.11 Protection of Excavation

6.11.1 The protection of excavation during construction shall be done in accordance with good practice [C-2(32)].

6.11.2 After excavation, the bottom of the excavation shall be cleared of all loose soil and rubbish and shall be levelled, where necessary. The bed shall be wetted and compacted by heavy rammers to an even surface.

6.11.3 Excavation in clay or other soils that are liable to be effected by exposure to atmosphere shall, wherever possible, be concreted as soon as they are dug. Alternatively, the bottom of the excavation shall be protected immediately by 80 mm thick layer of cement concrete not leaner than mix 1 : 5 : 10 over which shall come the foundation concrete; or in order to obtain a dry hard bottom, the last excavation of about 100 mm shall be removed only before concreting.

6.11.4 The backfilling of the excavation shall be done with care so as not to disturb the constructed foundation and shall be compacted in layers not exceeding 150 mm thick with sprinkling of minimum quantity of water necessary for proper compaction.

6.11.5 To support excavation sides where open excavation is required beyond a certain depth and safe slopes are not feasible, shoring shall be provided. This ensures stability, allows near-vertical soil cutting and prevents soil collapse. Shoring systems can include concrete shoring pile walls (secant or tangent wall systems), soldier pile walls, diaphragm walls, or other suitable methods, depending on soil conditions, excavation depth and selected construction technology.

Temporary shoring is generally defined as a system intended for service for a limited period. If earth retention structures are required for an extended duration, they should be designed as permanent structures.
For temporary shoring, the design should consider soil pressure and ensure that the selected system, including piles and anchors (if provided), can safely support the applied loads under service conditions.

The type of earth pressure to be considered depends on the flexibility and support conditions of the shoring system. Cantilevered or flexible walls with limited support require a specific earth pressure approach, whereas more rigid walls that exhibit minimal deflection require a different consideration.

For permanent conditions, all final loads, including surcharge effects, must be resisted by the structural walls, designed with appropriate safety considerations. The structural design should ensure:

a) deflections remain within allowable limits; and

b) structural elements maintain adequate strength and stability.

6.12 Alterations During Construction

a) Where during construction the soil or rock to which foundation is to transfer loads is found not to be the type or in the condition assumed, the foundation shall be redesigned and constructed for the existing type or conditions and the Authority notified;

b) Where a foundation bears on gravel, sand or silt and where the highest level of the ground water is or likely to be higher than an elevation defined by bearing surface minus the width of the footing, the bearing pressure shall be suitably altered;

c) Where the foundation has not been placed or located as indicated earlier or is damaged or bears on a soil whose properties may be adversely changed by climatic and construction conditions, the error shall be

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corrected, the damaged portion repaired or the design capacity of the affected foundation recalculated to the satisfaction of the Authority; and

d) Where a foundation is placed and if the results of a load test so indicate, the design of the foundation shall be modified to ensure structural stability of the same.

7 SHALLOW FOUNDATIONS

7.1 Design Information

For the satisfactory design of foundations, the following information is necessary:

a) Type and condition of the soil or rock to which the foundation transfers the loads;

b) General layout of the columns and load-bearing walls showing the estimated loads, including moments and torques due to various loads (dead load, imposed load, wind load, seismic load) coming on the foundation units;

c) Allowable bearing pressure of the soils;

d) Changes in ground water level, drainage and flooding conditions and also the chemical conditions of the subsoil water;

e) Behaviour of the buildings, topography and environment/surroundings adjacent to the site, the type and depths of foundations and the bearing pressure; and

f) Seismic zone of the region.

7.2 Design Considerations

7.2.1 *Design Loads*

The foundation shall be proportioned for the following combination of loads:

a) Dead load + Imposed load; and

b) Dead load + Imposed load + Wind load or seismic loads, whichever is critical.

For details, reference shall be made to Part C 'Structural Design/ Section 1 Loads and Load Effects' of this NBCS.

NOTE — For shallow foundations on coarse grained soils, settlements shall be estimated corresponding to **7.2.1**(b) and for foundations on fine grained soils, the settlement shall be estimated corresponding to permanent loads only. Permanent loads shall be in accordance with good practice [C-2(33)].

7.2.2 *Allowable Bearing Pressure*

The allowable bearing pressure shall be taken as either of the following, whichever is less:

a) the safe bearing capacity on the basis of shear strength characteristics of soil; and

b) the bearing pressure that the soil can take without exceeding the permissible settlement (*see* **7.2.3**).

7.2.2.1 *Bearing capacity by calculation*

Where the engineering properties of the soil are available, that is, cohesion, angle of internal friction, density, etc, the bearing capacity shall be calculated from stability considerations of shear; factor of safety of 2.5 shall be adopted for safe bearing capacity. The effect of interference of different foundations should be taken into account. The procedure for determining the ultimate bearing capacity and bearing pressure of shallow foundations based on shear and allowable settlement criteria shall be in accordance with good practice [C-2(34)]. Depth factor correction is to be applied only when backfilling is done with proper compaction.

7.2.2.2 *Field method for determining allowable bearing pressure*

Where appropriate, plate load tests can be performed and allowable pressure determined as per accepted standard [C-2(9)]. The allowable bearing pressure for sandy soils may also be obtained by loading tests. When such tests cannot be done, the allowable bearing pressure for sands may be determined using penetration test.

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7.2.2.3 Where the bearing materials directly under a foundation over-lie a stratum having smaller safe bearing capacity, these smaller values shall not be exceeded at the level of such stratum.

7.2.2.4 *Effect of wind and seismic force*

7.2.2.4.1 Where the bearing pressure due to wind is less than 25 percent of that due to dead and imposed loads, it may be neglected in design. Where this exceeds 25 percent, foundations may be so proportioned that the pressure due to combined dead, imposed and wind loads does not exceed the allowable bearing pressure by more than 25 percent.

7.2.2.4.2 When earthquake forces are considered for the computation of design loads, the permissible increase in bearing pressure of pertaining soil shall be as given in Part C 'Structural Design/Section 1 Loads and Load Effects'of this NBCS, depending upon the type of foundation of the structure and the type of soil.

7.2.2.5 *Bearing capacity of buried strata*

If the base of a foundation is close enough to a strata of lower bearing capacity, the latter may fail due to excess pressure transmitted to it from above. Care should be taken to see that the pressure transmitted to the lower strata is within the prescribed safe limits. When the footings are closely spaced, the pressure transmitted to the underlying soil will overlap. In such cases, the pressure in the overlapped zones will have to be considered. With normal foundations, it is sufficiently accurate to estimate the bearing pressure on the underlying layers by assuming the load to be spread at a slope of 2 (vertical) to 1 (horizontal).

7.2.3 *Settlement*

The permissible values of total and differential settlement for a given type of structure may be taken as given in Table 3. Total settlements of foundation due to net imposed loads shall be estimated in accordance with accepted standard [C-2(6)]. The following causes responsible for producing the settlement shall be investigated and taken into account.

a) *Causes of Settlement*:

1) Elastic compression of the foundation and the underlying soil;
2) Consolidation including secondary compression;
3) *Ground water lowering* — Specially repeated lowering and raising of water level in loose granular soils tend to compact the soil and cause settlement of the footings. Prolonged lowering of the water table in fine grained soils may introduce settlement because of the extrusion of water from the voids. Pumping water or draining water by tiles or pipes from granular soils without an adequate mat of filter material as protection may, in a period of time, carry a sufficient amount of fine particles away from the soil and cause settlement;
4) Seasonal swelling and shrinkage of expansive clays;
5) Ground movement on earth slope, for example, surface erosion, slow creep or landslides; and
6) Other causes, such as adjacent excavation, mining, subsidence and underground erosion.

b) *Causes of Differential Settlements*:

1) Geological and physical non-uniformity or anomalies in type, structure, thickness and density of the soil medium (pockets of sand in clay, clay lenses in sand, wedge like soil strata, that is, lenses in soil), an admixture of organic matter, peat, mud;
2) Non-uniform pressure distribution from foundation to the soil due to non-uniform loading and incomplete loading of the foundations;
3) Water regime at the construction site,
4) Overstressing of soil at adjacent site by heavy structures built next to light ones;
5) Overlap of stress distribution in soil from adjoining structures;
6) Unequal expansion of the soil due to excavation for footing;

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7) Non-uniform development of extrusion settlements; and

8) Non-uniform structural disruptions or disturbance of soil due to freezing and thawing, swelling and softening and drying of soils.

7.2.4 *Shallow Foundations on Rocks*

Estimation of the safe bearing pressures of rocks for shallow foundations based on strength, allowable settlement and classification criteria; and also design and construction of shallow foundations on rocks shall be carried out in accordance with the good practice [C-2(35)].

7.3 Pad or Spread and Strip Foundations

7.3.1 In such type of foundations, wherever the resultant of the load deviates from the centre line by more than 1/6 of its least dimension at the base of footing, it should be suitably reinforced.

7.3.2 For continuous wall foundations (plain or reinforced) adequate reinforcement should be provided particularly at places where there is abrupt change in magnitude of load or variation in ground support.

7.3.3 On sloping sites the foundation should have a horizontal bearing and stepped and lapped at changes of levels for a distance at least equal to the thickness of foundation or twice the height of step, whichever is greater. The steps should not be of greater height than thickness of the foundations.

7.3.4 *Ground Beams*

The foundation can also have the ground beam for transmitting the load. The ground beam carrying a load bearing wall should be designed to act with the wall forming a composite beam, when both are of reinforced concrete and structurally connected by reinforcement. The ground beam of reinforced concrete structurally connected to reinforced brick work can also be used.

7.3.5 *Dimensions of Foundation*

The dimensions of the foundation in plan should be such as to support loads as given in good practice [C-2(33)]. The width of the footings shall be such that maximum stress in the concrete or masonry is within the permissible limits. The width of wall foundation (in mm) shall not be less than that given by:

$$B = W + 300$$

where

B = width at base, in mm; and

W = width of supported wall, in mm.

7.3.6 In the base of foundations for masonry foundation it is preferable to have the steps in multiples of thickness of masonry unit.

7.3.7 The plan dimensions of excavation for foundations should be wide enough to ensure safe and efficient working in accordance with good practice [C-2(32)].

7.3.8 Unreinforced foundation may be of concrete or masonry (stone or brick) provided that angular spread of load from the base of column/wall or bed plate to the outer edge of the ground bearing is not more than 1 vertical to 1/2 horizontal for masonry or 1 vertical to 1 horizontal for cement concrete and 1 vertical to 2/3 horizontal for lime concrete. The minimum thickness of the foundation of the edge should not be less than 150 mm. In case the depth to transfer the load to the ground bearing is less than the permissible angle of spread, the foundations should be reinforced.

7.3.9 If the bottom of a pier is to be belled so as to increase its load carrying capacity such bell should be at least 300 mm thick at its edge. The sides should be sloped at an angle of not less than 45° with the horizontal. The least dimension should be 600 mm (circular, square or rectangular). The design should allow for the vertical tilt of the pier by 1 percent of its height.

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7.3.10 If the allowable bearing capacity is available only at a greater depth, the foundation can be rested at a higher level for economic considerations and the difference in level between the base of foundation and the depth at which the allowable bearing capacity occurs can be filled up with either: (a) concrete of allowable compressive strength not less than the allowable bearing pressure, (b) incompressible fill material, for example, sand, gravel, etc, in which case the width of the fill should be more than the width of the foundation by an extent of dispersion of load from the base of the foundation on either side at the rate of 2 vertical to 1 horizontal.

7.3.11 The cement concrete foundation (plain or reinforced) should be designed in accordance with Part C 'Structural Design/ Section 5 Structural Concrete' of this NBCS and masonry foundation in accordance with Part C 'Structural Design/ Section 4 Masonry' of this NBCS.

7.3.12 *Thickness of Footing*

The thickness of different types of footings, if not designed according to **7.2**, should be as given in Table 4.

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Table 3 Permissible Differential Settlements and Tilt (Angular Distortion) for Shallow Foundations in Soils

(*Clause* 7.2.3)

Sl No.	Type of Structure	Isolated Foundations						Raft Foundations					
		Sand and Hard Clay			Plastic Clay			Sand and Hard Clay			Plastic Clay		
		Maximum Settlement	Differential Settlement	Angular Distortion	Maximum Settlement	Differential Settlement	Angular Distortion	Maximum Settlement	Differential Settlement	Angular Distortion	Maximum Settlement	Differential Settlement	Angular Distortion
		mm	mm		mm	mm		mm	mm		mm	mm	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
i)	For steel structure	50	.003 3 *L*	1/300	50	.003 3 *L*	1/300	75	.003 3 *L*	1/300	100	.003 3 *L*	1/300
ii)	For reinforced concrete structures	50	.001 5 *L*	1/666	75	.001 5 *L*	1/666	75	.002 1 *L*	1/500	125	.002 *L*	1/500
iii)	For multistoreyed buildings												
	a) RC or steel framed buildings with panel walls	60	.002 *L*	1/500	75	.002 *L*	1/500	75	.0025 *L*	1/400	125	.003 3 L	1/300
	b) For load bearing walls												
	1) *L*/H=2[1)]	60	.000 2 *L*	1/5 000	60	.000 2 *L*	1/5 000	←		Not likely to be encountered		→	
	2) *L*/H=7[1)]	60	.000 4 *L*	½ 500	60	.000 4 *L*	1/2 500						
iv)	For water towers and silos	50	.001 5 *L*	1/666	75	.001 5 *L*	1/666	100	.0025*L*	1/400	125	.002 5 *L*	1/400

NOTES

1 The values given in the table may be taken only as a guide and the permissible total settlement/differential settlement and tilt (angular distortion) in each case should be decided as per requirements of the

2 *L* denotes the length of deflected part of wall/raft or centre-to-centre distance between columns.

3 *H* denotes the height of wall from foundation footing.

[1)] For intermediate ratios of *L*/*H*, the values can be interpolated.

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Table 4 Thickness of Footings

(*Clause* 7.3.12)

Sl No.	Type of Footings	Thickness of Footings, *Min*	Remarks
(1)	(2)	(3)	(4)
i)	Masonry	a) 250 mm; and b) Twice the maximum projection from the face of the wall.	Select the greater of the two values
ii)	Plain concrete		
	a) For normal structures	a) 200 mm; b) Twice the maximum offset in a stepped footing; and c) 300 mm.	— For footings resting on soil For footings resting on top of the pile
	b) For lightly loaded structures	a) 150 mm; and b) 200 mm.	Resting on soil Resting on pile
iii)	Reinforced concrete	a) 150 mm; and b) 300 mm.	Resting on soil Resting on pile

7.3.13 *Land Slip Area*

On a sloping site, spread foundation shall be on a horizontal bearing and stepped. At all changes of levels, they shall be lapped at the steps for a distance at least equal to the thickness of the foundation or twice the height of the step, whichever is greater. The steps shall not be of greater height than the thickness of the foundation unless special precautions are taken. On sloping ground on clay soils, there is always a tendency for the upper layers of soil to move downhill, depending on type of soil, the angle of slope, climatic conditions, etc. Special precautions are necessary to avoid such a failure.

7.3.14 In the foundations, the cover to the reinforcement shall be as prescribed in Part C 'Structural Design/ Section 5 Structural Concrete' of this NBCS for the applicable environment exposure condition.

7.3.15 For detailed information regarding preparation of ground work, reference shall be made to good practice [C-2(36)].

7.4 Raft Foundations

7.4.1 *Design Considerations*

Design provisions given in 7.2 shall generally apply.

7.4.1.1 The structural design of reinforced concrete rafts shall conform to Part C 'Structural Design/Section 5 Structural Concrete' of this NBCS.

7.4.1.2 In the case of raft, whether resting on soil directly or on lean concrete, the cover to the reinforcement shall be as prescribed in Part C 'Structural Design/ Section 5 Structural Concrete' of this NBCS for the applicable environment exposure condition.

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7.4.1.3 In case the structure supported by the raft consists of several parts with varying loads and heights, it is advisable to provide separation joints between these parts. Joints shall also be provided, wherever there is a change in the direction of the raft.

7.4.1.4 Foundations subject to heavy vibratory loads should preferably be isolated.

7.4.1.5 The minimum depth of foundation shall generally be not less than 1 m.

7.4.1.6 *Dimensional parameters*

The size and shape of the foundation adopted affect the magnitude of subgrade modulus and long term deformation of the supporting soil and this, in turn, influences the distribution of contact pressure. This aspect needs to be taken into consideration in the analysis.

7.4.1.7 *Eccentricity of loading*

A raft generally occupies the entire area of the building and often it is not feasible and rather uneconomical to proportion it coinciding the centroid of the raft with the line of action of the resultant force. In such cases, the effect of the eccentricity on contact pressure distribution shall be taken into consideration.

7.4.1.8 *Properties of supporting soil*

Distribution of contact pressure underneath a raft is affected by the physical characteristics of the soil supporting it. Consideration shall be given to the increased contact pressure developed along the edges of foundation on cohesive soils and the opposite effect on granular soils. Long term consolidation of deep soil layers shall be taken into account in the analysis. This may necessitate evaluation of contact pressure distribution both immediately after construction and after completion of the consolidation process. The design shall be based on the worst conditions.

7.4.1.9 *Rigidity of foundations*

Rigidity of the foundation tends to iron out uneven deformation and thereby modifies the contact pressure distribution. High order of rigidity is characterized by long moments and relatively small, uniform settlements. A rigid foundation may also generate high secondary stresses in structural members. The effect of rigidity shall be taken into account in analysis.

7.4.1.10 *Rigidity of the super structure*

Free response of the foundations to soil deformation is restricted by the rigidity of the superstructure. In the extreme case, a stiff structure may force a flexible foundation to behave as rigid. This aspect shall be considered to evaluate the validity of the contact pressure distribution.

7.4.1.11 *Modulus of elasticity and modulus of subgrade reaction*

Annex A enumerates the methods of determination of modulus of elasticity (E_s). The modulus of subgrade reaction (k) may be determined in accordance with Annex B.

7.4.2 *Necessary Information*

The following information is necessary for a satisfactory design and construction of a raft foundation:

a) Site plan showing the location of the proposed as well as the neighbouring structures;

b) Plan and cross-sections of building showing different floor levels, shafts and openings, etc, layout of load bearing walls, columns, shear walls, etc;

c) Loading conditions, preferably shown on a schematic plan indicating combination of design loads transmitted to the foundation;

d) Information relating to geological history of the area, seismicity of the area, hydrological information indicating ground water conditions and its seasonal variations, etc;

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e) Geotechnical information giving subsurface profile with stratification details, engineering properties of the founding strata (namely, index properties, effective shear parameters determined under appropriate drainage conditions, compressibility characteristics, swelling properties, results of field tests like static and dynamic penetration tests, pressure meter tests, etc); and

f) A review of the performance of similar structure, if any, in the locality.

7.4.3 *Choice of Raft Type*

7.4.3.1 For fairly small and uniform column spacing and when the supporting soil is not too compressible a flat concrete slab having uniform thickness throughout (a true mat) is most suitable [*see* Fig. 7(A)].

7.4.3.2 A slab may be thickened under heavy loaded columns to provide adequate strength for shear and negative moment. Pedestals may also be provided in such cases [*see* Fig. 7(B)].

7.4.3.3 A slab and beam type of raft is likely to be more economical for large column spacing and unequal column loads particularly when the supporting soil is very compressive [*see* Fig. 7(C) and Fig. 7(D)].

7.4.3.4 For very heavy structures, provision of cellular raft or rigid frames consisting of slabs and basement walls may be considered.

7.4.4 *Methods of Analysis*

The essential task in the analysis of a raft foundation is the determination of the distribution of contact pressure underneath the raft which is a complex function of the rigidity of the superstructure, the supporting soil and the raft itself and cannot be determined with exactitude, except in very simple cases. This necessitates a number of simplifying assumptions to make the problem amenable to analysis. Once the distribution of contact pressure is determined, design bending moments and shears can be computed based on statics. The methods of analysis suggested are distinguished by the assumptions involved. Choice of a particular method should be governed by the validity of the assumptions in the particular case.

7.4.4.1 *Rigid foundation* (*conventional method*)

7.4.4.1.1 This method is based on the assumption of linear distribution of contact pressure. The basic assumptions of this method are:

a) the foundations rigid relative to the supporting soil and the compressible soil layer is relatively shallow; and

b) the contact pressure variation is assumed as planar, such that the centroid of the contact pressure coincides with the line of action of the resultant force of all loads acting on the foundation.

7.4.4.1.2 This method may be used when either of the following conditions is satisfied:

a) the structure behaves as rigid (due to the combined action of the superstructure and the foundation) with relative stiffness factor $K > 0.5$ (for evaluation of K, *see* Annex C); and

b) the column spacing is less than $1.75/\lambda$ (*see* Annex C).

7.4.4.1.3 The raft is analyzed as a whole in each of the two perpendicular directions. The contact pressure distribution is determined by the procedure outlined in Annex D. Further analysis is also based on statics.

7.4.4.1.4 In the case of uniform conditions when the variations in adjacent column loads and column spacings do not exceed 20 percent of the higher value, the raft may be divided into perpendicular strips of widths equal to the distance between mid-spans and each strip may be analyzed as an independent beam with known column loads and known contact pressures. Such beams will not normally satisfy statics due to shear transfer between adjacent strips and design may be based on suitable moment coefficients, or by moment distribution.

NOTE — On soft soils, for example, normally consolidated clays, peat, muck, organic silts, etc, the assumptions involved in the conventional method are commonly justified.

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7.4.4.2 *Flexible foundations*

a) *Simplified method* — In this method, it is assumed that the subgrade consists of an infinite array of individual elastic springs each of which is not affected by others. The spring constant is equal to the modulus of subgrade reaction (k). The contact pressure at any point under the raft is, therefore, linearly proportional to the settlement at the point. Contact pressure may be determined as given in Annex E. This method may be used when all the following conditions are satisfied; and

1) The structure (combined action of superstructure and raft) may be considered as flexible (relative stiffness factor $K < 0.5$, *see* Annex C); and

2) Variation in adjacent column load does not exceed 20 percent of the higher value.

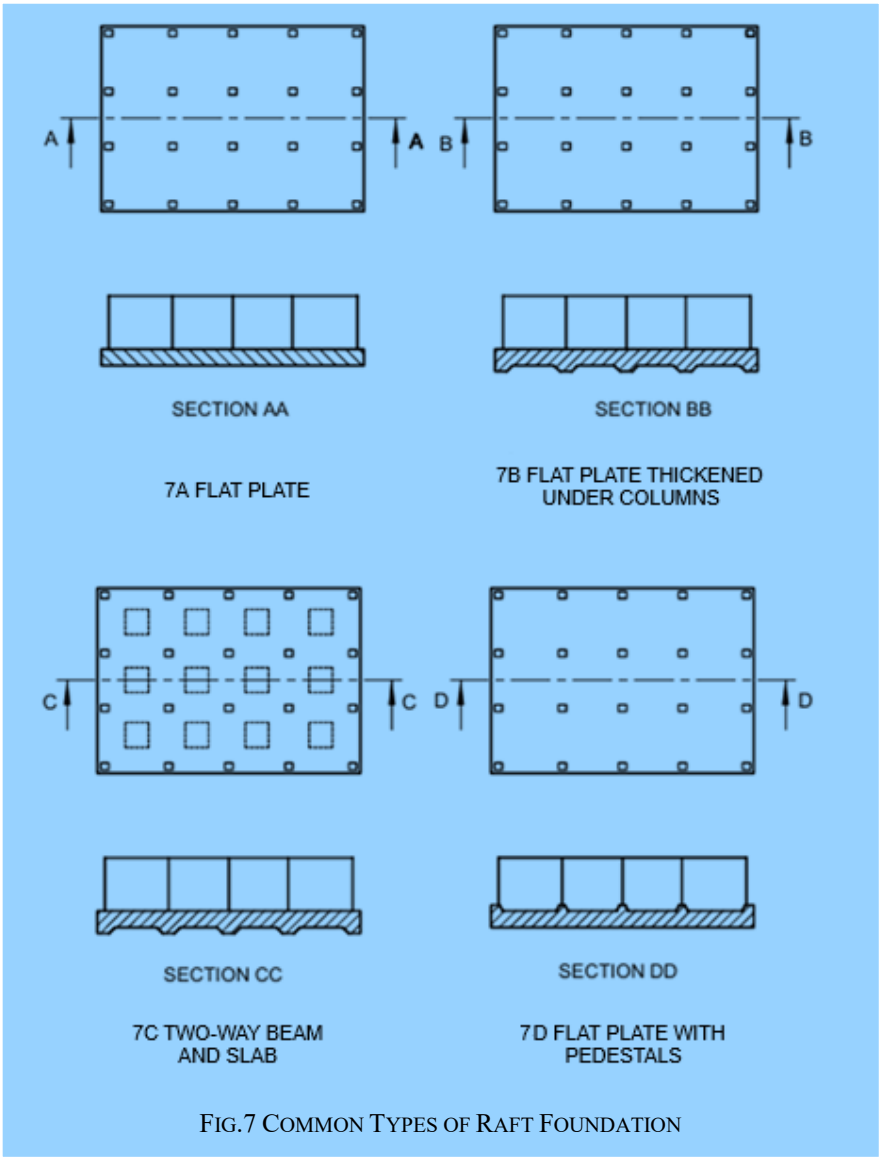

FIG.7 COMMON TYPES OF RAFT FOUNDATION

b) *General method* — For the general case of a flexible foundation not satisfying the requirements of (a) the method based on closed form solution of elastic plate theory may be used. This method is based on the theory of plates on winkler foundation which takes into account the restraint on deflection of a point provided by continuity of the foundation in orthogonal foundation. The distribution of deflection and contact pressure on the raft due to a column load is determined by the plate theory. Since the effect of a column load on an elastic foundation is damped out rapidly, it is possible to determine the total effect at a point of all column loads within the zone of influence by the method of superimposition. The computation of effect at any point may be restricted to columns of two adjoining bays in all directions. The procedure is outlined in Annex F.

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7.5 Ring Foundations

For provisions regarding ring foundations, good practice [C-2(37)] shall be referred to.

7.6 Shell Foundations

For provisions regarding shell foundations, good practice [C-2(38)] shall be referred to.

8 DEEP FOUNDATIONS

8.1 Deep foundations are structural elements used to transfer loads from a building or structure through weak, compressible soil to stronger, more competent soil or rock at depth. They're essential when shallow foundations, such as footings, are not feasible due to inadequate bearing capacity or excessive settlement.

Some important types of deep foundations which are being used in building and other structures are as follows:

a) Pile foundations; and
   1) driven/bored cast *in-situ* concrete piles;
   2) driven precast concrete piles;
   3) precast concrete piles in prebored holes;
   4) under- reamed piles; and
   5) timber piles.

b) Pier foundation

8.2 Driven/Bored Cast *In-Situ* Concrete Piles

8.2.1 *General*

Piles find application in foundations to transfer loads from a structure to competent subsurface strata having adequate load-bearing capacity. The load transfer mechanism from a pile to the surrounding ground is complicated and is not yet fully understood, although application of pile foundations is in practice over many decades. Broadly, piles transfer axial loads either substantially by friction along its shaft and/or by end bearing. Piles are used where either of the above load transfer mechanism is possible depending upon the subsoil stratification at a particular site. Construction of pile foundations require a careful choice of piling system depending upon the subsoil conditions, the load characteristics of a structure and the limitations of total settlement, differential settlement and any other special requirement of a project.

8.2.2 *Materials and Stresses*

8.2.2.1 *Concrete*

Consistency of concrete to be used shall be consistent with the method of installation of piles. Concrete shall be so designed or chosen as to have a homogeneous mix having a slump/workability consistent with the method of concreting under the given conditions of pile installation.

The slump should be 150 mm to180 mm at the time of pouring.

The minimum grade of concrete to be used for bored piling shall be M 25. For sub-aqueous concrete, the requirements specified in Part C ‘Structural Design/ Section 5 Structural Concrete’ of this NBCS shall be followed. The minimum cement content shall be 400 kg/m^3. However, with proper mix design and use of proper admixture the cement content may be reduced but in no case the cement content shall be less than 350 kg/m^3 [*See also* Table 5 (and Notes thereunder) of Part C ‘Structural Design/ Section 5 Structural Concrete’ of this NBCS].

For the concrete, water and aggregates, specifications laid down in Part C ‘Structural Design/ Section 5 Structural Concrete’ of this NBCS shall be followed in general.

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The average compressive stress under working load should not exceed 25 percent of the specified works cube strength at 28 days calculated on the total cross-sectional area of the pile. Where the casing of the pile is permanent, of adequate thickness and of suitable shape, the allowable compressive stress may be increased.

8.2.2.2 *Steel reinforcement*

Steel reinforcement shall conform to any one of the types of steel specified in Part C 'Structural Design/ Section 5 Structural Concrete' of this NBCS.

8.2.3 *Design Considerations*

8.2.3.1 *General*

Pile foundations shall be designed in such a way that the load from the structure can be transmitted to the subsurface with adequate factor of safety against shear failure of subsurface and without causing such settlement (differential or total), which may result in structural damage and/or functional distress under permanent/transient loading. The pile shaft should have adequate structural capacity to withstand all loads (vertical, axial or otherwise) and moments which are to be transmitted to the subsoil and shall be designed according to Part C 'Structural Design/ Section 5 Structural Concrete' of this NBCS.

8.2.3.2 *Adjacent structures*

8.2.3.2.1 When working near existing structures, care shall be taken to avoid damage to such structures. The good practice [C-2(39)] may be used as a guide for studying qualitatively the effect of vibration on persons and structures.

8.2.3.2.2 In case of deep excavations adjacent to piles, proper shoring or other suitable arrangement shall be made to guard against undesired lateral movement of soil.

8.2.3.3 *Pile capacity*

The load carrying capacity of a pile depends on the properties of the soil in which it is embedded. Axial load from a pile is normally transmitted to the soil through skin friction along the shaft and end bearing at its tip. A horizontal load on a vertical pile is transmitted to the subsoil primarily by horizontal subgrade reaction generated in the upper part of the shaft. Lateral load capacity of a single pile depends on the soil reaction developed and the structural capacity of the shaft under bending. It would be essential to investigate the lateral load capacity of the pile using appropriate values of horizontal subgrade modulus of the soil. Alternatively, piles may be installed in rake.

8.2.3.3.1 The ultimate load capacity of a pile may be estimated by means of static formula on the basis of soil test results, or by using a dynamic pile formula using data obtained during driving the pile. However, dynamic pile driving formula should be generally used as a measure to control the pile driving at site. Pile capacity should preferably be confirmed by initial load tests {*see* good practice [C-2(40)]}. For rock-socketed piles, reference shall also be made to good practice [C-2(41)] for estimating the load capacity of piles.

The settlement of pile obtained at safe load/working load from load-test results on a single pile shall not be directly used for estimating the settlement of a structure. The settlement may be determined on the basis of subsoil data and loading details of the structure as a whole using the principles of soil mechanics.

8.2.3.3.1.1 *Static formula*

The ultimate load capacity of a single pile may be obtained by using static analysis, the accuracy being dependent on the reliability of the soil properties for various strata. When computing capacity by static formula, the shear strength parameters obtained from a limited number of borehole data and laboratory tests should be supplemented, wherever possible by *in-situ* shear strength obtained from field tests. The two separate static formulae, commonly applicable for cohesive and non-cohesive soil respectively, are indicated in Annex G. Other formula based on static cone penetration test {*see* the accepted standards [C-2(42)]} and standard penetration test {*see* the accepted standard [C-2(6)]} are given in **G-3** and **G-4**.

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8.2.3.3.1.2 *Dynamic formula*

For driven piles, any established dynamic formula may be used to control the pile driving at site giving due consideration to limitations of various formulae.

Whenever, double acting diesel hammers or hydraulic hammers are used for driving of piles, manufacturer's guidelines about energy and set criteria may be referred to. Dynamic formulae are not directly applicable to cohesive soil deposits such as saturated silts and clays as the resistance to impact of the tip of the casing will be exaggerated by their low permeability while the frictional resistance on the sides is reduced by lubrication.

8.2.3.3.1.3 *Load test results*

The ultimate load capacity of a single pile is determined with reasonable accuracy from test loading as per good practice [C-2(40)]. The load test on a pile shall not be carried out earlier than four weeks from the time of casting the pile.

8.2.3.3.1.4 *Uplift capacity*

The uplift capacity of a pile is given by sum of the frictional resistance and the weight of the pile (buoyant or total as relevant). The recommended factor of safety is 3.0 in the absence of any pullout test results and 2.0 with pullout test results. Uplift capacity can be obtained from static formula (*see* Annex G) by ignoring end bearing but adding weight of the pile (buoyant or total as relevant).

8.2.3.4 *Negative skin friction or dragdown force*

When a soil stratum, through which a pile shaft has penetrated into a underlying hard stratum, compresses as a result of either it being unconsolidated or it being under a newly placed fill or as a result of remoulding during driving of the pile, a dragdown force is generated along the pile shaft up to a point in depth where the surrounding soil does not move downward relative to the pile shaft. Existence of such a phenomenon shall be assessed and suitable correction shall be made to the allowable load where appropriate.

8.2.3.5 *Structural capacity*

The piles shall have necessary structural strength to transmit the loads imposed on it, ultimately to the soil. In case of uplift, the structural capacity of the pile, that is, under tension should also be considered.

8.2.3.5.1 *Axial capacity*

Where a pile is completely embedded in the soil (having an undrained shear strength not less than 0.01 N/mm^2), its axial load carrying capacity is not necessarily limited by its strength as a long column. Where piles are installed through very weak soils (having an undrained shear strength less than 0.01 N/mm^2), special considerations shall be made to determine whether the shaft would behave as a long column or not. If necessary, suitable reductions shall be made for its structural strength following the normal structural principles covering the buckling phenomenon.

When the finished pile projects above ground level and is not secured against buckling by adequate bracing, the effective length will be governed by the fixity imposed on it by the structure it supports and by the nature of the soil into which it is installed. The depth below the ground surface to the lower point of contraflexure varies with the type of the soil. In good soil the lower point of contraflexure may be taken at a depth of 1 m below ground surface subject to a minimum of 3 times the diameter of the shaft. In weak soil (undrained shear strength less than 0.01 N/mm^2) such as soft clay or soft silt, this point may be taken at about half the depth of penetration into such stratum but not more than 3 m or 10 times the diameter of the shaft, whichever is more. The degree of fixity of the position and inclination of the pile top and the restraint provided by any bracing shall be estimated following accepted structural principles.

The permissible stress shall be reduced in accordance with similar provision for reinforced concrete columns as laid down in Part C 'Structural Design/ Section 5 Structural Concrete' of this NBCS.

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8.2.3.5.2 *Lateral load capacity*

A pile may be subjected to lateral force for a number of causes, such as wind, earthquake, water current, earth pressure, effect of moving vehicles or ships, plant and equipment, etc. The lateral load capacity of a single pile depends not only on the horizontal subgrade modulus of the surrounding soil but also on the structural strength of the pile shaft against bending, consequent upon application of a lateral load. While considering lateral load on piles, effect of other co-existent loads, including the axial load on the pile, should be taken into consideration for checking the structural capacity of the shaft. A recommended method for the pile analysis under lateral load is given in Annex H.

Because of limited information on horizontal subgrade modulus of soil and pending refinements in the theoretical analysis, it is suggested that the adequacy of a design should be checked by an actual field load test. In the zone of soil susceptible to liquefaction, the lateral resistance of the soil shall not be considered.

8.2.3.5.2.1 *Fixed and free head conditions*

A group of three or more piles connected by a rigid pile cap shall be considered to have fixed head condition. Caps for single piles shall be interconnected by grade beams in two directions and for twin piles by grade beams in a line transverse to the common axis of the pair so that the pile head is fixed. In all other conditions the pile shall be taken as free headed.

8.2.3.5.3 *Raker piles*

Raker piles are normally provided where vertical piles cannot resist the applied horizontal forces. Generally, the rake will be limited to 1 horizontal to 6 vertical. In the preliminary design, the load on a raker pile is generally considered to be axial. The distribution of load between raker and vertical piles in a group may be determined by graphical or analytical methods. Where necessary, due consideration should be made for secondary bending induced as a result of the pile cap movement, particularly when the cap is rigid. Free-standing raker piles are subjected to bending moments due to their own weight or external forces from other causes. Raker piles, embedded in fill or consolidating deposits, may become laterally loaded owing to the settlement of the surrounding soil. In consolidating clay, special precautions, like provision of permanent casing should be taken for raker piles.

8.2.3.6 *Spacing of piles*

The minimum centre to centre spacing of pile is considered from three aspects, namely:

a) Practical aspects of installing the piles;

b) Diameter of the pile; and

c) Nature of the load transfer to the soil and possible reduction in the load capacity of piles group.

NOTE — In the case of piles of non-circular cross-section, diameter of the circumscribing circle shall be adopted.

8.2.3.6.1 In case of piles founded on hard stratum and deriving their capacity mainly from end bearing the minimum spacing shall be 2.5 times the diameter of the circumscribing circle corresponding to the cross-section of the pile shaft. In case of piles resting on rock, the spacing of two times the said diameter may be adopted.

8.2.3.6.2 Piles deriving their load carrying capacity mainly from friction shall be spaced sufficiently apart to ensure that the zones of soils from which the piles derive their support do not overlap to such an extent that their bearing values are reduced. Generally, the spacing in such cases shall not be less than 3 times the diameter of the pile shaft.

8.2.3.7 *Pile groups*

8.2.3.7.1 In order to determine the load carrying capacity of a group of piles a number of efficiency equations are in use. However, it is difficult to establish the accuracy of these efficiency equations as the behaviour of pile group is dependent on many complex factors. It is desirable to consider each case separately on its own merits.

8.2.3.7.2 The load carrying capacity of a pile group may be equal to or less than the load carrying capacity of individual piles multiplied by the number of piles in the group. The former holds true in case of friction piles, driven into progressively stiffer materials or in end-bearing piles. For driven piles in loose sandy soils, the group

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capacity may even be higher due to the effect of compaction. In such cases a load test may be carried out on a pile in the group after all the piles in the group have been installed.

8.2.3.7.3 In case of piles deriving their support mainly from friction and connected by a rigid pile cap, the group may be visualized as a block with the piles embedded within the soil. The ultimate load capacity of the group may then be obtained by taking into account the frictional capacity along the perimeter of the block and end bearing at the bottom of the block using the accepted principles of soil mechanics.

8.2.3.7.3.1 When the cap of the pile group is cast directly on reasonably firm stratum which supports the piles, it may contribute to the load carrying capacity of the group. This additional capacity along with the individual capacity of the piles multiplied by the number of piles in the group shall not be more than the capacity worked out as per **8.2.3.7.3**.

8.2.3.7.4 When a pile group is subjected to moment either from superstructure or as a consequence of inaccuracies of installation, the adequacy of the pile group in resisting the applied moment should be checked. In case of a single pile subjected to moment due to lateral loads or eccentric loading, beams may be provided to restrain the pile cap effectively from lateral or rotational movement.

8.2.3.7.5 In case of a structure supported on single piles/group of piles resulting in large variation in the number of piles from column to column it may result in excessive differential settlement. Such differential settlement should be either catered for in the structural design or it may be suitably reduced by judicious choice of variations in the actual pile loading. For example, a single pile cap may be loaded to a level higher than that of the pile in a group in order to achieve reduced differential settlement between two adjacent pile caps supported on different number of piles.

8.2.3.8 *Factor of safety*

8.2.3.8.1 Factor of safety should be chosen after considering:

a) the reliability of the calculated value of ultimate load capacity of a pile;

b) the types of superstructure and the type of loading; and

c) allowable total/differential settlement of the structure.

8.2.3.8.2 When the ultimate load capacity is determined from either static formula or dynamic formula, the factor of safety would depend on the reliability of the formula and the reliability of the subsoil parameters used in the computation. The minimum factor of safety on static formula shall be 2.5. The final selection of a factor of safety shall take into consideration the load settlement characteristics of the structure as a whole at a given site.

8.2.3.8.3 Higher value of factor of safety for determining the safe load on piles may be adopted, where:

a) settlement is to be limited or unequal settlement avoided;

b) large impact or vibrating loads are expected; and

c) the properties of the soil may deteriorate with time.

8.2.3.9 *Transient loading*

The maximum permissible increase over the safe load of a pile, as arising out of wind loading, is 25 percent. In case of loads and moments arising out of earthquake effects, the increase of safe load on a single pile shall be as given in **7.2.2.4.2**. For transient loading arising out of superimposed loads, no increase is allowed.

8.2.3.10 *Overloading*

When a pile in a group, designed for a certain safe load is found, during or after execution, to fall just short of the load required to be carried by it, an overload up to 10 percent of the pile capacity may be allowed on each pile. The total overloading on the group should not, however, be more than 10 percent of the capacity of the group subject to the increase of the load on any pile being not more than 25 percent of the allowable load on a single pile.

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8.2.3.11 *Reinforcement*

8.2.3.11.1 The design of the reinforcing cage varies depending upon the driving and installation conditions, the nature of the subsoil and the nature of load to be transmitted by the shaft - axial, or otherwise. The minimum area of longitudinal reinforcement of any type or grade within the pile shaft shall be 0.4 percent of the cross-sectional area of the pile shaft. The minimum reinforcement shall be provided throughout the length of the shaft.

8.2.3.11.2 The curtailment of reinforcement along the depth of the pile, in general, depends on the type of loading and subsoil strata. In case of piles subjected to compressive load only, the designed quantity of reinforcement may be curtailed at appropriate level according to the design requirements. For piles subjected to uplift load, lateral load and moments, separately or with compressive loads, it would be necessary to provide reinforcement for the full depth of pile. In soft clays or loose sands, or where there may be danger to green concrete due to driving of adjacent piles, the reinforcement should be provided to the full pile depth, regardless of whether or not it is required from uplift and lateral load considerations. However, in all cases, the minimum reinforcement specified in **8.2.3.11.1** shall be provided throughout the length of the shaft.

8.2.3.11.3 Piles shall always be reinforced with a minimum amount of reinforcement as dowels keeping the minimum bond length into the pile shaft below its cut-off level and with adequate projection into the pile cap, irrespective of design requirements.

8.2.3.11.4 Clear cover to all main reinforcement in pile shaft shall be not less than 50 mm. The laterals of a reinforcing cage may be in the form of links or spirals. The diameter and spacing of the same is chosen to impart adequate rigidity of the reinforcing cage during its handling and installations. The minimum diameter of the links or spirals shall be 8 mm and the spacing of the links or spirals shall be not less than 150 mm. Stiffner rings preferably of 16 mm diameter at every 1.5 m centre to centre should be provided along the length of the cage for providing rigidity to reinforcement cage. Minimum 6 numbers of vertical bars shall be used for a circular pile and minimum diameter of vertical bar shall be 12 mm. The clear horizontal spacing between the adjacent vertical bars shall be four times the maximum aggregate size in concrete. If required, the bars can be bundled to maintain such spacing.

8.2.3.12 *Design of pile cap*

8.2.3.12.1 The pile caps may be designed by assuming that the load from column is dispersed at 45º from the top of the cap to the mid-depth of the pile cap from the base of the column or pedestal. The reaction from piles may also be taken to be distributed at 45º from the edge of the pile, up to the mid-depth of the pile cap. On this basis the maximum bending moment and shear forces should be worked out at critical sections. The method of analysis and allowable stresses should be in accordance with Part C 'Structural Design/ Section 5 Structural Concrete' of this NBCS.

8.2.3.12.2 Pile cap shall be deep enough to allow for necessary anchorage of the column and pile reinforcement.

8.2.3.12.3 The pile cap should be rigid enough so that the imposed load could be distributed on the piles in a group equitably.

8.2.3.12.4 In case of a large cap, where differential settlement may occur between piles under the same cap, due consideration for the consequential moment should be given.

8.2.3.12.5 The clear overhang of the pile cap beyond the outermost pile in the group shall be a minimum of 150 mm.

8.2.3.12.6 The cap is generally cast over a 75 mm thick levelling course of concrete. The clear cover for main reinforcement in the cap slab shall not be less than 60 mm.

8.2.3.12.7 The embedment of pile into cap should be 75 mm.

8.2.3.13 *Grade beams*

8.2.3.13.1 The grade beams supporting the walls shall be designed taking due account of arching effect due to masonry above the beam. The beam with masonry due to composite action behaves as a deep beam.

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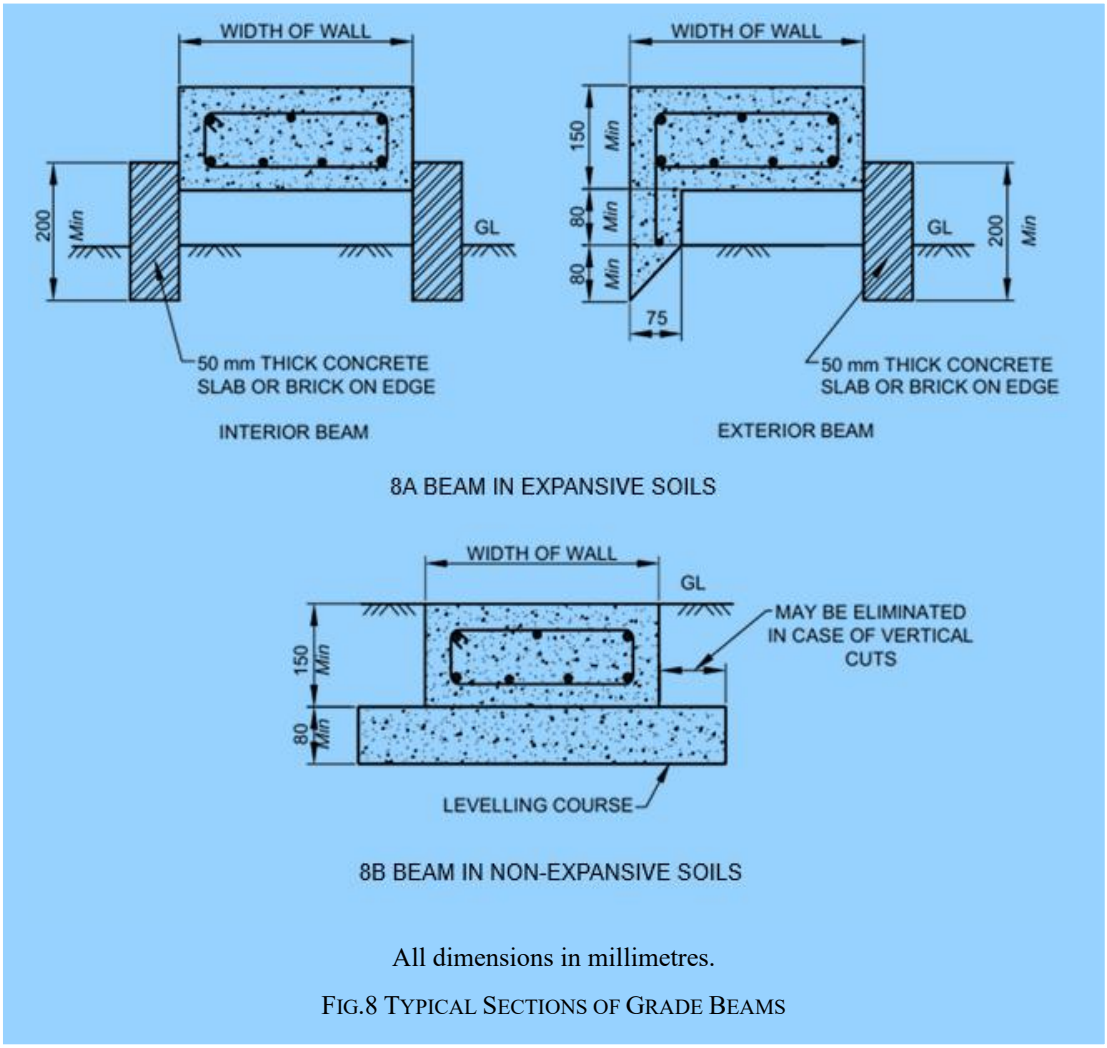
For the design of grade beams, a maximum bending moment of $\frac{wl^2}{30}$, where w is uniformly distributed load per metre run and l is the effective span in metres, shall be considered. For considering composite action, the minimum height of wall shall be 0.6 times the pile spacing. The brick strength should not be less than 3 N/mm^2. For concentrated and other loads which come directly over the beam, full bending moment should be considered.

8.2.3.13.2 The minimum overall depth of grade beams shall be 200 mm. An equal amount of reinforcement may be provided at the bottom and top of the beam. The longitudinal reinforcement both at top and bottom should not be less than three bars of 12 mm diameter with clear cover space of 50 mm. S pacing of stirrups of 8 mm diameter bars should not be more than the depth of grade beam.

8.2.3.13.3 In expansive soils, the grade beams and 75 mm levelling concrete course shall be provided over 40 mm down gravel layer/mechanically stabilized mix (MSM) layer having a minimum compacted thickness of 300 mm and minimum width equal to five times the width of grade beams (*see* Fig. 8).

8.2.3.13.4 In the case of exterior beams over piles in expansive soils, plinth protection with proper slope shall be provided.

8.2.4 For detailed information on driven/bored cast *in-situ* concrete piles regarding control of piling, installation, defective pile and recording of data, reference shall be made to good practice [C-2(43)].



All dimensions in millimetres.

FIG.8 TYPICAL SECTIONS OF GRADE BEAMS

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8.2.5 *Bored Cast In-Situ Concrete Piles on Rocks*

Design and construction of bored cast *in-situ* piles founded on rocks shall be carried out in accordance with good practice [C-2(41)].

8.2.6 *Non-Destructive Testing*

For quality assurance of concrete piles, non-destructive integrity test may be carried out prior to laying of beam/caps, in accordance with accepted standard [C-2(44)].

8.3 Driven Precast Concrete Piles

8.3.1 Provisions of **8.2** except **8.2.3.11** shall generally apply.

8.3.2 *Design of Pile Section*

8.3.2.1 Design of pile section shall be such as to ensure the strength and soundness of the pile against lifting from the casting bed, transporting, handling and driving stresses without damage.

8.3.2.2 Any shape having radial symmetry will be satisfactory for precast piles. The most commonly used cross-sections are square and octagonal.

8.3.2.3 Where exceptionally long lengths of piles are required, hollow sections can be used. If the final condition requires larger cross-sectional area, the hollow sections may be filled with concrete after driving in position.

8.3.2.4 Wherever, final pile length is so large that a single length precast pile unit is either uneconomical or impracticable for installation, the segmental precast RCC piles with a number of segments using efficient mechanical jointing could be adopted.

Excessive whipping during handling precast pile may generally be avoided by limiting the length of pile to a maximum of 50 times the least width. As an alternative, segmental precast piling technique could be used.

The design of joints shall take care of corrosion by providing additional sacrificial thickness for the joint, wherever warranted.

8.3.2.5 Stresses induced by bending in the cross-section of precast pile during lifting and handling may be estimated as for any reinforced concrete section in accordance with relevant provisions of Part C 'Structural Design/ Section 5 Structural Concrete' of this NBCS. The calculations for bending moment for different support conditions during handling are given in Table 5.

Table 5 Bending Moment for Different Support Conditions (*Clause* 8.3.2.5)

Sl No.	Number of Points of Pick Up	Location of Support From End in Terms of Length of Pile For Minimum Moments	Bending Moment to be Allowed for Design Kn-M
(1)	(2)	(3)	(4)
i)	One	0.293 *L*	4.3 *WL*
ii)	Two	0.207 *L*	2.2 *WL*
iii)	Three	0.145 *L*, the middle point will be at the centre	1.05 *WL*

NOTE — *W* = weight of pile, in kN;
L = length of pile, in m.

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8.3.2.6 The driving stresses on a pile may be estimated by the following formula:

$$\frac{\text{Driving resistance}}{\text{Cross} - \text{sectional area of the pile}} \times \left[\frac{2}{\sqrt{n}} - 1\right]$$

where

n = efficiency of the blow (*see* 8.3.2.6.1 for probable value of n).

NOTE — For the purpose of this formula, cross-sectional area of the pile shall be calculated as the overall sectional area of the pile including the equivalent area for reinforcement.

8.3.2.6.1 The formula for efficiency of the blow, representing the ratio of energy after impact to striking energy of ram, n, is:

where W is greater than $P.e$ and the pile is driven into penetrable ground,

$$\boldsymbol{n} = \frac{\boldsymbol{W} + (\boldsymbol{P} \cdot \boldsymbol{e^2})}{\boldsymbol{W} + \boldsymbol{P}}$$

where W is less than $P.e$ and the pile is driven into penetrable ground,

$$\boldsymbol{n} = \left[\frac{\boldsymbol{W}+(\boldsymbol{P}\cdot\boldsymbol{e^2})}{\boldsymbol{W}+\boldsymbol{P}}\right] - \left[\frac{\boldsymbol{W} - (\boldsymbol{P}\cdot\boldsymbol{e})}{\boldsymbol{W} + \boldsymbol{P}}\right]^2$$

The following are the values of n in relation to e and to the ratio of P/W:

Sl No.	*Ratio of P/W*	*e = 0.5*	*e = 0.4*	*e = 0.32*	*e = 0.25*	*e = 0*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	½	0.75	0.72	0.70	0.69	0.67
ii)	1	0.63	0.58	0.55	0.53	0.50
iii)	1½	0.55	0.50	0.47	0.44	0.40
iv)	2	0.50	0.44	0.40	0.37	0.33
v)	2½	0.45	0.40	0.36	0.33	0.28
vi)	3	0.42	0.36	0.33	0.30	0.25
vii)	3½	0.39	0.33	0.30	0.27	0.22
viii)	4	0.36	0.31	0.28	0.25	0.20
ix)	5	0.31	0.27	0.24	0.21	0.16
x)	6	0.27	0.24	0.21	0.19	0.14
xi)	7	0.24	0.21	0.19	0.17	0.12
xii)	8	0.22	0.20	0.17	0.15	0.11

NOTES

1 W = mass of the ram, in tonne and P = weight of the pile, anvil, helmet and follower (if any), in tonne.

2 Where the pile finds refusal in rock, $0.5P$ should be substituted for P in the above expressions for n.

3 e is the coefficient of restitution of the materials under impact as tabulated below:

a) For steel ram of double-acting hammer striking on steel anvil and driving reinforced concrete pile, $e = 0.5$;

b) For cast-iron ram of single-acting or drop hammer striking on head of reinforced concrete pile, $e = 0.4$;

c) Single-acting or drop hammer striking a well-conditioned driving cap and helmet with hard wood dolly in driving reinforced concrete piles or directly on head of timber pile, $e = 0.25$; and

d) For a deteriorated condition of the head of pile or of dolly, $e = 0$.

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8.3.3 *Reinforcement*

8.3.3.1 The longitudinal reinforcement of any type or grade shall be provided in precast reinforced concrete piles for the entire length. All the main longitudinal bars shall be of the same length and should fit tightly into the pile shoe, if there is one. Shorter rods to resist local bending moments may be added but the same should be carefully detailed to avoid any sudden discontinuity of the steel which may lead to cracks during heavy driving. The area of main longitudinal reinforcement shall not be less than the following percentages of the cross-sectional area of the piles:

a) For piles with a length less than 30 times the least width: 1.25 percent;

b) For piles with a length 30 to 40 times the least width: 1.5 percent; and

c) For piles with a length greater than 40 times the least width: 1.5 percent.

8.3.3.2 Piles shall always be reinforced with a minimum amount of reinforcement as dowels keeping the minimum bond length into the pile shaft below its cut-off level and with adequate projection into the pile cap, irrespective of design requirements.

8.3.3.3 Clear cover to all main reinforcement in pile shaft shall be not less than 50 mm. The laterals of a reinforcing cage may be in the form of links or spirals. The diameter and spacing of the same is chosen to impart adequate rigidity of the reinforcing cage during its handing and installations. The minimum diameter of the links or spirals shall be 8 mm and the spacing of the links or spirals shall be not less than 150 mm Stiffner rings preferably of 16 mm diameter at every 1.5 m centre to centre to be provided along the length of the cage for providing rigidity to reinforcement cage. Minimum 6 numbers of vertical bars shall be used for a circular pile and minimum diameter of vertical bar shall be 12 mm. The clear horizontal spacing between the adjacent vertical bars shall be four times the maximum aggregate size in concrete. If required, the bars may be bundled to maintain such spacing.

8.3.4 Additional provision for prestressed concrete piles shall be as per good practice [C-2(45)].

8.3.5 For detailed information regarding casting and curing, storing and handling, control of pile driving and recording of data, reference shall be made to good practice [C-2(45)].

8.3.6 *Non-Destructive Testing*

For quality assurance of concrete piles, non-destructive integrity test may be carried out prior to laying of beam/caps, in accordance with good practice [C-2(44)].

8.4 Precast Concrete Piles in Prebored Holes

8.4.1 Provisions of 8.3 except 8.3.3 shall generally apply.

8.4.2 *Handling Equipment for Lowering and Grouting Plant*

Handling equipment such as crane, scotch derricks, movable gantry may be used for handling and lowering the precast piles in the bore. The choice of equipment will depend upon length, mass and other practical requirements.

The mixing of the grout shall be carried out in any suitable high speed colloidal mixer. Normally the colloidal mixer is adequate to fill the annular space with grouts. Where this is not possible, a suitable grout pump shall be used.

8.4.3 *Reinforcement*

8.4.3.1 The design of the reinforcing cage varies depending upon the handling and installation conditions, the nature of the subsoil and the nature of load to be transmitted by the shaft - axial, or otherwise. The minimum area of longitudinal reinforcement of any type or grade within the pile shaft shall be 0.4 percent of the cross-sectional area of the pile shaft or as required to cater for handling stresses (*see* 8.3.2.5), whichever is greater. The minimum reinforcement shall be provided throughout the length of the shaft.

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8.4.3.2 Piles shall always be reinforced with a minimum amount of reinforcement as dowels keeping the minimum bond length into the pile shaft below its cut-off level and with adequate projection into the pile cap, irrespective of design requirements.

8.4.3.3 Clear cover to all main reinforcement in pile shaft shall be not less than 50 mm. The laterals of a reinforcing cage may be in the form of links or spirals. The diameter and spacing of the same is chosen to impart adequate rigidity of the reinforcing cage during its handing and installations. The minimum diameter of the links or spirals shall be 8 mm and the spacing of the links or spirals shall be not less than 150 mm. Stiffner rings preferably of 16 mm diameter at every 1.5 m centre to centre to be provided along length of the cage for providing rigidity to reinforcement cage. Minimum 6 numbers of vertical bars shall be used for a circular pile and minimum diameter of vertical bar shall be 12 mm. The clear horizontal spacing between the adjacent vertical bars shall be four times the maximum aggregate size in concrete. If required, the bars may be bundled to maintain such spacing.

8.4.3.4 A thin gauge sheathing pipe of approximately 40 mm diameter may be attached to the reinforcement cage, in case of solid piles, to form the central duct for pumping grout to the bottom of the bore. The bottom end of the pile shall have proper arrangements for flushing/cleaning for grouting. Air lift technique may be used for cleaning the borehole, however this technique should be used carefully in case of silty and sandy soil.

8.4.4 For detailed information regarding casting and curing, storing and handling, control of pile installation and recording of data, reference shall be made to good practice [C-2(46)].

8.4.5 *Non-Destructive Testing*

For quality assurance of concrete piles, non-destructive integrity test may be carried out prior to laying of beam/caps, in accordance with accepted standard [C-2(44)].

8.5 Under-Reamed Piles

8.5.1 *General*

Under-reamed piles are bored cast *in-situ* and bored compaction concrete types having one or more bulbs formed by suitably enlarging the borehole for the pileshaft. These piles are suited for expansive soils which are often subjected to considerable ground movements due to seasonal moisture variations. These also find wide application in other soil strata where economics are favorable. When the ground consists of expansive soil, for example black cotton soils, the bulb of under-reamed pile provide anchorage against uplift due to swelling pressure, apart from the increased bearing, provided topmost bulb is formed close to or just below the bottom of active zone. Negative slopes may not be stable in certain strata conditions, for example, in pure sands (clean sands with fines less than five per cent) and very soft clayey strata having N of SPT less than 2 (undrained shear strength of less than 12.5 kN/m^2). Hence formation of bulb(s) in such strata is not advisable. In subsoil strata above water table, the maximum number of bulbs in under-reamed pile should be restricted to four. In the strata such as clay, silty clay and clayey silt with high water table where sides of bore hole stand by itself without needing any stabilization by using drilling mud or otherwise, the maximum number of bulbs in under-reamed piles should be restricted to two. In strata for example clayey sand, silty sand and sandy silt with high water table where bore hole needs stabilization by using drilling mud, under-reamed piles with more than one bulb shall not be used. In loose to medium pervious strata such as clayey sand, silty sand and sandy silt strata, compaction under-reamed piles may be used as the process of compaction increases the load carrying capacity of piles. From practical considerations, under-reamed piles of more than 10 m depth shall not be used without ensuring their construction feasibility and load carrying capacity by initial load tests in advance. In view of additional anchorage available with the provision of bulbs, under-reamed piles can be used with advantage to resist uplift loads.

8.5.2 *Materials*

Provisions of **8.2.2** shall generally apply.

8.5.3 *Design Considerations*

8.5.3.1 *General*

Under-reamed pile foundation shall be designed in such a way that the load from the structure they support can be transmitted to the sub-surface with adequate factor of safety against shear failure of sub-surface and without

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causing such settlement (differential or total) which may result in structural damage and/or functional distress under permanent/transient loading. The pile shaft should have adequate structural capacity to withstand all loads (vertical compression, uplift, lateral) and moments which are to be transmitted to the subsoil and shall be designed according to [C-2(47)].

8.5.3.1.1 In expansive soils, the minimum length of piles shall be 3.5 m below ground level. In case of expansive soil deposits of shallow depth overlying non-expansive competent strata or rock, piles of shorter length can also be provided.

However, in case of high to very high expansive soils, due care shall be taken to establish the minimum depth of pile with the top-most (first) bulb located below the level of significant moisture variations/active zone.

8.5.3.1.2 The diameter of under-reamed bulbs may vary from 2 times to 2.5 times shaft diameter depending upon the feasibility of construction and design requirements. In bored cast *in-situ* under-reamed piles and under-reamed bored compaction piles, the bulb diameter shall be normally 2.5 times and 2 times the shaft diameter, respectively.

8.5.3.1.3 The spacing of bulbs should not be less than 1.5 times the diameter of bulb.

8.5.3.1.4 The first bulb should be at a minimum depth of 2 times the bulb diameter below ground level or cut-off level, whichever is lower.

8.5.3.1.5 Under-reamed piles with more than two bulbs are not advisable without ensuring their feasibility in strata needing stabilization of boreholes by drilling mud. The number of bulbs in case of under-reamed bored compaction piles should not exceed two in such strata.

8.5.3.1.6 The minimum diameter of pile shaft shall be 250 mm. However, in case of strata/ground water consisting of excessive chemicals such as sulphates and chlorides, or piles with tremie concreting, the minimum diameter of the pile shaft shall be 300 mm.

8.5.3.1.7 Under-reamed batter piles in dry soil conditions can be constructed with batter not exceeding 15°.

8.5.3.1.8 The construction feasibility of under-ream/ bulb should be examined as per soil characteristics, particularly in case of clean/river sands.

8.5.3.1.9 For guidance, typical details of bored cast *in-situ* under-reamed pile foundations are shown in Fig. 9.

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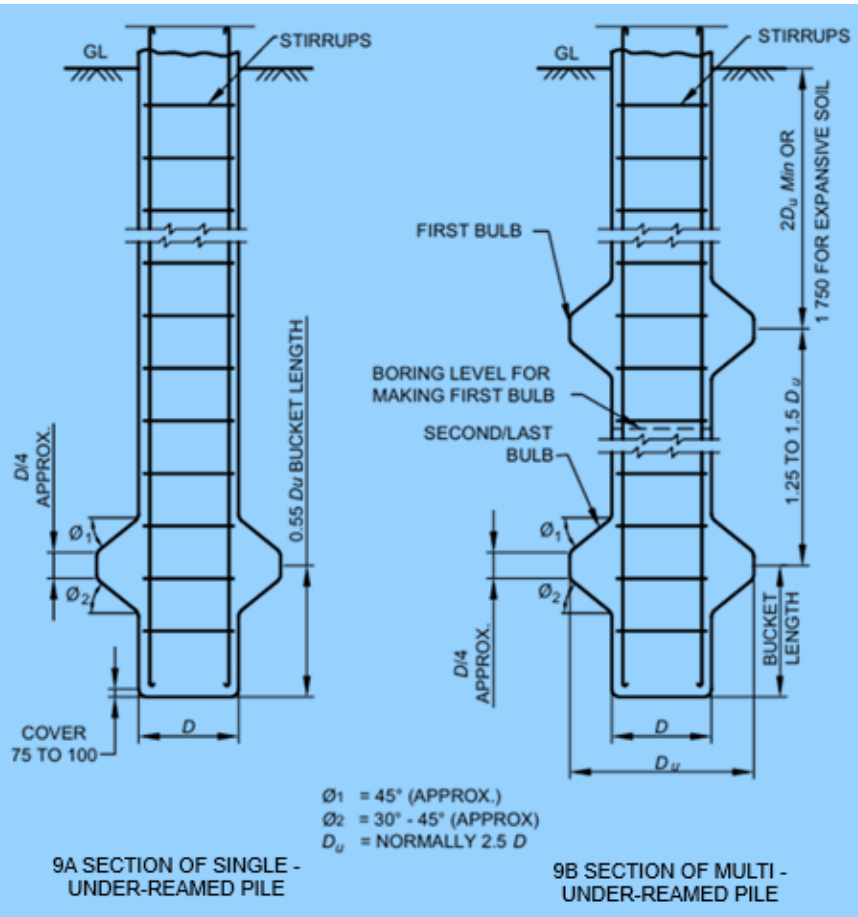

FIG. 9 TYPICAL DETAILS OF BORED CAST IN-SITU UNDER-REAMED PILE FOUNDATION

8.5.3.2 *Safe load*

Safe load on a pile shall be taken as the least of the following:

a) By calculating the ultimate load from soil properties and applying a suitable factor of safety (*see* Annex J);

b) By load test on pile as good practice [C-2(40)]; and

c) From safe load tables.

8.5.3.2.1 *Load test*

Provisions of **8.2.3.3.1.3** shall generally apply.

8.5.3.2.2 In the absence of detailed subsoil investigations and pile load tests for minor and less important structures, a rough estimate of safe load on piles may be made from the safe load table given in the good practice [C-2(48)].

NOTE — Safe loads as given in the above referred Table are symptomatic. Safe load carrying capacity of pile shall be worked out for the actual geotechnical data using 6.2.3.1 of good practice [C-2(48)], subjected to confirmation by initial pile load test in accordance with good practice [C-2(39)] and other provisions in 6.2.3.2 of good practice [C-2(48)].

8.5.3.3 *Negative skin friction or dragdown force*

Provisions of **8.3.4** shall generally apply subject to the condition that the under-reamed bulb is provided below the strata susceptible to negative skin friction.

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8.5.3.4 *Structural capacity*

Provisions of **8.3.5** shall generally apply except that the under-reamed pile stem is designed for axial capacity as a short column. Under-reamed piles under lateral loads and moments tend to behave more as rigid piles due to the presence of bulbs and therefore the analysis can be done on rigid pile basis. Nominally reinforced long single bulb piles which are not rigid may be analyzed as per the method given in Annex G or as per other accepted methods.

8.5.3.5 *Spacing*

Generally, the centre to centre spacing for bored cast *in-situ* under-reamed piles in a group should be two times the bulb diameter ($2D_u$). It shall not be less than 1.5 D_u. For under-grade beams, the maximum spacing of piles should generally not exceed 3 m. In under-reamed compaction piles, generally the spacing should not be less than 1.5 D_u. If the adjacent piles are of different diameter, the value of larger bulb diameter shall be considered for spacing.

8.5.3.6 *Pile groups*

In order to determine the load-carrying capacity of a group of piles, a number of efficiency equations are in use. However, it is difficult to establish the accuracy of these efficiency equations as the behaviour of pile group is dependent on many complex factors. It is desirable to consider each case separately on its own merits.

The load-carrying capacity of a pile group may be equal to or less than the load-carrying capacity of individual piles multiplied by the number of piles in the group.

In load bearing walls, piles should generally be provided under all wall junctions to avoid point loads on beams.

8.5.3.7 *Transient and overloading*

Provisions of **8.2.3.9** and **8.2.3.10** shall generally apply.

8.5.3.8 *Reinforcement*

8.5.3.8.1 The minimum area of longitudinal reinforcement shall be 0.4 percent of the cross-sectional area of the pile shaft and shall be provided throughout the length.

8.5.3.8.2 The main reinforcement of the pile shall extend above cut-off level for at least the development length to be embedded into the pile cap/grade beam.

8.5.3.8.3 The minimum diameter of the rings shall be 8 mm and the spacing of the rings shall be not less than 150 mm. Stiffener rings of suitable diameter and spacing should be provided in the reinforcing cage to impart adequate rigidity to the reinforcing cage during handling and installation. Stiffener rings preferably of 12 mm diameter at every 1.5 m centre to centre spacing should be provided along the entire length of the cage. Minimum 6 numbers of vertical bars shall be used for a circular pile and minimum diameter of vertical bar shall be 12 mm. The minimum clear horizontal spacing between the adjacent vertical bars shall be four times the maximum aggregate size in concrete. If required, the bars can be bundled to maintain such spacing.

8.5.3.8.4 Clear cover to all reinforcement including rings in pile shaft shall be not less than 50 mm, which shall be increased to 75 mm in case of aggressive environment of sulphates, etc. In addition, the clear cover of the main reinforcement from the pile tip shall be 75 mm to 100 mm.

8.5.3.9 The design of pile cap and grade beams shall conform to the requirements specified in **8.2.3.12** and **8.2.3.13** respectively.

8.5.4 For detailed information on under-reamed piles regarding control of pile, installation, reference shall be made to good practice [C-2(46)].

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8.5.5 *Non-Destructive Testing*

For quality assurance of concrete piles, non-destructive integrity test may be carried out prior to laying of beam/caps, in accordance with accepted standard [C-2(44)].

8.6 Timber Piles

8.6.1 *General*

Piles find application in foundations to transfer loads from a structure to competent sub-surface strata having adequate load-bearing capacity. The load transfer mechanism from a pile to the surrounding ground is complicated and is not yet fully understood, although application of pile foundations is in practice over many decades. Broadly, piles transfer axial loads either substantially by friction along its shaft and/or by the end bearing. Piles are used where either of the above load transfer mechanism is possible depending upon the subsoil stratification at a particular site. Construction of pile foundation requires a careful choice of piling system depending upon the subsoil conditions, the load characteristics of the structure and the limitations of total settlement, differential settlement and any other special requirement of a project. The installation of piles demands careful control on position, alignment and depth and involves specialized skill and experience.

Timber piles, currently do not find extensive use in civil engineering works, however, these piles have been used for compaction of soils, for supporting and protecting water-front structures and also as pile foundations. The choice for using a timber pile is mainly governed by the availability of good quality structural timber, site conditions, particularly the water-table conditions. Timber piles, however, should be avoided because of two reasons, (a) to preserve the environment and green vegetative cover and (b) at most of the places in India, water table has gone down substantially which implies that the timber pile will be only partially under water and this condition is highly undesirable for durability of timber.

Use of treated or untreated piles will depend upon the site conditions and upon whether the work is of permanent or of temporary nature. The timber pile installed should have its entire length embedded under water so that the pile may not get deteriorated. They have the advantages of being comparatively light for their strength and are easily handled. However, they will not withstand as hard driving as steel or concrete piles. Timber has to be selected carefully and treated where necessary for use as piles, as the durability and performance would considerably depend upon the quality of the material and relative freedom from natural defects. If the piles are perennially under salt water condition, treatment of timber is not necessary since by immersion in salt water the life of the pile will be very long. Timber piles are generally used for single and two storeyed structures in back water area where bearing strata is available at a depth less than 12 m. Coconut/Palmyra tree trunks have also been used as timber piles in coastal areas.

8.6.2 *Materials*

8.6.2.1 *Timber*

The timber shall have the following characteristics:

a) Only structural timber shall be used for piles (*see* Part C 'Structural Design/Section 3 Timber and Bamboo, 3(A) Timber' of this NBCS);

b) The length of an individual pile shall be,

   1) the specified length ± 300 mm for piles up to and including 12 m in length; and

   2) the specified length ± 600 mm for piles above 12 m in length.

c) The ratio of heartwood diameter to the pile butt diameter shall be not less than 0.8; and

d) Piles to be used untreated shall have as little sapwood as possible.

8.6.3 *Design Considerations*

8.6.3.1 *General*

Pile foundations shall be designed in such a way that the load from the structure can be transmitted to the sub-surface with adequate factor of safety against shear failure of sub-surface and without causing such settlement,

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(differential or total), which may result in structural damage and/or functional distress under permanent/transient loading. The pile shaft should have adequate structural capacity to withstand all loads (vertical, lateral or otherwise) which are to be transmitted to the subsoil.

8.6.3.2 *Adjacent structures*

When working near existing structures, care shall be taken to avoid damage to such structures. In case of deep excavations adjacent to piles, proper shoring or other suitable arrangement shall be made to guard against undesired lateral movement of soil. Good practice [C-2(39)] may be used as a guide for studying qualitatively the effect of vibration on persons and structures.

8.6.3.3 *Pile capacity*

See **8.2.3.3**.

8.6.3.4 *Structural capacity*

8.6.3.4.1 The pile shall have necessary structural strength to transmit the loads imposed on it, ultimately to the soil. The structural capacity of the timber piles shall be based on the permissible compressive stress of the timber species.

8.6.3.4.2 The length of pile projected into the pile cap is critical for ensuring lateral load transfer and pull-out resistance. Effective transfer of lateral load may be possible with 100 mm to 200 mm projection into the pile cap concrete depending on the safe lateral load capacity. However, pull-out resistance that can be developed by such small penetration may be limited and the use of timber piles for resisting pull-out resistance may not be viable.

8.6.3.4.3 It is preferable to conduct an initial load test as given in good practice [C-2(40)] to assess the axial capacity, lateral capacity and pull-out capacity of piles. For compaction piles, tests should be done on a group of piles with the cap resting on the ground. The acceptance criteria shall also be as per the provisions given in good practice [C-2(40)]

8.6.3.5 For detailed information on timber piles regarding spacing, classification, control of pile driving, storing and handling, reference shall be made to good practice [C-2(49)].

9 DESIGN AND CONSTRUCTION OF COMBINED PILED RAFT FOUNDATIONS

9.1 General

This clause covers the design and construction of combined piled raft foundation (CPRF) for residential, commercial and industrial structures, tall structures, store houses, storage tanks, etc. The CPRF transmits the load to soil by resistance developed by the components of the foundation, that is, piles and raft, the raft being placed on competent ground. The piles develop resistance along the pile shaft by skin friction and at the pile tip by end bearing. The raft develops the resistance by generating the contact pressure below the raft.

CPRF shall not be used in cases where soil layers of relatively low stiffness (For example, very soft cohesive soil, organic soil, collapsible soil, liquefiable soil) are situated closely beneath the raft. Provisions of this standard shall not be applicable to layered soil with significantly high stiffness (modulus) contrast between the top and bottom layers.

9.2 Necessary Information

For satisfactory design and construction of a combined pile-raft foundation, the following information is necessary:

a) Details of site plan showing location of proposed and adjacent structures;

b) Details of plan and vertical cross-section of the proposed building with position of beam and columns;

c) Different load combinations and their intensity indicating design loads, preferably shown in schematic plan;

d) All the loads due to seismic, wind and water current, etc indicated separately;

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e) Limiting values of performance parameters, such as allowable bearing pressure, allowable total settlement, allowable differential settlement, allowable angular distortion, that the foundation and superstructure can withstand;

f) Information on geological history, seismicity, seasonal variation of ground water, climatic factors, for example, sudden flooding, erosion, etc;

g) The design and construction of CPRF requires rigorous geotechnical investigations by using both field and laboratory techniques. The proper dimensioning of CPRF will purely depend up on the outcome of geotechnical investigations. The data obtained from the both the investigations, that is, preliminary investigation for feasibility study and detailed investigation for analyses and design shall be analysed by geotechnical experts;

h) *Geotechnical information* — Sub-surface profile with stratification details, engineering and index properties of founding media to be in accordance with good practice [C-2(8)] and [C-2(50)] and other relevant standards which are essential for the design of CPRF system. Any supplementary investigations should be carried out to obtain additional geotechnical information for design of CPRF;

j) *Extent of geotechnical investigations* — The structure and properties of the substrata and the groundwater conditions shall be known in sufficient detail for any piled-raft construction project. This is necessary to reliably assess the stability and serviceability of the pile and piled-raft foundations and of the overall structure as required by relevant Indian Standards and to assess the effects of foundations on their surroundings. The geotechnical investigations shall extend to sufficient depth below pile tip to record all ground formations and strata influencing the structure and its execution and to identify the load-bearing and deformation properties of the ground. The geotechnical investigations shall be in accordance with good practice [C-2(8)] and [C-2(50)];

k) The geotechnical investigation report and the geotechnical design report, respectively, shall contain all relevant data that can affect CPRF capacity, choice of the execution method and the pile installation. Design soil parameters shall be derived for the CPRF system. The ground information and the parameters related not only to pile capacity, but also to drilling ability, drivability, etc shall be provided;

m) The results of field test on single piles (both ultimate capacity and load- displacement curve). Static test as initial test can be used for design of CPRF, whereas dynamic test is not essential for CPRF design. Dynamic test may be used for additional check, if necessary, on later stage, but not directly related to CPRF design;

n) Dynamic properties of founding media to be taken from geotechnical investigation report; and

p) A review of performance of CPRF in similar locality or ground conditions, if available.

9.3 Design Considerations for Static Load

9.3.1 *General Design Considerations*

9.3.1.1 The design of CPRF should consider the following criteria:

a) Fluctuation in the water table condition and long-term stability of the bearing strata;

b) Consideration for handling sensitive clays and loose bearing soils;

c) Effect of soil excavations;

d) The soil stratigraphy shall be considered when calculating pile responses in the CPRF system;

e) Differential movement of the foundations between the old and new structures;

f) Wherever possible, the centre area of the foundation should be located directly beneath the centre of gravity of the imposed load or else effect of eccentricity should be considered;

g) The load carrying capacity of the combined system subjected to vertical, lateral and moment loadings, total and differential settlement; and

h) Some of the key design considerations are:

   1) Load carrying capacity of the combined system at the desired settlement level for vertical, lateral and moment loadings;

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2) Maximum settlement;

3) Differential settlement;

4) Raft moments and shears for the structural design of the raft; and

5) Pile loads and moments, for the structural design of the piles.

9.3.2 *Stages of CPRF Design*

9.3.2.1 The three main stages of CPRF design are:

a) *Preliminary Stage* (*Feasibility Study*) — Assessment of the performance of raft without piles that includes estimation of vertical and lateral bearing capacity and the settlements. Assessment of the feasibility of using a pile raft and the required number of piles to satisfy the design requirement;

b) *General Pile Characteristics* (*Preliminary Design Stage*) — Pile should be designed for the combined resistances; and

c) *Final Detailed Design* — Final detailed design to obtain the number of piles, location and configuration, settlement, bending moment and shear forces in raft and the pile loads and moments.

9.3.3 *Design Steps*

Design philosophy is illustrated in Fig. 10 and the subsequent design steps to be followed are given in **9.3.3.1** to **9.3.3.5**, **9.3.4** and **9.3.5**.

9.3.3.1 The static pile formula based on the ground parameters of good practice [C-2(51)] can be used for estimation of ultimate load capacity of piles. The provision made in good practice [C-2(52)] as applicable should be followed for the design and construction of piles unless otherwise stated. The procedure for pile load testing should be as per good practice [C-2(40)]. Information obtained (load-settlement response) from pile load test shall be interpreted by geotechnical engineer to arrive at the allowable load with reference to a particular settlement. Pile loads and moment for the structural design of the piles should be considered.

9.3.3.2 The stability and capacity of raft based on the ground parameters as per good practice [C-2(34)] and [C-2(53)] can be estimated. The provision made in good practice [C-2(53)] should be followed for the design and construction of raft foundation unless otherwise stated. For estimation of raft settlement, the guidance given in good practice [C-2(54)] may be used. However, the calculation of settlement for entire CPRF system will be different as described in subsequent section of this standard. Raft moment and shear forces for structural design of raft should be considered.

9.3.3.3 The following are the design philosophies:

a) *Conventional Approach* — Piles are designed as a group while making some allowances for contribution of the raft, primarily to load carrying capacity;

b) *Settlement Control Approach* — Piles are placed strategically below the raft in order to reduce the total and differential settlement. However differential settlement reduction is more important than to reduce the overall average settlement; and

c) *Creep Piling Approach* — Piles are designed to operate at a working load at which significant creep starts to occur at some fraction, typically 70 percent to 80 percent, of the ultimate load capacity.

9.3.3.3.1 For CPRF, the design philosophy is generally governed by settlement control approach to meet the serviceability criteria.

9.3.3.4 The vertical load carrying capacity (Q_{CPRF}) of CPRF should be calculated as per Annex K.

9.3.3.5 Dimensionless factors for CPRF design with practical range of parameters are given in Table 6. This guideline should ensure the complete behavioural mechanism of CPRF system.

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Table 6 Dimensionless Factors for Combined Piled Raft Foundations

(*Clause* 9.3.3.5)

Sl No.	Dimensionless Factors	Definition	Practical Range
(1)	(2)	(3)	(4)
i)	Pile slenderness ratio	L_p/D_p	10 to 100
ii)	Pile spacing ratio	S_p/D_p	2.5 to 8
iii)	Pile-soil stiffness ratio	$K_{ps} = E_p/E_s$	100 to 10 000
iv)	Raft plan aspect ratio	L_r/B_r	1 to 10
v)	Raft-soil stiffness ratio	$K_{rs} = \frac{4E_r B_r t_r^3(1-\vartheta_s^2)}{3\pi E_s L_r^4(1-\vartheta_r^2)}$	0.001 to 10

where

B_r = width of the raft;

D_p = diameter of the pile;

E_p = elastic modulus of pile material;

E_r = elastic modulus of raft material;

E_s = elastic modulus of soil;

L_p = length of the pile;

L_r = length of the raft;

S_p = spacing between the piles;

t_r = thickness of raft;

ϑ_r = poisson's ratio of raft material; and

ϑ_s = poisson's ratio of soil.

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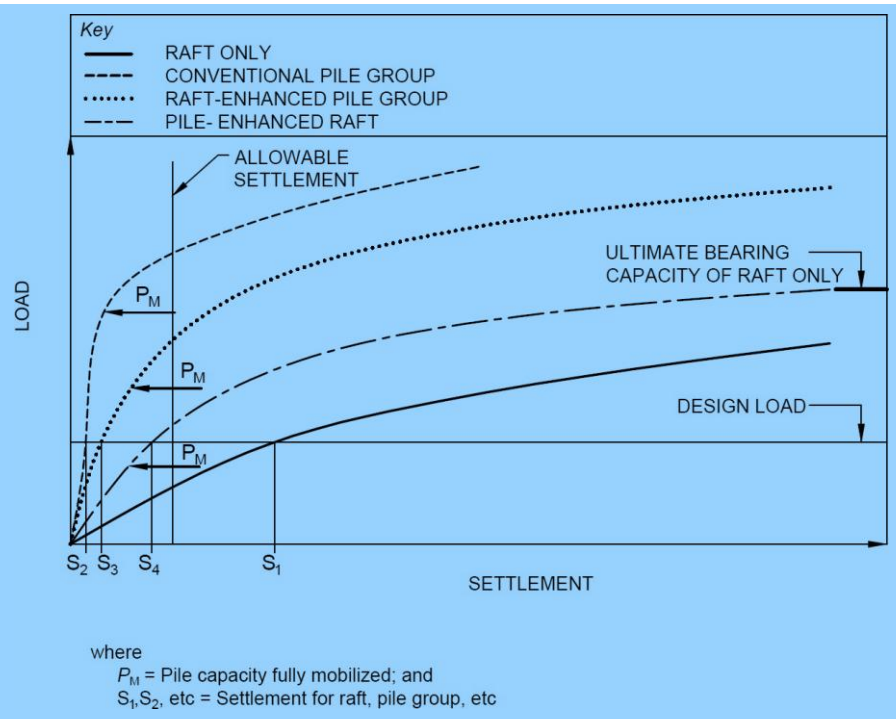

where
P_M = Pile capacity fully mobilized; and
S_1,S_2, etc = Settlement for raft, pile group, etc

FIG. 10(A) LOAD-SETTLEMENT BEHAVIOUR OF RAFTS, CONVENTIONAL PILE GROUPS AND DIFFERENT TYPES OF CPRF

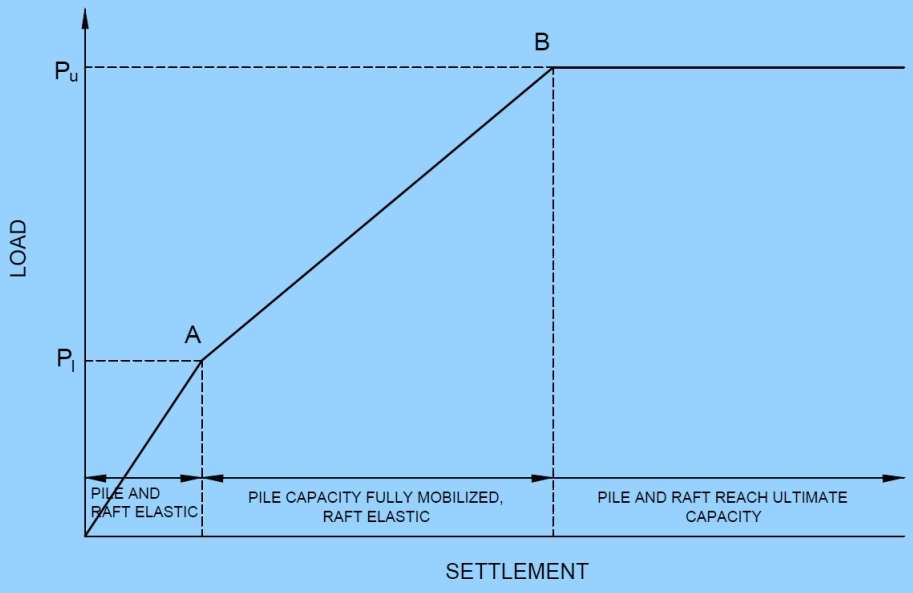

FIG.10(B) SIMPLIFIED LOAD SETTLEMENT CURVE OF A CPRF

FIG. 10 LOAD-SETTLEMENT BEHAVIOUR OF A CPRF

9.3.4 *Computational Model*

Analysis of CPRF shall be performed using computational model which will be able to simulate the appropriate behaviour of single pile. Later from the actual field test results on single pile, the computational model needs to be revised. The computational model shall capture all the interactions as mentioned in Fig. 2(A). Also, the superstructure effect shall be considered appropriately as shown in Fig. 11(A). The two features of estimating the stiffness of soil-pile-raft system are through estimation of pile spring stiffness and raft-soil spring stiffness. The iterative process of convergence of pile and raft response (if implicit) will only come after that. Fig. 11 presents

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the design model of CPRF system considering soil-pile-raft stiffness. Different values of pile stiffness can occur within a group of piles, depending on their position and on the relative stiffness of the raft.

9.3.4.1 *Estimation of pile spring stiffness*

Stiffness of spring representing the pile shall be estimated by dividing the load carried by pile top element in respective direction with the corresponding top displacement of the pile.

$$\text{Pile spring stiffness, } K_p = \frac{\text{Normal contact force at nodes}}{\text{Nodal displacement}}$$

9.3.4.2 *Estimation of raft-soil spring stiffness*

The soil spring stiffness shall be estimated from the load deformation behaviour of the founding medium as given below:

$$\text{Raft-soil Spring stiffness, } K_r = \frac{\text{Normal contact force at nodes}}{\text{Nodal displacement}}$$

9.3.5 *Allowable Settlement for Static Design Consideration*

For CPRF system, total settlement under gravity load should be restricted within 125 mm and the maximum angular distortion of raft as 1/500. There are four angular distortions (θ_{IL}, θ_{IIL}, θ_{IB} and θ_{IIB}) (Fig. 1) and maximum of these four angular distortions will govern CPRF design for Serviceability Limit State (SLS). The total settlement under gravity load may be taken only as a guide and the permissible total settlement should be decided as per serviceability criteria to fulfil the load sharing mechanism of CPRF system.

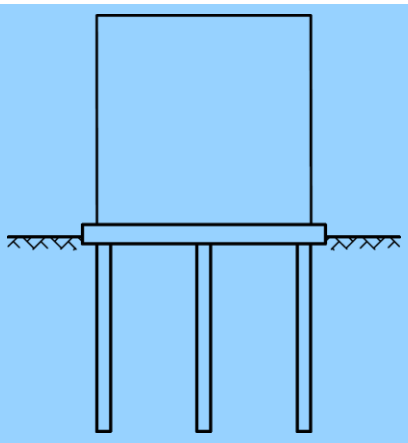

FIG. 11(A) RAFT–PILE–SOIL IN A CPRF

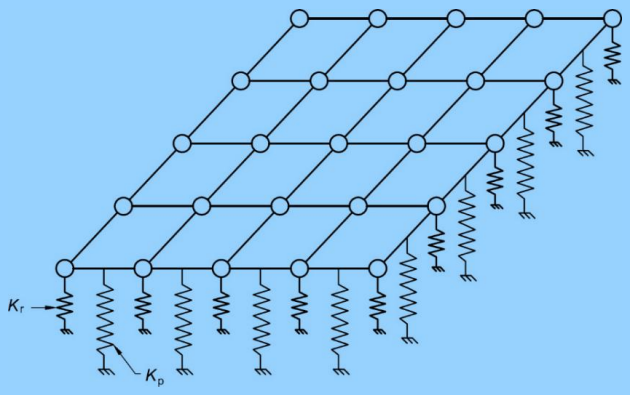

FIG.11(B) PILE AND RAFT-SOIL SPRINGS

FIG. 11 COMBINED ACTION OF SOIL-PILE SYSTEM (TYPICAL DESIGN MODEL)

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9.4 Proof of Design and Construction of a CPRF

The examination of the design and the construction of a CPRF with respect to the geotechnical engineering aspects should be controlled by a geotechnical expert particularly qualified on this subject. This includes the construction for piles and deciding the foundation level. The protocols of the acceptance procedure and the measured values have to be included into the examination. The following aspects shall be considered:

a) Critical review of the soil investigation report (field and laboratory tests), including the selection of input parameters;

b) Evaluation of the computational model used for the design of the CPRF and the computation results by using independent comparative calculations. Apart from the independent comparative calculations, additional quality assurance measures in design should be ensured;

c) Assessment of the effects on the adjacent structures; and

d) Examination of the field monitoring (measuring) program within the construction process of the CPRF.

9.5 Monitoring of a CPRF

The load-settlement behaviour and the load transfer within a CPRF should be monitored for further use by a geotechnical expert. The monitoring comprises of geotechnical and geodetic measurements at the new building and also at the adjacent buildings for settlement. The monitoring of the contact pressure at the level of raft, axial load in selected piles, settlement at the level of raft, ground water table and pore-pressure may be adopted for important projects. The monitoring of a CPRF is an elementary and indispensable component of the safety concept and should be used for the following purposes:

a) Observational method should be followed for monitoring of the response of CPRF both during and post construction phases for projects of high importance;

b) Verification of the computational model and the computational approaches;

c) Detection of possible critical conditions in CPRF system;

d) Examination of the quality assurance during the construction and post construction process; and

e) Examination of distribution of loads may be implemented on piles in centre and edges to capture soil-pile-raft interactions.

The monitoring program should be designed by a geotechnical expert during the design phase depending on importance of the project, complexity of the ground condition and long-term performance. Summary of desired instrumentations and their locations should be highlighted in plan as well as sectional view of the foundation layout.

9.6 Structural Design

The general provision for load, shrinkage, creep and temperature effects and provision of design, reinforcement and detailing shall conform to good practice [C-2(47)] and other relevant standards.

9.7 Seismic Design

Design of the CPRF system considering the seismicity of the ground and entire structure and sub-structural system should be handled by experts to ensure conformity to various relevant standards. Both inertial and kinematic effects should be considered in the seismic design.

10 OTHER FOUNDATIONS, SUBSTRUCTURES AND FOUNDATIONS FOR SPECIAL STRUCTURES

10.1 Pier Foundations

10.1.1 *Design Considerations*

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10.1.1.1 *General*

The design of concrete piers shall conform to the requirements for columns specified in Part C 'Structural Design/ Section 5 Structural Concrete' of this NBCS. If the bottom of the pier is to be belled so as to increase its load carrying capacity, such bell shall be at least 300 mm thick at its edge. The sides shall slope at an angle of not less than 60° with the horizontal. The least permissible dimensions shall be 600 mm, irrespective of the pier being circular, square or rectangular. Piers of smaller dimensions if permitted shall be designed as piles (*see* **8**).

10.1.1.2 *Plain concrete piers*

The height of the pier shall not exceed 6 times the least lateral dimension. When the height exceeds 6 times the least lateral dimension, buckling effect shall be taken into account, but in no case shall the height exceed 12 times the least lateral dimension.

When the height exceeds 6 times the least lateral dimension, the deduction in allowable stress shall be given by the following formula:

$$f_c' = f_c \ (1.3 - \frac{H}{20D})$$

where

f_c' = reduced allowable stress;

f_c = allowable stress;

D = least lateral dimension; and

H = height of pier.

NOTE — The above provision shall not apply for piers where the least lateral dimension is 1.8 m or greater.

10.1.1.3 *Reinforced concrete piers*

When the height of the pier exceeds 18 times its least dimension, the maximum load shall not exceed:

$$P' = P\,(1.5 - \frac{H}{36D})$$

where

D = least lateral dimension;

H = height of the pier measured from top of bell, if any, to the level of cut-off of pier;

P' = permissible load; and

P = permissible load when calculated as axially loaded short column.

10.2 Design and construction of machine foundations, diaphragm walls, etc, shall be carried out in accordance with good practice [C-2(55)].

11 GROUND IMPROVEMENT

11.1 In poor and weak subsoils, the design of conventional foundation for structures and equipment may present problems with respect to both sizing of foundation as well as control of foundation settlements. A viable alternative in certain situations, is to improve the subsoil to an extent such that the subsoil would develop an adequate bearing capacity and foundations constructed after subsoil improvement would have resultant settlements within acceptance limits. This method/technique is called ground improvement which is used to improve *in-situ* soil characteristics by improving its engineering performance as per the project requirement by altering its natural state, instead of having to alter the design in response to the existing ground limitations. The improvement is in terms of increase in bearing capacity, shear strength, reducing settlement and enhancing drainage facility, mitigating liquefaction potential, etc, of soil, as also in improving lateral capacity in case of deep foundations.

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11.2 For provisions with regard to necessary data to be collected to establish the need for ground improvement at a site; considerations for establishing need for ground improvement methods; selection of ground improvement techniques; equipment and accessories for ground improvement; control of ground improvement works and recording of data, reference shall be made to accepted standard [C-2(56)]. Annex L presents various methods of ground improvement along with principles, applicability to various soil conditions, material requirements, equipment required, results likely to be achieved and limitations. This table may be referred to as guidance for selecting the proper method for a situation. *See also* accepted standard [C-2(56)] for details.

11.2.1 For provisions relating to ground improvement by reinforcing the ground using stone columns so as to meet the twin objective of increasing the bearing capacity with simultaneous reduction of settlements, reference shall be made to accepted standard [C-2(57)].

11.2.2 Whenever soft cohesive soil strata underlying a structure are unable to meet the basic requirements of safe bearing capacity and tolerable settlement, ground improvement is adopted to make it suitable for supporting the proposed structure. Both the design requirements that is shear strength and settlement under loading, can be fulfilled by consolidating the soil by applying a preload, if necessary, before the construction of the foundation. This consolidation of soil is normally accelerated with the use of vertical drains. For provisions relating to ground improvement by preconsolidation using vertical drains, reference shall be made to accepted standard [C-2(58)].

11.2.3 Use of suitable geo-synthetics/geo-textiles may be made in an approved manner for ground improvement, where applicable; *see also* accepted standards [C-2(59)].

12 GEOTECHNICAL INSTRUMENTATION

12.1 Geotechnical instrumentation is essential for monitoring the behaviour of soil, rock and structures in civil engineering projects, ensuring safety and design performance. A variety of instruments are deployed at specific locations, where they can provide critical data for various purposes. The data gathered from these instruments allows engineers to assess risks, verify design assumptions and implement corrective actions if necessary.

The typical list of geotechnical instruments, but not limited to these, along with their objectives, is provided in Table 7.

Table 7 Typical List of Geotechnical Instrumentation
(*Clause* 12.1)

Sl No.	Instrumentation	Objective	Applicable for
(1)	(2)	(3)	(4)
i)	Inclinometers	Measurement of lateral deflections	Deflection monitoring of deep excavations, shoring systems, landslide/slope movements, lateral load test on piles
ii)	Piezometers	Measurement of underground water pressure (pore water pressure)	To monitor groundwater conditions
iii)	Vibration monitors (Seismographs)	To monitor vibrations due to various construction activities	Monitoring of adjacent critical structures during general construction, demolition, pile driving, blasting etc.
iv)	Strain gauges	To measure strain in structural elements	Strain (force) measurement in pile foundations, struts

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Table 7 (*Concluded*)

Sl No.	Instrumentation	Objective	Applicable for
(1)	(2)	(3)	(4)
v)	Load cells	To measure loads in various structural elements	Load measurement in rock bolts, foundation anchors, struts, pile foundation during load testing
vi)	Pressure pads/Pressure cells	monitoring long-term pressure imposed by soil on the foundation element	For embankments, rafts, retaining walls,
vii)	Extensometer	Deformation measurement	Measurement of deformation of soil, rock, or can be used during testing of pile foundation to measure deformation of piles at various levels
viii)	Permanent settlement markers	Settlement monitoring	Can be used for monitoring of settlement/ deflections of various structures such as raft, retaining system, slopes, etc.
ix)	Crackmeters	Measure displacement	To measure plate displacement during bi-directional static load test (BDSLT)

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ANNEX A

(*Clauses* 7.4.1.11 *and* B-3.3)

DETERMINATION OF MODULUS OF ELASTICITY (E_s) AND POISSONS'S RATIO (μ)

A-1 DETERMINATION OF MODULUS OF ELASTICITY(E_s)

The modulus of elasticity is a function of composition of the soil, its void ratio, stress history and loading rate. In granular soils it is a function of the depth of the strata, while in cohesive soil it is markedly influenced by the moisture content. Due to its great sensitivity to sampling disturbance, accurate evaluation of the modulus in the laboratory is extremely difficult. For general cases, therefore, determination of the modulus may be based on field tests (*see* **A-2**). Where properly equipped laboratory and sampling facility is available, E_s may be determined in the laboratory (*see* **A-3**).

A-2 FIELD DETERMINATION

A-2.1 The value of E_s shall be determined from plate load test in accordance with accepted standard [C-2(9)].

$$E_s = qB\frac{(1-\mu^2)}{s}I_w$$

where

B = least lateral dimension of test plate;

influence ratio;

I_w = 0.82 for a square plate;

1.00 for a circular plate;

q = intensity of contact pressure;

s = settlement; and

μ = poisson's ratio

NOTE — While this procedure may be adequate for light or less important structures under normal conditions, relevant laboratory tests or field tests are essential in the case of unusual soil types and for all heavy and important structures. Plate load test, though useful in obtaining the necessary information about the soil with particular reference to design of foundation has some limitations. The test results reflect only the character of the soil located within a depth of less than twice the width of the bearing plate. Since the foundations are generally larger than the test plates, the settlement and shear resistance will depend on the properties of a much thicker stratum. Moreover, this method does not give the ultimate settlements particularly in case of cohesive soils. Thus, the results of the test are likely to be misleading, if the character of the soil changes at shallow depths, which is not uncommon. A satisfactory load test should, therefore, include adequate soil exploration {*see* good practice [C-2(8)]} with due attention being paid to any weaker stratum below the level of the footing. Another limitation is the concerning of the effect of size of foundation. For clayey soils the bearing capacity (from shear consideration) for a larger foundation is almost the same as that for the smaller test plate. But in dense sandy soils the bearing capacity increases with the size of the foundation. Thus, tests with smaller size plate tend to give conservative values in dense sandy soils. It may, therefore, be necessary to test with plates of at least three sizes and the bearing capacity results extrapolated for the size of the actual foundation (minimum dimensions in the case of rectangular footings).

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A.2.1.1 The average value of E_s shall be based on a number of plate load tests carried out over the area, the number and location of the tests, depending upon the extent and importance of the structure.

A-2.1.2 *Effect of Size*

In granular soils the value of E_s corresponding to the size of the raft shall be determined as follows:

$$E_s = E_p \frac{B_f}{B_p} \left[\frac{B_f + B_p}{2B_f} \right]^2$$

Where B_f, B_p represent sizes of foundation and plate and E_p is the modulus determined by the plate load test.

A-2.2 For stratified deposits or deposits with lenses of different materials, results of plate load test will be unreliable and static cone penetration tests may be carried out to determine E_s.

A-2.2.1 Static cone penetration tests shall be carried out in accordance with accepted standard [C-2(41)]. Several tests shall be carried out at regular depth intervals up to a depth equal to the width of the raft and the results plotted to obtain an average value of E_s.

A-2.2.2 The value of E_s may be determined from the following relationship:

$$E_s = 2\ C_{kd}$$

where C_{kd} = cone resistance, in kN/m^2

A-3 LABORATORY DETERMINATION OF *E*s

A-3.1 The value of E_s shall be determined by conducting triaxial test in the laboratory in accordance with accepted standard [C-2(60)] on samples collected with least disturbances.

A-3.2 In the first phase of the triaxial test, the specimen shall be allowed to consolidate fully under an all-round confining pressure equal to the vertical effective overburden stress for the specimen in the field. In the second phase, after equilibrium has been reached, further drainage shall be prevented and the deviator stress shall be increased from zero value to the magnitude estimated for the field loading condition. The deviator stress shall then be reduced to zero and the cycle of loading shall be repeated.

A-3.3 The value of E_s shall be taken as the tangent modulus at the stress level equal to one-half the maximum deviator stress applied during the second cycle of loading.

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ANNEX B

(*Clauses* 7.4.1.11 *and* C-3)

DETERMINATION OF MODULUS OF SUB-GRADE REACTION

B-1 GENERAL

B-1 The modulus of subgrade reaction (k) as applicable to the case of load through a plate of size 300 mm × 300 mm or beams 300 mm wide on the soils is given in Table 8 for cohesionless soils and in Table 9 for cohesive soils. Unless more specific determination of k is done (*see* **B-2** and **B-3**), these value may be used for design of raft foundation in cases where the depth of the soil affected by the width of the footing may be considered isotropic and the extrapolation of plate load test results is valid.

B-2 FIELD DETERMINATION

B-2.1 In cases where the depth of the soil affected by the width of the footing may be considered as isotropic, the value of k may be determined in accordance with accepted standard [C-2(10)]. The test shall be carried out with a plate of size not less than 300 mm.

B-2.2 The average value of k shall be based on a number of plate load tests carried out over the area, the number and location of the tests depending upon the extent and importance of the structure.

B-3 LABORATORY DETERMINATION

B-3.1 For stratified deposits or deposits with lenses of different materials, evaluation of k from plate load test will be unrealistic and its determination shall be based on laboratory tests {*see* good practice [C-2(60)]}.

B-3.2 In carrying out the test, the continuing cell pressure may be so selected as to be representative of the depth of the average stress influence zone (about 0.5 B to B).

B-3.3 The value of k shall be determined from the following relationship:

$$k = 0.65 \times \sqrt[12]{\left(\frac{E_s B^4}{EI}\right)} \cdot \frac{E_s}{(1-\mu^2)} \cdot \frac{1}{B}$$

where

B = width of the footing;
E = young's modulus of foundation material;
E_s = modulus of elasticity of soil (*see* Annex A);
I = moment of inertia of the foundation; and
μ = poisson's ratio of soil.

B-4 CALCULATIONS

When the structure is rigid (*see* Annex C), the average modulus of subgrade reaction may also be determined as follows:

$$k_s = \frac{\text{Average contact pressure}}{\text{Average settlement of the raft}}$$

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Table 8 Modulus of Subgrade Reaction (*k*) for Cohesionless Soils

(*Clause* B-1)

Sl No.	Soil Characteristic		Modulus of Subgrade Reaction (*k*)[1] kN/m³	
	Relative Density	Standard Penetration Test Value (N) (Blows per 300 mm)	For Dry or Moist State	For Submerged State
(1)	(2)	(3)	(4)	(5)
i)	Loose	< 10	15 000	9 000
ii)	Medium	10 to 30	15 000 to 47 000	9 000 to 29 000
iii)	Dense	30 and over	47 000 to180 000	29 000 to 108 000

Table 9 Modulus of Subgrade Reaction (*k*) for Cohesive Soils

(*Clause* B-1)

Sl No.	Soil Characteristic		Modulus of Subgrade Reaction (*k*)[1] kN/m³
	Consistency	Unconfined Compressive Strength kN/m²	
(1)	(2)	(3)	(4)
i)	Stiff	100 to 200	27 000
ii)	Very stiff	200 to 400	27 000 to 54 000
iii)	Hard	400 and over	54 000 to 108 000

[1] The above values apply to a square plate 300 mm × 300 mm or beams 300 mm wide.

[1] The values apply to a square plate 300 mm × 300 mm. The above values are based on the assumption that the average loading intensity does not exceed half the ultimate bearing capacity.

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ANNEX C

(*Clauses* 7.4.4.1.2 (a), (b), 7.4.4.2 *and* B-4)

RIGIDITY OF SUPERSTRUCTURE AND FOUNDATION

C-1 DETERMINATION OF THE RIGIDITY OF THE STRUCTURE

The flexural rigidity *EI* of the structure of any section may be estimated according to the relation given below (*see also* Fig. 12)

$$EI = \frac{E_1 I_i b^2}{2H^2} + \sum E_2 I_b \left[1 + \frac{(I_u' + I_1')b^2}{(I_b' + I_u' + I_f')l^2}\right]$$

where

b = length or breadth of the structure in the direction of bending, in m;

E_1 = modulus of elasticity of the infilling material (wall material), in kN/m^2;

E_2 = modulus of elasticity of the frame material, in kN/m^2;

H = total height of the infilling, in m;

h_l = length of the lower column, in m;

h_u = length of the upper column, in m;

I_b = moment of inertia of the beam, in m^4;

$I_b' = \frac{I_b}{l}$;

$I_f' = \frac{I_f}{l}$;

I_i = moment of inertia of the infilling, in m^4;

$I_u' = \frac{I_U}{h_U}$;

$I_l' = \frac{I_l}{h_i}$;

I_l = moment of inertia of the lower column, in m^4;

I_f = moment of inertia of the foundation beam or raft in m^4;

I_u = moment of inertia of the upper column, in m^4; and

l = spacing of the columns, in m.

NOTE — The summation is to be done over all the storeys including the foundation beam or raft. In the case of the foundation, I_f' replaces I_b' and I_l becomes zero, whereas for the topmost beam I_u becomes zero.

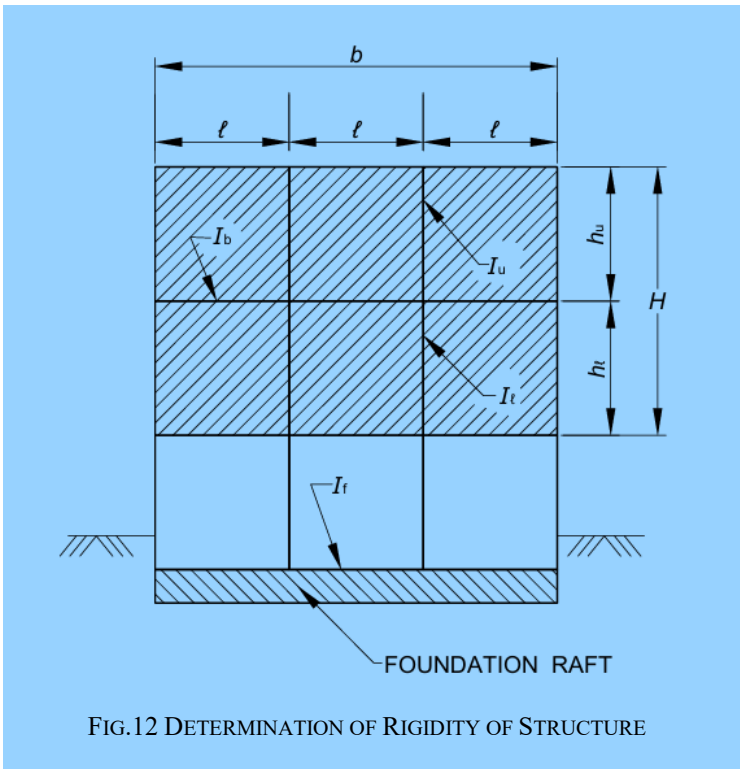

FIG.12 DETERMINATION OF RIGIDITY OF STRUCTURE

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C-2 RELATIVE STIFFNESS FACTOR, *K*

C-2.1 Whether a structure behaves as rigid or flexible depends on the relative stiffness of the structure and the foundation soil. This relation is expressed by the relative stiffness factor *K* given below:

a) For the whole structure, $K = \frac{EI}{E_s b^3 a}$;

b) For rectangular rafts, $K = \frac{E}{12E_s}\left(\frac{d}{b}\right)^3$; and

c) For circular rafts, $K = \frac{E}{12E_s}\left(\frac{d}{2R}\right)^3$

where

a = length perpendicular to the section under investigation, in m;

b = length of the section in the bending axis, in m;

d = thickness of the raft or beam, in m;

EI = flexural rigidity of the structure over the length (a), in kN/m^2;

E_s = modulus of compressibility of the foundation soil, in kN/m^2; and

R = radius of the raft, in m.

C-2.1.1 For $K > 0.5$, the foundation may be considered as rigid [*See* 7.4.4.1(a)].

C-3 DETERMINATION OF CRITICAL COLUMN SPACING

Evaluation of the characteristics λ is made as follows:

$$\lambda = \sqrt[4]{\left(\frac{kB}{4E_c I}\right)}$$

where

B = width of raft B, in m;

E_c = modulus of elasticity of concrete in kN/m^2;

I = moment of inertia of raft, in m^4; and

k = modulus of subgrade reaction, in kN/m^3, for footing of width B, in m (*see* Annex B).

ANNEX D

(*Clause* 7.4.4.1.3)

CALCULATION OF PRESSURE DISTRIBUTION BY CONVENTIONAL METHOD

D-1 DETERMINATION OF PRESSURE DISTRIBUTION

The pressure distribution (q) under the raft shall be determined by the following formula:

$$q = \frac{Q}{A} \pm \frac{Qe_y'}{I_x'} y \pm \frac{Qe_x'}{I_y'} x$$

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where

A = total area of the raft,

e_x', e_y', I_x', I_y' = eccentricities and moments of inertia about the principal axes through the centroid of the section;

x, y = co-ordinates of any given point on the raft with respect to the x and y axes passing through the centroid of the area of the raft; and

Q = total vertical load on the raft.

I_x', I_y', e_x', e_y' may be calculated from the following equations:

$$I_x' = I_x - \frac{I_{xy}^2}{I_y}$$

$$I_y' = I_y - \frac{I_{xy}^2}{I_x}$$

$$e_x' = e_x - \frac{I_{xy}}{I_x} e_y$$

$$e_y' = e_y - \frac{I_{xy}}{I_y} e_x$$

where

e_x, e_y = eccentricities in the x and y directions of the load from the centroid;

I_{xy} = $\int xy \cdot dA$ for the whole area about x and y axes through the centroid; and

I_x, I_y = moment of inertia of the area of the raft respectively about the x and y axes through the centroid.

For a rectangular raft, the equation simplifies to :

$$q = \frac{Q}{A}\left(1 \pm \frac{12e_y y}{b^2} \pm \frac{12e_x x}{a^2}\right)$$

where

a and b = the dimensions of the raft in the x and y directions, respectively.

NOTE — If one or more of the values of (q) are negative as calculated by the above formula, it indicates that the whole area of foundation is not subject to pressure and only a part of the area is in contact with the soil and the above formula will still hold good, provided the appropriate values of I_x, I_y, I_{xy}, e_x and e_y, are used with respect to the area in contact with the soil instead of the whole area.

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ANNEX E

[*Clause* 7.4.4.2(A)]

CONTACT PRESSURE DISTRIBUTION AND MOMENTS BELOW FLEXIBLE FOUNDATION

E-1 CONTACT PRESSURE DISTRIBUTION

E-1.1 The distribution of contact pressure is assumed to be linear with the maximum value attained under the columns and the minimum value at mid span.

E-1.2 The contact pressure for the full width of the strip under an interior column load located at a point *i* can be determined as [*see* Fig. 13(A)]:

$$p_i = \frac{5P_i}{\bar{l}} + \frac{48M_i}{(\bar{l})^2}$$

where

l = average length of adjacent span, in m;

M_i = moment under an interior columns loaded at *I*; and

P_i = column load, in t at point *i*.

E-1.3 The minimum contact pressure for the full width of the strip at the middle of the adjacent spans can be determined as [*see* Fig. 13(A) and Fig. 13(B)]:

$$p_{ml} = 2P_i\left(\frac{l_r}{l_l\bar{l}}\right) - p_i\left(\frac{\bar{l}}{l_l}\right)$$

$$p_{mr} = 2P_i\left(\frac{l_l}{l_r\bar{l}}\right) - p_i\left(\frac{\bar{l}}{l_r}\right)$$

$$p_m = \frac{p_{ml} + p_{mr}}{2}$$

where l_r, l_l are as shown in Fig. 13(A).

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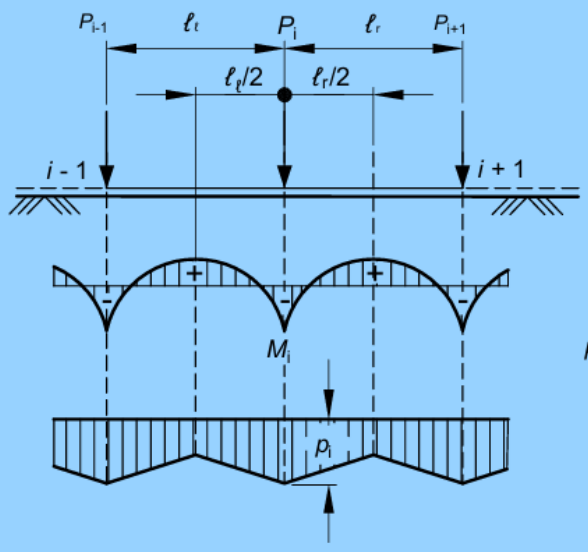

FIG.13(A) MOMENT AND PRESSURE DISTRIBUTION AT INTERIOR COLUMN

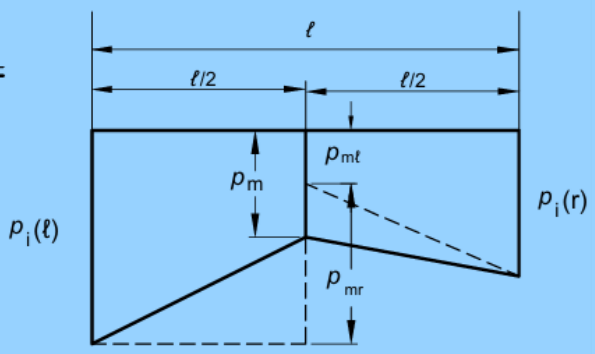

FIG.13(B) PRESSURE DISTRIBUTION OVER AN INTERIOR SPAN

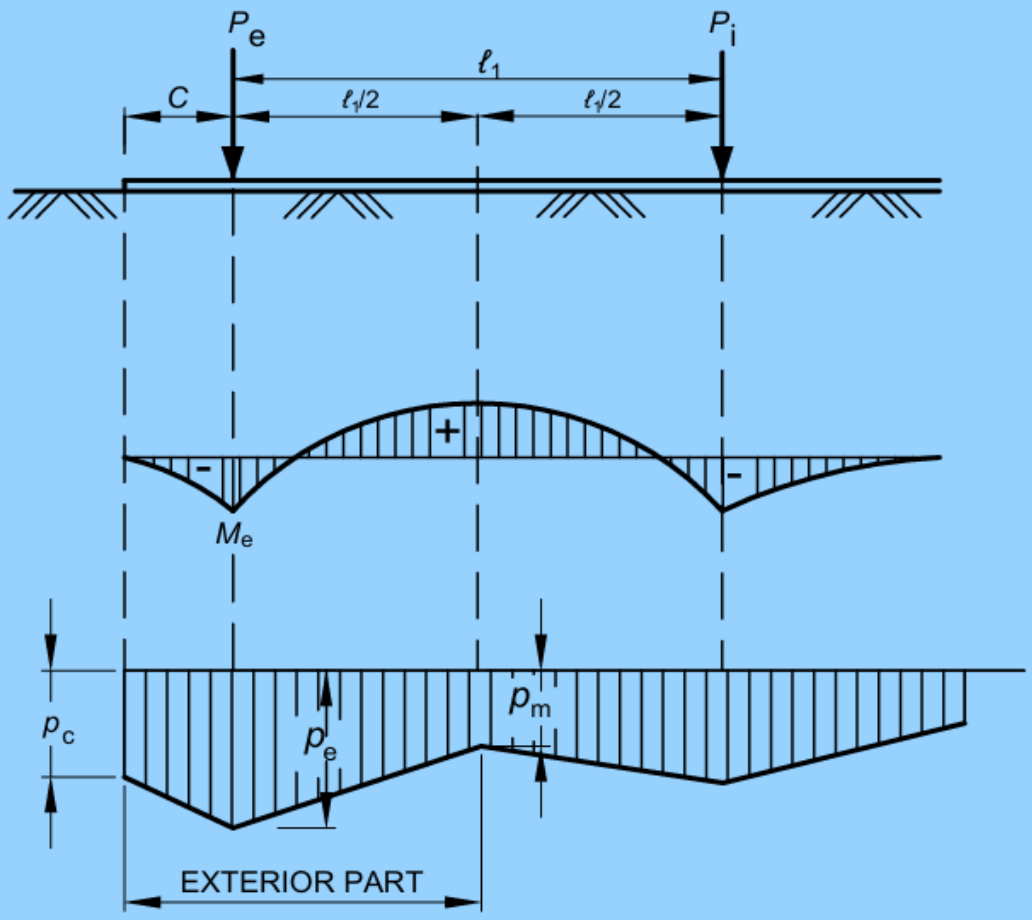

FIG.13(C) MOMENT AND PRESSURE DISTRIBUTION AT EXTERIOR COLUMNS

FIG. 13 MOMENT AND PRESSURE DISTRIBUTION AT COLUMNS

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E-1.4 If E-2.3(a) governs the moment under the exterior columns, contact pressures under the exterior columns and at end of strip can be determined as [*see* Fig. 13(C)]:

$$p_e = \frac{4P_e + \frac{6M_e}{C} - p_m l_1}{C + l_1}$$

$$p_c = -\frac{3M_e}{C^2} - \frac{p_e}{2}$$

where P_e, p_m, M_e, l_1, C are as shown in Fig. 13(C).

E-1.5 If E-2.3(b) governs the moment under the exterior columns, the contact pressures are determined as [*see* Fig. 13(C)] :

$$p_e = p_c \frac{4P_e - p_m l_1}{4C + l_1}$$

E-2 BENDING MOMENT DIAGRAM

E-2.1 The bending moment under an interior column located at *i* [*see* Fig. 11(A)] can be determined as :

$$M_i = -\frac{P_i}{4\lambda}(0.24\lambda\bar{l} + 0.16)$$

E-2.2 The bending moment at mid span is obtained as [*see* Fig. 11(A)]:

$$M_m = M_o + M_i$$

M_i = average of negative moments M_i at each end of the bay

M_o = moment of simply supported beam

= $\frac{l^2}{8}[p_i(l) + 4\overline{p_m} + p_i(r)]$.

where l, $p_i(l)$, $p_i(r)$, $\overline{p_m}$ are as shown in Fig. 11(B).

E-2.3 The bending moment M_e under exterior columns can be determined as the least of [*see* Fig. 13(C)]:

a) $M_{e1} = -\frac{P_e}{4\lambda}(0.13\lambda l_1 + 1.06\lambda C - 0.50)$; and

b) $M_{e2} = -\frac{(4P_e - p_m l_1)}{(4C + l_1)}\frac{C^2}{2}$;

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ANNEX F

[*Clause* 7.4.4.2(b)]

FLEXIBLE FOUNDATION — GENERAL CONDITION

F-1 CLOSED FORM SOLUTION OF ELASTIC PLATE THEORY

F-1.1 For a flexible raft foundation with non-uniform column spacing and load intensity, solution of the differential equation governing the behaviour of plates on elastic foundation (Winkler type) gives radial moment (M_r) tangential moment (M_t) and deflection (w) at any point by the following expressions:

$$M_r = -\frac{P}{4}\left[Z_4\left(\frac{r}{L}\right) - (1-\mu)\frac{Z_3'\left(\frac{r}{L}\right)}{\left(\frac{r}{L}\right)}\right]$$

$$M_t = -\frac{P}{4}\left[\mu Z_4\left(\frac{r}{L}\right) + (1-\mu)\frac{Z_3'\left(\frac{r}{L}\right)}{\left(\frac{r}{L}\right)}\right]$$

$$w = \frac{PL^2}{4D}\cdot Z_3\left(\frac{r}{l}\right)$$

where

L = radius of effective stiffness;

= $\left(\frac{D}{k}\right)^{1/4}$;

P column load; and

r distance of the point under investigation from column load along radius.

where

D = flexural rigidity of the foundation;

k = modulus of subgrade reaction for footing of width B;

P = $\frac{Et^2}{12(1-\mu)^2}$

E = modulus of elasticity of the foundation material;

T = raft thickness; and

μ = possion's ratio of the foundation material.

$Z_3\left(\frac{r}{L}\right)$, $Z_3'\left(\frac{r}{L}\right)$, $Z_4\left(\frac{r}{L}\right)$ = functions of shear, moment and deflection (*see* Fig. 14)

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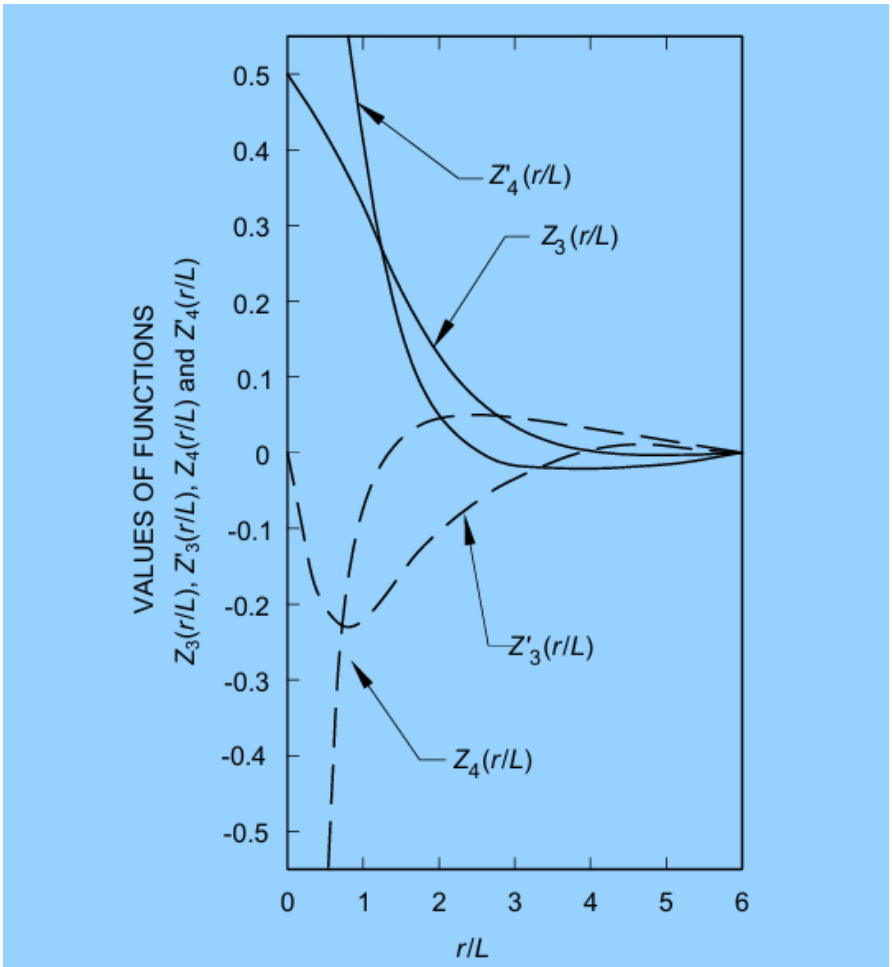

FIG.14 FUNCTIONS FOR SHEAR MOMENT AND DEFLECTION

F-1.2 The radial and tangential moments can be converted to rectangular co-ordinates:

$$M_x = M_r \cos^2 \varphi + M_t \sin^2 \varphi$$
$$M_y = M_r \sin^2 \varphi + M_t \cos^2 \varphi$$

where

Φ = angle with x-axis to the line jointing origin to the point under consideration.

F-1.3 The shear, Q per unit width of raft can be determined by :

$$Q = -\frac{P}{4L} Z_4' \left(\frac{r}{L}\right)$$

$Z_4' \left(\frac{r}{L}\right)$ = function for shear (*see* Fig. 14).

F-1.4 When the edge of the raft is located within the radius of influence, the following corrections are to be applied. Calculate moments and shears perpendicular to the edge of the raft within the radius of influence, assuming the raft to be infinitely large. Then apply opposite and equal moments and shears on the edge of the mat. The method for beams on elastic foundation may be used.

F-1.5 Finally, all moments and shears calculated for each individual column and wall are superimposed to obtain the total moment and shear values.

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ANNEX G

(*Clauses* 8.2.3.3.1.1, 8.2.3.3.1.4 *and* 8.5.3.4)

LOAD CARRYING CAPACITY OF PILES — STATIC ANALYSIS

G-1 PILES IN GRANULAR SOILS

The ultimate load capacity (Q_u) of piles, in kN, in granular soils is given by the following formula:

$$Q_u = A_p(\tfrac{1}{2} D \gamma N_\gamma + P_D N_q) + \sum_{i=1}^{n} Ki\ P_{Di} \tan \delta_i A_{si} \quad \ldots\ldots\ldots(1)$$

The first term gives end bearing resistance and the second term gives skin friction resistance.

where

A_{si} = surface area of pile shaft in the ith layer, in m^2;

A_p = cross-sectional area of pile tip, in m^2;

D = diameter of pile shaft, in m;

N_γ, N_q = bearing capacity factors depending upon the angle of internal friction, ϕ at pile tip;

K_i = coefficient of earth pressure applicable for the ith layer (*see* Note 3);

P_D = effective overburden pressure at pile tip, in kN/m^2 (*see* Note 5);

P_{Di} = effective overburden pressure for the ith layer, in kN/m^2;

$\sum_{i=1}^{n}$ = summation for layers 1 to n in which pile is installed and which contribute to positive skin friction;

δ_i = angle of wall friction between pile and soil for the ith layer; and

γ = effective unit weight of the soil at pile tip, in kN/m^3.

NOTES

1 N_γ factor can be taken for general shear failure according to good practice [C-2(34)].

2 N_q factor will depend on the nature of soil, type of pile, the L/B ratio and its method of construction. The values applicable for driven and bored piles are given in Fig. 15(A) and Fig. 15(B) respectively.

3 K_i, the earth pressure coefficient depends on the nature of soil strata, type of pile, spacing of piles and its method of construction. For bored piles in loose to dense sand with ϕ varying between 30° and 40°, K_i values in the range of 1 to 2 may be used for driven piles and in case of bored piles, K_i values in the range of 1 to 1.5 may be used.

4 δ, the angle of wall friction may be taken equal to the friction angle of the soil around the pile shaft.

5 In working out pile capacity by static formula, the maximum effective overburden at the pile tip should correspond to the critical depth, which may be taken as 15 times the diameter of the pile shaft for $\phi \leq 30°$ and increasing to 20 times for $\phi \geq 40°$.

6 For piles passing through cohesive strata and terminating in a granular stratum, a penetration of at least twice the diameter of the pile shaft should be given into the granular stratum.

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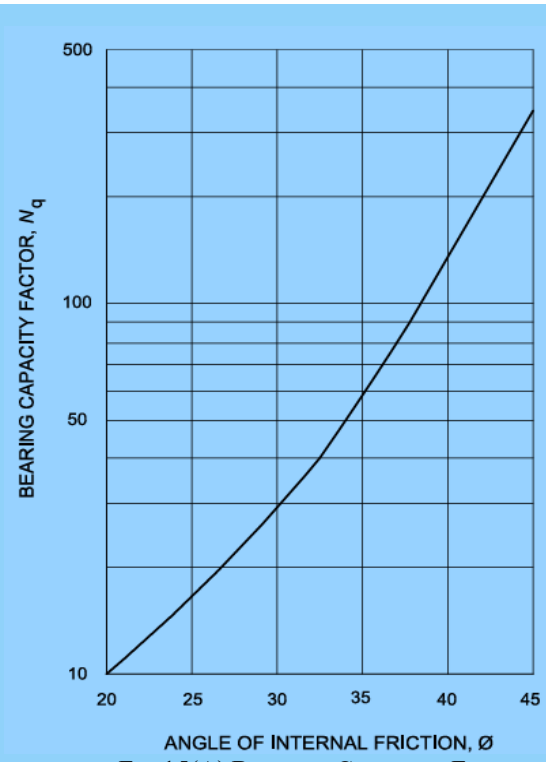

FIG.15(A) BEARING CAPACITY FACTOR, N_Q FOR DRIVEN PILES

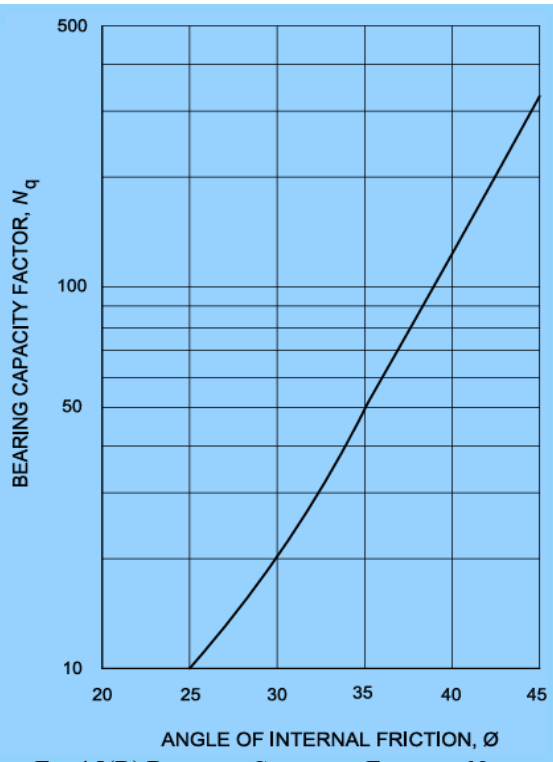

FIG.15(B) BEARING CAPACITY FACTOR, N_Q FOR BORED PILES

FIG. 15 BEARING CAPACITY FACTOR, N_Q

G-2 PILES IN COHESIVE SOILS

The ultimate load capacity (Q_u) of piles, in kN, in cohesive soils is given by the following formula:

$$Q_u = A_p N_c c_p + \sum_{i=1}^{n} \alpha_i c_i A_{si} \qquad \text{.....(2)}$$

The first term gives end bearing resistance and the second term gives the skin friction resistance.

where

α_i = adhesion factor for the *i*th layer depending on the consistency of soil (*see* Note);

A_p = cross-sectional area of pile tip, in m^2;

A_{si} = surface area of pile shaft in the *i*th layer, in m^2;

c_i = average cohesion for the *i*th layer, in kN/m^2;

c_p = average cohesion at pile tip, in kN/m^2;

N_c = bearing capacity factor, may be taken as 9; and

$\sum_{i=1}^{n}$ = summation for layers 1 to *n* in which pile is installed and which contribute to positive skin friction.

NOTE — The value of adhesion factor, α_i depends on the undrained shear strength of the clay and may be obtained from Fig. 16.

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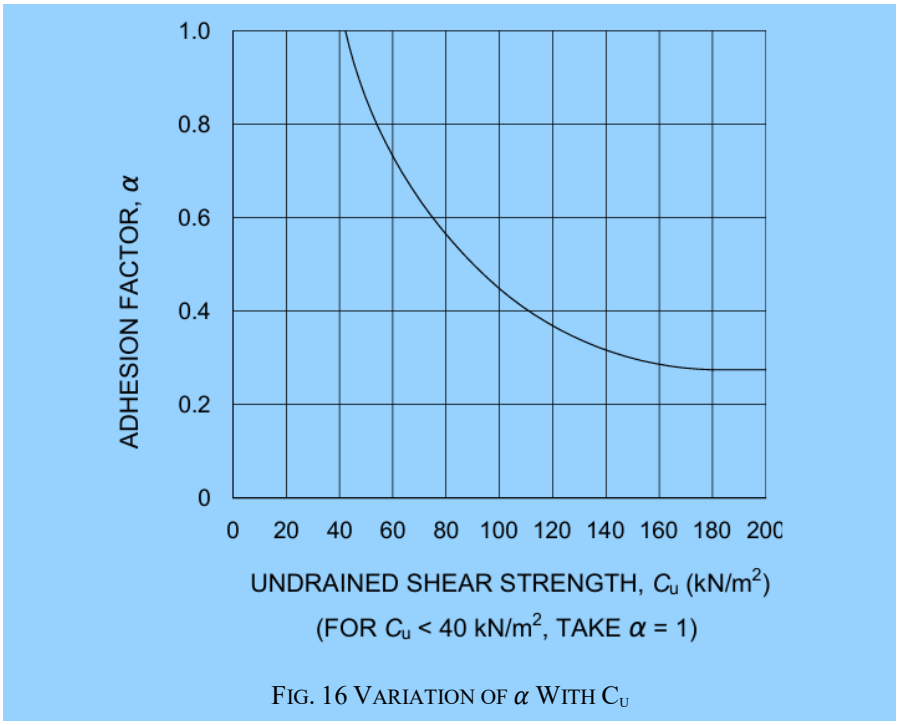

FIG. 16 VARIATION OF α WITH C_U

G-3 USE OF STATIC CONE PENETRATION DATA

G-3.1 When full static cone penetration data are available for the entire depth, the following correlation may be used as a guide for the determination of ultimate load capacity of a pile.

G-3.2 Ultimate end bearing resistance (q_u), in kN/m^2, may be obtained as:

$$q_u = \frac{\frac{q_{c0}+q_{c1}}{2} + q_{c2}}{2}$$

where

D = diameter of pile shaf;.

q_{c0} = average static cone resistance over a depth of $2D$ below the pile tip, in kN/m^2;

q_{c1} = minimum static cone resistance over the same $2D$ below the pile tip, in kN/m^2; and

q_{c2} = average of the envelope of minimum static cone resistance values over the length of pile of $8D$ above the pile tip, in kN/m^2.

G-3.3 Ultimate skin friction resistance can be approximated to local side friction (f_s), in kN/m^2, obtained from static cone resistance as given in Table 10.

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Table 10 Side Friction for Different Soil Types

(*Clause* G-3.3)

Sl No.	Type of Soil	Local Side Friction, f_s kN/m^2
(1)	(2)	(3)
i)	q_c less than 1 000 kN/m^2	$q_c/30 < f_s < q_c/10$
ii)	Clay	$q_c/25 < f_s < 2q_c/25$
iii)	Silty clay and silty sand	$q_c/100 < f_s < q_c/25$
iv)	Sand	$q_c/100 < f_s < q_c/50$
v)	Coarse sand and gravel	$q_c/100 < f_s < q_c/150$

where q_c = cone resistance, in kN/m^2.

G-3.4 The correlation between standard penetration resistance, N (blows/30 cm) and static cone resistance, q_c, in kN/m^2, as given in Table 11 may be used for working out the end bearing resistance and skin friction resistance of piles. This correlation should only be taken as a guide and should preferably be established for a given site as they can vary substantially with the grain size, Atter berg limits, water table, etc.

Table 11 Co-Relation between *N* and q_c for Different Types of Soil

(*Clause* G-3.4)

Sl No.	Type of Soil	q_c/N
(1)	(2)	(3)
i)	Clay	150 to 200
ii)	Silts, sandy silts and slightly cohesive silt-sand mixtures	200 to 250
iii)	Clean fine to medium sand and slightly silty sand	300 to 400
iv)	Coarse sand and sands with little gravel	500 to 600
v)	Sandy gravel and gravel	800 to 1 000

G-4 USE OF STANDARD PENETRATION TEST DATA FOR COHESIONLESS SOIL

G-4.1 The correlation suggested by Meyerhof using standard penetration resistance, N in saturated cohesionless soil to estimate the ultimate load capacity of driven pile is given below. The ultimate load capacity of pile (Q_u), in kN, is given as:

$$Q_u = 40\ N \frac{L_b}{D} A_p + \frac{\overline{N} A_s}{0.50} \quad \text{.....(3)}$$

The first term gives the end bearing resistance and the second term gives the frictional resistance.

where

A_s = surface area of pile shaft, in m^2;

A_p = cross-sectional area of pile tip, in m^2;

D = diameter or minimum width of pile shaft, in m;

L_b = length of penetration of pile in the bearing strata, in m;

$\overline{N}$ = average N along the pile shaft; and

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N = average N values at pile tip;

NOTE — The end bearing resistance should not exceed 400 NA_p

G-4.2 For non-plastic silt or very fine sand the equation has been modified as:

$$Q_u = 30\ N \frac{L_b}{D} A_p + \frac{\overline{N} A_s}{0.60} \qquad (4)$$

The meaning of all terms is same as for equation 3.

G-4.3 The correlation suggested by Meyerhof using standard penetration resistance, N in saturated cohesionless soil to estimate the ultimate load capacity of bored pile is given below. The ultimate capacity of pile (Q_u), in kN, is given as:

$$Q_u = 13\ N \frac{L_b}{D} A_p + \frac{\overline{N} A_s}{0.50} \qquad \text{.....(5)}$$

The first term gives end bearing resistance and the second term gives frictional resistance.

The meaning of all terms is same as for equation 3.

NOTE —The end bearing resistance should not exceed 130 NA_p.

G-4.4 For non-plastic silt or very fine sand the equation has been modified as:

$$Q_u = 10\ N \frac{L_b}{D} A_p + \frac{\overline{N} A_s}{0.60} \qquad \text{.....(6)}$$

The meaning of all terms is same as for equation 3.

G-5 FACTOR OF SAFETY

The minimum factor of safety for arriving at the safe pile capacity from the ultimate capacity obtained by using static formulae shall be 2.5.

G-6 PILES IN STRATIFIED SOIL

In stratified soil/C-ϕ soil, the ultimate load capacity of piles should be determined by calculating the skin friction and end bearing in different strata by using appropriate expressions given in G-1 and G-2.

G-7 PILES IN HARD ROCK

When the crushing strength of the rock is more than characteristic strength of pile concrete, the rock should be deemed as hard rock. Bored piles resting directly on hard rock may be loaded to their safe structural capacity.

G-8 PILES IN WEATHERED / SOFT ROCK

For bored piles founded in weathered/soft rock different empirical approaches are used to arrive at the socket length necessary for utilizing the full structural capacity of the pile.

Since it is difficult to collect cores in weathered/soft rocks, the method suggested by Cole and Stroud using 'N' values is more widely used. The allowable load on the pile, Q_a, in kN, by this approach, is given by:

$$Q_a = c_{u1} N_c \cdot \frac{\pi B^2}{4 F_s} + \alpha\ c_{u2} \cdot \frac{\pi B L}{F_s}$$

where

α = 0.9 (recommended value);

B = minimum width of pile shaft (diameter in case of circular piles),in m;

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c_{u1} = shear strength of rock below the base of the pile, in kN/m^2 (*see* Fig. 17);

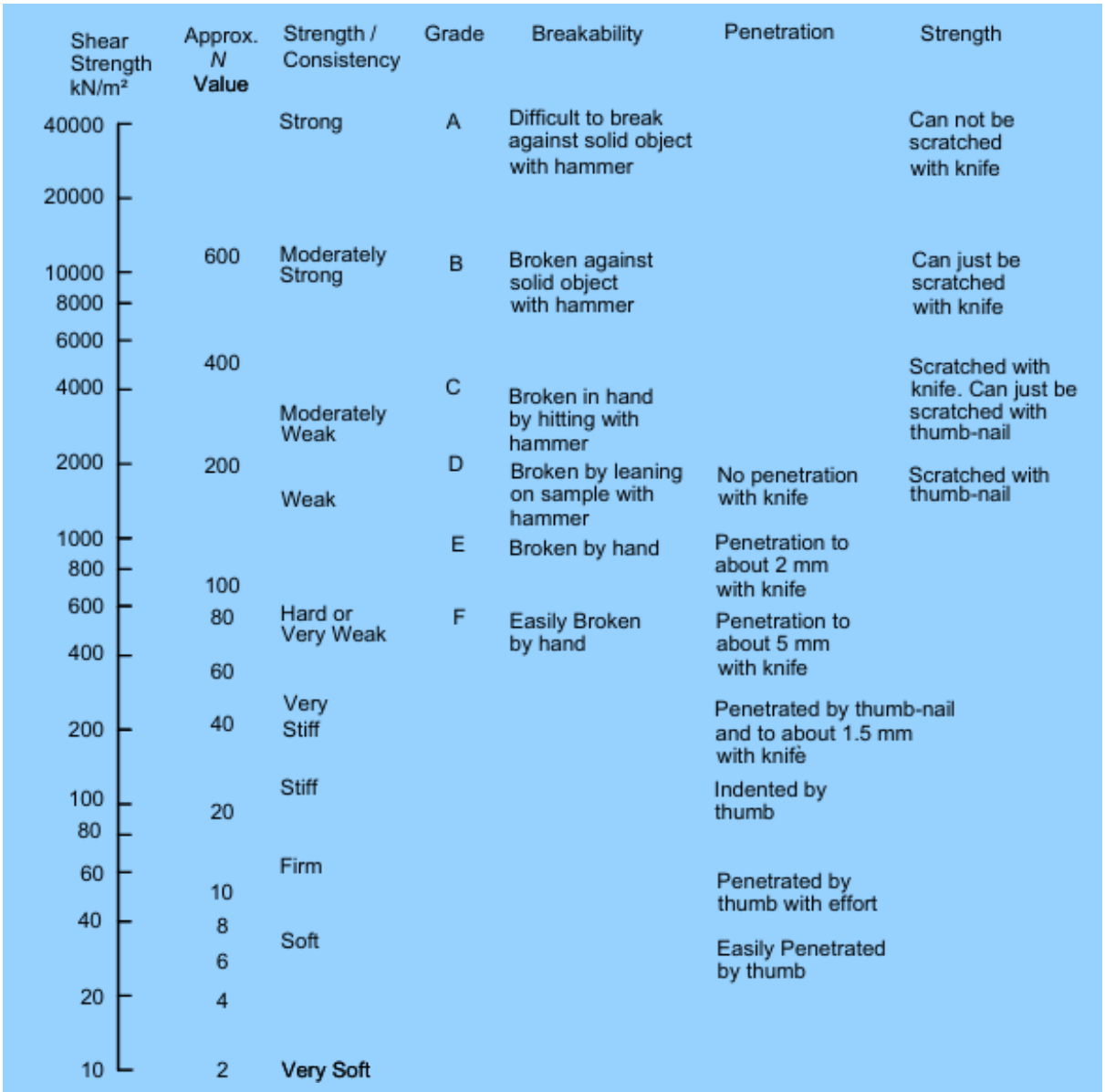
c_{u2} = average shear strength of rock in the socketed length of pile, in kN/m^2 (*see* Fig. 17);

F_s = factor of safety usually taken as 3;

L = socket length of pile, in m; and

N_c = bearing capacity factor taken as 9.

NOTE — For $N \geq 60$, the stratum is to be treated as weathered rock rather than soil.



NOTE – Standard penetration test may not be practicable for N values greater than 200. In such cases, design may be done on the basis of shear strength of rock.

FIG. 17 CONSISTENCY AND SHEAR STRENGTH OF WEATHERED ROCK

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ANNEX H

(*Clause* 8.2.3.5.2)

ANALYSIS OF LATERALLY LOADED PILES

H-1 GENERAL

H-1.1 The ultimate resistance of a vertical pile to a lateral load and the deflection of the pile as the load builds up to its ultimate value are complex matters involving the interaction between a semi-rigid structural element and soil which deforms partly elastically and partly plastically. The failure mechanisms of an infinitely long pile and that of a short rigid pile are different. The failure mechanisms also differ for a restrained and unrestrained pile head condition.

Because of the complexity of the problem only a procedure for an approximate solution that is adequate in most of the cases is presented here. Situations that need a rigorous analysis shall be dealt with accordingly.

H-1.2 The first step is to determine, if the pile will behave as a short rigid unit or as an infinitely long flexible member. This is done by calculating the stiffness factor R or T for the particular combination of pile and soil.

Having calculated the stiffness factor, the criteria for behaviour as a short rigid pile or as a long elastic pile are related to the embedded length L_e of the pile. The depth from the ground surface to the point of virtual fixity is then calculated and used in the conventional elastic analysis for estimating the lateral deflection and bending moment.

H-2 STIFFNESS FACTORS

H-2.1 The lateral soil resistance for granular soils and normally consolidated clays which have varying soil modulus is modelled according to the equation:

$$\frac{p}{y} = \eta_h z$$

where

η_h = modulus of subgrade reaction for which the recommended values are given in Table 12;

p = lateral soil reaction per unit length of pile at depth z below ground level; and

y = lateral pile deflection.

H-2.2 The lateral soil resistance for over-consolidated clays with constant soil modulus is modelled according to the equation:

$$\frac{p}{y} = K$$

where

$$K = \frac{k_1}{1.5} \times \frac{0.3}{B}$$

where k_1 is Terzaghi's modulus of subgrade reaction as determined from load deflection measurements on a 30 cm square plate and B is the width of the pile (diameter in case of circular piles). The recommended values of k_1 are given in Table 13.

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Table 12 Modulus of Subgrade Reaction for Granular Soils and Normally Consolidated Clays, η_h, kN/m^3

(*Clause* H-2.1 *and* H-2.3.1)

Sl No.	Soil Type	*N* (Blows/30 cm)	Range of η_h kN/m^3 × 10^3	
			Dry	Submerged
(1)	(2)	(3)	(4)	(5)
i)	Soft organic silt	—	—	0.15
ii)	Soft normally consolidated clay	0 to 4	—	0.35 to 0.7
iii)	Very loose sand	0 to 4	< 0.4	< 0.2
iv)	Loose sand	4 to 10	0.4 to 2.5	0.2 to 1.4
v)	Medium sand	10 to 35	2.5 to 7.5	1.4 to 5.0
vi)	Dense sand	> 35	7.5 to 20.0	5.0 to 12.0

NOTE — The η_h values may be interpolated for intermediate standard penetration values, *N*.

Table 13 Modulus of Subgrade Reaction for Cohesive Soil, k_1, KN/m^3

(*Clauses* H-2.2 *and* H-2.3.2)

Sl No.	Soil consistency	Unconfined compression strength, q_u kN/m^2	Range of k_1 kN/m^3 × 10^3
(1)	(2)	(3)	(4)
i)	Soft	25 to 50	4.5 to 9.0
ii)	Medium stiff	50 to 100	9.0 to 18.0
iii)	Stiff	100 to 200	18.0 to 36.0
iv)	Very stiff	200 to 400	36.0 to 72.0
v)	Hard	> 400	>72.0

NOTE — For q_u less than 25, k_1 may be taken as zero, which implies that there is no lateral resistance.

H-2.3 Stiffness Factors

H-2.3.1 *For Piles in Sand and Normally Consolidated Clays*

$$\text{Stiffness factor } T, \text{ in m} = \sqrt[5]{\frac{EI}{\eta_h}}$$

where

E = young's modulus of pile material, in MN/m^2;

I = moment of inertia of the pile cross-section, in m^4; and

η_h = modulus of subgrade reaction variation, in MN/m^3 (*see* Table 12).

H-2.3.2 *For Piles in Over-Consolidated Clays*

$$\text{Stiffness factor } R, \text{ in m} = \sqrt[4]{\frac{EI}{KB}}$$

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where

B = width of pile shaft (diameter in case of circular piles), in m;

E = young's modulus of pile material, in MN/m^2;

I = moment of inertia of the pile cross-section, in m^4; and

K = $\frac{k_1}{1.5} \times \frac{0.3}{B}$, (*See* Table 13 for values of k_1, in MN/m^3, *See also* Note).

NOTE — Table 13 shall be referred if the average unconfined compression strength, q_u is greater than or equal to 100 kN/m^2.

H-3 CRITERIA FOR SHORT RIGID PILES AND LONG ELASTIC PILES

Having calculated the stiffness factor T or R, the criteria for behaviour as a short rigid pile or as a long elastic pile are related to the embedded length L_e as given in Table 14.

Table 14 Criteria for Behaviour of Pile Based on its Embedded Length

(*Clause* H-3)

Sl No.	Type of Pile Behaviour	Relation of Embedded Length with Stiffness Factor	
		Linearly Increasing	Constant
(1)	(2)	(3)	(4)
i)	Short (rigid) pile	$L_e \leq 2T$	$L_e \leq 2R$
ii)	Long (elastic) pile	$L_e \geq 4T$	$L_e \geq 3.5R$

NOTE — The intermediate L shall indicate a case between rigid pile behaviour and elastic pile behaviour.

H-4 DEFLECTION AND MOMENTS IN LONG ELASTIC PILES

H-4.1 Equivalent cantilever approach gives a simple procedure for obtaining the deflections and moments due to relatively small lateral loads. This requires the determination of depth of virtual fixity, z_f.

The depth to the point of fixity may be read from the plots given in Fig. 18. e is the effective eccentricity of the point of load application obtained either by converting the moment to an equivalent horizontal load or by actual position of the horizontal load application. R and T are the stiffness factors described earlier.

H-4.2 The pile head deflection, y shall be computed using the following equations:

Deflection, $y = \frac{H(e+z_f)^3}{3EI} \times 10^3$ .....for free head pile

Deflection, $y = \frac{H(e+z_f)^3}{12EI} \times 10^3$ .....for fixed head pile

where

e cantilever length above ground/bed to the point of load application, in m;

E young's modulus of pile material, in kN/m^2;

H lateral load, in kN;

I moment of inertia of the pile cross section, in m^4;

y deflection of pile head, in mm; and

z_f depth to point of fixity, in m.

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H-4.3 The fixed end moment of the pile for the equivalent cantilever may be determined from the following expressions.

Fixed end moment, $M_F = H(e + z_f)$ …….for free head pile

Fixed end moment, $M_F = \frac{H(e+z_f)}{2}$ …….for fixed head pile

The fixed head moment, M_F of the equivalent cantilever is higher than the actual maximum moment M in the pile. The actual maximum moment may be obtained by multiplying the fixed end moment of the equivalent cantilever by a reduction factor, m, given in Fig. 19.

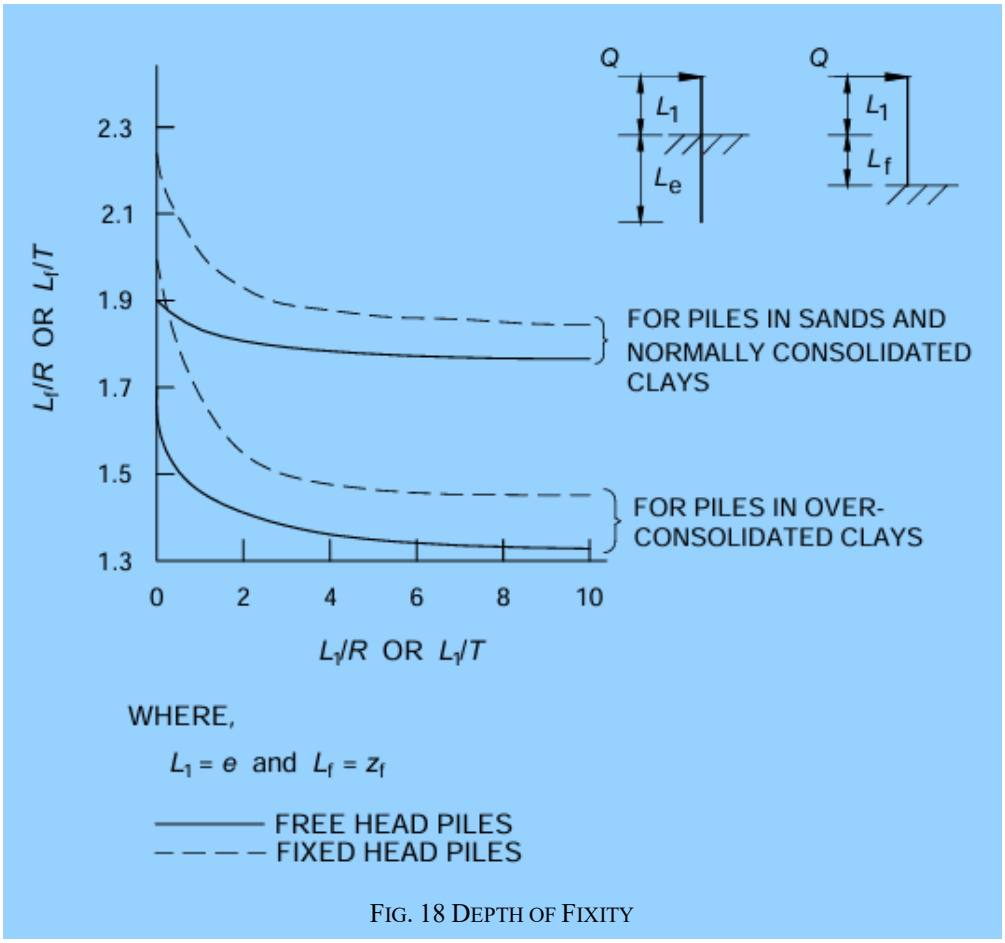

FIG. 18 DEPTH OF FIXITY

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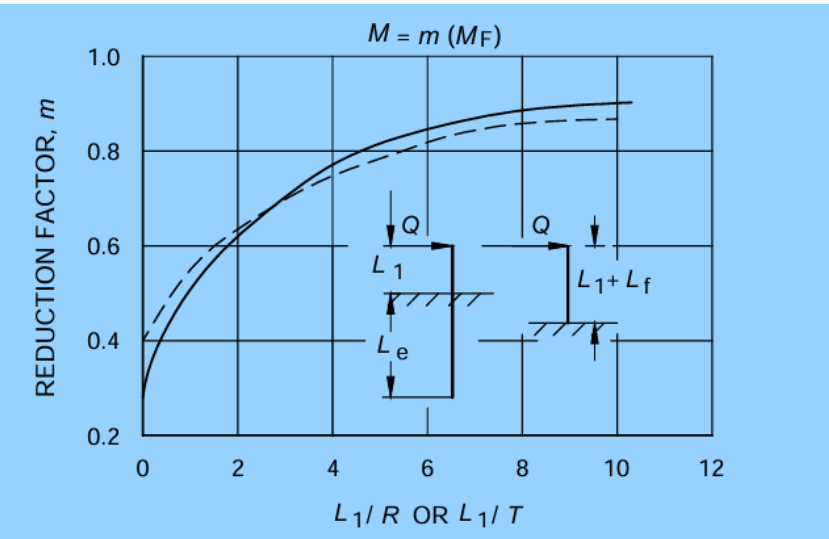
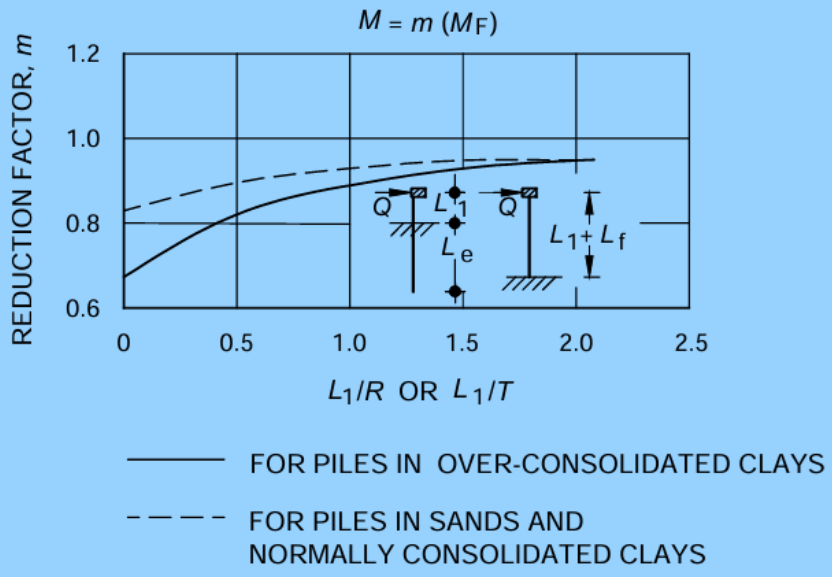


19A FOR FREE HEAD PILE



19B FOR FIXED HEAD PILE

FIG. 19 DETERMINATION OF REDUCTION FACTORS FOR COMPUTATION OF MAXIMUM MOMENT IN PILE

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ANNEX J

(*Clause* 8.5.3.2)

LOAD CARRYING CAPACITY OF UNDER-REAMED PILES FROM SOIL PROPERTIES

J-1 ULTIMATE LOAD CAPACITY

The ultimate load capacity of a pile can be calculated from soil properties. The soil properties required are strength parameters, cohesion, angle of internal friction and soil density.

a) *Clayey soils* — For cohesive soils, the ultimate load carrying capacity of an under-reamed pile may be worked out from the following expression:

$$Q_u = A_p N_c C_p + A_a N_c C'_a + C'_a A'_s + \alpha C_a A_s$$

where

A_a = $(\pi/4)(D_u^2 - D^2)$, where D_u and D are the under-reamed and stem diameter, respectively, in m;

A_p = cross-sectional area of the pile stem at the pile tip, in m^2;

A_s = surface area of the stem, in m^2;

A'_s = surface area of the cylinder circumscribing the under-reamed bulbs, in m^2;

α = reduction factor (usually taken 0.5 for clays);

C_a = average cohesion of the soil along the pile stem, in kN/m^2;

C'_a = average cohesion of the soil around the under-reamed bulbs;

C_p = cohesion of the soil around toe, in kN/m^2;

N_c = bearing capacity factor, usually taken as 9; and

Q_u = ultimate bearing capacity of pile, in kN.

NOTES

1 The above expression holds for the usual spacing of under-reamed bulbs spaced at not more than one and a half times their diameter.

2 If the pile is with one bulb only, the third term will not occur. For calculating uplift load, the first term will not occur in the formula.

3 In case of expansive soil, top 2 m strata should be neglected for computing skin friction.

b) *Sandy soils*

$$Q_u = A_p \left(\frac{1}{2} D.\gamma.N_\gamma + \gamma.d_f.N_q\right) + A_a \left[\frac{1}{2} D_u.n.\gamma.N_\gamma + \gamma.N_q.\left(\sum_{r=1}^{r=n} d_r\right)\right] + \frac{1}{2}.\pi.D.\gamma.K \tan\delta.(d_1^2 + d_f^2 - d_n^2)$$

where

A_a = $\pi/4\ (D_u^2 - D^2)$ where D_u is the under-reamed bulb diameter, in m;

A_p = $\pi\ D^2/4$, where D is stem diameter, in m;

d_1 = depth of the centre of the first under-reamed bulb, in m;

d_f = total depth of pile below ground level, in m;

d_n = depth of the centre of the last under-reamed bulb, in m;

d_r = depth of the centre of different under-reamed bulbs below ground level, in m;

K = earth pressure coefficient (usually taken as 1.75 for sandy soils);

n = number of under-reamed bulbs;

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N_Y, N_q = bearing capacity factors, depending upon the angle of internal friction;

γ = average unit weight of soil (submerged unit weight in strata below water table), in kN/m^3; and

δ = angle of wall friction (may be taken as equal to the angle of internal friction Φ)

NOTES

1 For uplift bearing on pile tip, A_p will not occur.

2 N_Y will be as specified in good practice [C-2(34)] and N_q will be taken from Fig. 15(B).

c) *Soil strata having both cohesion and friction* — In soil strata having both cohesion and friction or in layered strata having two types of soil, the bearing capacity may be estimated using both the formulae. However, in such cases load test will be a better guide.

d) *Compaction piles in sandy strata* — For bored compaction piles in sandy strata, the formula in (b) shall be applied but with the modified value of Φ_1 as given below:

$$\Phi_1 = (\Phi + 40)/2$$

where

Φ = angle of internal friction of virgin soil.

The values of N_Y, N_q and δ are taken corresponding to Φ_1. The value of the earth pressure coefficient K will be 3.

e) *Piles resting on rock* — For piles resting on rock, the bearing component will be obtained by multiplying the safe bearing capacity of rock with bearing area of the pile stem plus the bearing provided by the bulb portion.

NOTE — To obtain safe load in compression and uplift from ultimate load capacity generally the factors of safety will be 2.5 and 3, respectively.

ANNEX K

(*Clause* 9.3.3.4)

LOAD CARRYING CAPACITY

K-1 LOAD CARRYING CAPACITY OF CPRF SYSTEM

K-1.1 The ultimate vertical capacity shall be taken as the least of:

a) the capacity of the block containing the piles, plus that of the portion of the raft outside the periphery of the pile group;

b) the sum of the capacity of the raft, $Q_{\text{unpiled-raft}}$ and of all the piles $Q_{\text{pile-group}}$ in the system, expressed as, $Q_{\text{CPRF}} = 0.8\ (Q_{\text{unpiled-raft}} + Q_{\text{pile-group}})$; and

c) the capacity of CPRF system considering pile-soil-raft interactions. The capacity of CPRF should be expressed as:

$$Q_{\text{CPRF}} = \alpha_{pr}\alpha_{pp}\sum_{n=1}^{n} Q_{single\ pile} + \alpha_{rp}Q_{unpiled\ raft}$$

In the above expression:

1) $Q_{unpiled\ raft}$ should be calculated based on good practice [C-2(34)];

2) $Q_{single\ pile}$ should be calculated based on good practice [C-2(51)]; and

3) α_{pp}, α_{rp} and α_{pr} are the pile-pile, raft-pile and pile-raft interaction factors, respectively. These interaction factors may be suitably taken from various relevant literature.

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ANNEX L

(*Clause* 11.2)

SOIL IMPROVEMENT METHODS

L-1 SUMMARY OF SOIL IMPROVEMENT METHODS

	Method	Principle	Most Suitable Soil Conditions/ Types	Maximum Effective Treatment Depth	Special Materials Required	Special Equipment Required	Properties of Treated Material	Special Advantages and Limitation	Relative Cost
In-Situ Deep compaction of Cohesionless Soils	Blasting	Shock waves and vibrations cause liquefaction; displacement with settlement to higher density	Saturated, clean sands: partly saturated sands and silts (collapsible loss) after flooding	> 30 m	Explosives, backfill to plug drill holes, hole casings	Jetting or drilling machine	Can obtain relative densities to 70 to 80, may get variable density strength gain	Rapid, inexpensive, can treat any size areas: variable properties, no improvement near surface, dangerous	Low
	Vibratory Probe	Densification by vibration; liquefaction induced settlement under overburden	Saturated or dry clean sand	20 m (Ineffective above 3 m to 4 m depth)	None	Vibratory pile driver and 750 mm dia open steel pipe	Can obtain relative densities of up to 80 percent. Ineffective in some sands	Rapid, simple, good underwater, soft under layers may damp vibrations, difficult to penetrate, stiff over layers, not good in partly saturated soils	Moderate
	Vibro-compaction	Densification by vibration and compaction of backfill material	Cohesionless soils with less than 20 fines	30 m	Granular backfill, water supply	Vibroflot, crane, pumps	Can obtain high relative densities, good uniformity	Useful in saturated and partly saturated soils, uniformity	Moderate
In-Situ Deep compaction of Cohesionless Soils	Compaction piles	Densification by displacement of pile volume and by vibration during driving	Loose sandy soils: partly saturated clayey soils, loess	> 20 m	Pile material (often sand or soil plus cement mixture)	Pile driver, special sand pile equipment	Can obtain high densities, good uniformity	Useful in soils with fines, uniform compaction, easy to check results, slow, limited improvement in upper 1 m to 2 m	Moderate to high

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Table (*Continue*)

	Method	Principle	Most Suitable Soil Conditions/ Types	Maximum Effective Treatment Depth	Special Materials Required	Special Equipment Required	Properties of Treated Material	Special Advantages and Limitation	Relative Cost
In-Situ Deep compaction of Cohesionless Soils	Blasting	Shock waves and vibrations cause liquefaction; displacement with settlement to higher density	Saturated, clean sands: partly saturated sands and silts (collapsible loss) after flooding	> 30 m	Explosives, backfill to plug drill holes, hole casings	Jetting or drilling machine	Can obtain relative densities to 70 to 80, may get variable density strength gain	Rapid, inexpensive, can treat any size areas: variable properties, no improvement near surface, dangerous	Low
	Vibratory Probe	Densification by vibration; liquefaction induced settlement under overburden	Saturated or dry clean sand	20 m (Ineffective above 3 m to 4 m depth)	None	Vibratory pile driver and 750 mm dia open steel pipe	Can obtain relative densities of up to 80 percent. Ineffective in some sands	Rapid, simple, good underwater, soft under layers may damp vibrations, difficult to penetrate, stiff over layers, not good in partly saturated soils	Moderate
	Vibro-compaction	Densification by vibration and compaction of backfill material	Cohesionless soils with less than 20 fines	30 m	Granular backfill, water supply	Vibroflot, crane, pumps	Can obtain high relative densities, good uniformity	Useful in saturated and partly saturated soils, uniformity	Moderate
In-Situ Deep compaction of Cohesionless Soils	Compaction piles	Densification by displacement of pile volume and by vibration during driving	Loose sandy soils: partly saturated clayey soils, loess	> 20 m	Pile material (often sand or soil plus cement mixture)	Pile driver, special sand pile equipment	Can obtain high densities, good uniformity	Useful in soils with fines, uniform compaction, easy to check results, slow, limited improvement in upper 1 m to 2 m	Moderate to high

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Table (*Continue*)

	Method	Principle	Most Suitable Soil Conditions/ Types	Maximum Effective Treatment Depth	Special Materials Required	Special Equipment Required	Properties of Treated Material	Special Advantages and Limitation	Relative Cost
	Displacements grout	Highly viscous grout acts as radical hydraulic jack when pumped in under high pressure	Soft, fine-grained soils; foundation soils with large voids or cavities	Unlimited, but a few metre usual	Soil, cement water	Batching equipment, high pressure pumps, hoses	Grout bulbs within compressed soil matrix	Good for correction of differential settlements, filling large voids; careful control required	Low material high injection
	Electro-kinetic injection	Stabilization chemicals moved into soil by electro-osmosis or colloids into pores by electrophoresis	Saturated silts; silty clays (clean sands in case of colloid injection)	Unknown	Chemicals stabilizer colloidal void fillers	d.c. power supply, anodes, cathodes	Increased strength, reduced compressibility reduced liquefaction potential	Existing soil and structures not subjected to high pressures; not good in soils with high conductivity	Expansive
	Jet grouting	High speed jets at depth excavate, inject and mix stabilizer with soil to form columns or panels	Sands, silts, clays	—	Water, stabilizing chemicals	Special jet nozzle, pumps, pipes and hoses	Solidified columns and walls	Useful in soils that can't be permeation grouted, precision in locating treated zones	--

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Table (*Continue*)

	Method	Principle	Most Suitable Soil Conditions/ Types	Maximum Effective Treatment Depth	Special Materials Required	Special Equipment Required	Properties of Treated Material	Special Advantages and Limitation	Relative Cost
Precompression	Preloading with/without drain	Load is applied sufficiently in advance of construction so that compression of soft soils is completed prior to development of the site	Normally consolidated soft clays, silts, organic deposits, completed sanitary landfills	—	Earth fill or other material for loading the site; sand or gravel for drainage blanket	Earth moving equipment, large water tanks or vacuum drainage systems sometimes used; settlement markers, piezometers	Reduced water content and void ratio, increased strength	Easy, theory well developed, uniformity; requires long time (vertical drains can be used to reduce consolidation time)	Low (moderate, if vertical drains are required)
	Surcharge fills	Fill in excess of that required permanently is applied to achieve a given amount of settlement in a shorter time; excess fill then removed	Normally consolidated soft clays, silts, organic deposits, completed sanitary landfills	—	Earth fill or other material for loading the site; sand or gravel for drainage blanket	Earth moving equipment; settlement markers, piezometers	Reduced water content, void ratio and compressibility increased strength	Faster than preloading without surcharge, theory well developed, extra material handling; can use vertical drains to reduce consolidation time	Moderate
	Electro-osmosis	d. c. current causes water flow from anode towards cathode where it is removed	Normally consolidated silts and silty clays	—	Anodes (usually rebars or aluminium) cathodes (well points or rebars)	d. c. power supply, wiring, metering systems	Reduced water content and compressibility, increased strength, electrochemical hardening	No fill loading required, can be used in confined area, relatively fast; non-uniform properties between electrodes; not good in highly conductive soils	High

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Table (*Continue*)

	Method	Principle	Most Suitable Soil Conditions/ Types	Maximum Effective Treatment Depth	Special Materials Required	Special Equipment Required	Properties of Treated Material	Special Advantages and Limitation	Relative Cost
Admixtures	Remove and replace	Foundation soil excavated, improved by drying or admixture and re-compacted	Inorganic soils	10 m	Admixture stabilizers	Excavating, mixing and compaction equipment, dewatering system	Increased strength and stiffness, reduced compressibility	Uniform, controlled foundation soil when replaced; may require large area dewatering	High
	Structural fills	Structural fill distributes loads to underlying soft soils	Use over soft clays or organic soils, marsh deposits	—	Sand, gravel fly ash, bottom ash, slag, expanded aggregate, clam shell or oyster shell, incinerator ash	Mixing and compaction equipment	Soft subgrade protected by structural load-bearing fill	High strength, good load distribution to underlying soft soils	Low to high
	Mix-in-place piles and walls	Lime cement or asphalt introduced through rotating auger or special in-place mixer	All soft or loose inorganic soils	>20 m	Cement lime asphalt, or chemical stabilizer	Drill rig, rotary cutting and mixing head, additive pro-portioning equipment	Solidified soil piles for walls of relatively high strength	Use native soil, reduced lateral support requirements during excavation; difficult quality control	Moderate to high

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Table (*Continue*)

	Method	Principle	Most Suitable Soil Conditions/ Types	Maximum Effective Treatment Depth	Special Materials Required	Special Equipment Required	Properties of Treated Material	Special Advantages and Limitation	Relative Cost
Thermal	Heating	Drying at low temperatures; alteration of clays at intermediate temperatures (400 to 600 °C); fusion at high temperatures (>1 000 °C)	Fine- grained soils, especially partly saturated clays and silts, loess	15 m	Fuel	Fuel tanks, burners, blowers	Reduced water content, plasticity, water sensitivity; increased strength	Can obtain irreversible improvements in properties; can introduce stabilizers with hot gases	High
	Freezing	Freeze soft, wet ground to increase its strength stiffness	All soils	Several metre	Refrigerant	Refrigeration system	Increased strength and stiffness, reduced permeability	No good in flowing ground water, temporary	High
Reinforcement	Vibro replacement sand/stone columns	Hole jetted into soft, fine-grained soil and backfilled with densely compacted gravel or sand	Soft clays and alluvial deposits	20 m	Gravel or crushed rock backfill	Vibroflot, crane or vibrocat, water	Increased bearing capacity, reduced settlement	Faster than precompression, avoids dewatering required for remove and replace; limited bearing capacity	Moderate to high
	Root piles, soils nailing	Inclusions used to carry tension, shear, compression	All soils	—	Reinforcing bars, cement grout	Drilling and grouting equipment	Reinforced zone behaves as a coherent mass	*In-situ* reinforcement for soils that can't be grouted or mixed in-place with admixtures	Moderate to high

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Table (*Concluded*)

	Method	Principle	Most Suitable Soil Conditions/ Types	Maximum Effective Treatment Depth	Special Materials Required	Special Equipment Required	Properties of Treated Material	Special Advantages and Limitation	Relative Cost
	Strips and membranes	Horizontal tensile strips, membranes buried in soil under embankments, gravel base courses and footings	All soils	Can construct earth structures to heights of several metres	Metal or plastic strips, geotextiles	Excavating, earth handling and compaction equipment	Self-supporting earth structures, increased bearing capacity, reduced deformations	Economical, earth structures coherent, can tolerate deformations; increased allowable bearing pressure	Low to moderate

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LIST OF STANDARDS

The following list records those standards which are acceptable as 'good practice' and 'accepted standards' in the fulfilment of the requirements of this NBCS. The latest version of a standard shall be adopted at the time of enforcement of this NBCS. The standards listed may be used for conformance with the requirements of the referred clauses in this NBCS.

In the following list, the number appearing in the first column within parentheses indicates the number of the reference in this Part/Section of this NBCS.

	IS No.	*Title*
(1)	IS 1892 : 2021	Subsurface investigation for foundations — Code of practice (*second revision*)
	IS 2131 : 2025	Standard penetration test of soil — Method of test (*second revision*)
	IS 2132 : 1986	Code of practice for thin walled tube sampling of soils (*second revision*)
	IS 4434 : 1978	Code of practice for *in-situ* vane shear test for soils (*first revision*)
	IS 4968	Method for subsurface sounding for soils
	(Part 1) : 1976	Dynamic method using 50 mm cone without bentonite slurry (*first revision*)
	(Part 2) : 1976	Dynamic method using cone and bentonite slurry (*first revision*)
	(Part 3) : 1976	Static cone penetration test (*first revision*)
	IS 8763:1978	Guide for undisturbed sampling of sands
	IS 9214 : 1979	Method for determination of modulus of subgrade reaction (*k*-value) of soils in the field
(2)	IS 19235 : 2025	Geotechnical engineering services — Requirements
(3)	IS 6926 : 1996	Diamond core drilling — Site investigation for river valley projects — Code of practice (*first revision*)
(4)	IS 10208 : 1982	Specification for diamond core drilling equipment
(5)	IS 4078 : 1980	Code of practice for indexing and storage of drill cores (*first revision*)
(6)	IS 2131 : 2025	Standard penetration test of soil — Method of test (*second revision*)
(7)	IS 4434 : 1978	Code of practice for *In-situ* vane shear test for soils (*first revision*)
(8)	IS 1892 : 2021	Subsurface investigation for foundations — Code of Practice (*second revision*)
(9)	IS 1888 : 1982	Method of load tests on soils (*second revision*)
(10)	IS 9214 : 1979	Method for determination of modulus of subgrade reaction (*K* value) of soils in the field
(11)	IS 1888:1982	Method of load test on soils (*second revision*)
(12)	IS 5249 : 1992	Determination of dynamic properties of soil — Method of test (*second revision*)
(13)	IS 4968	Method for subsurface sounding for soils
(14)	(Part 1) : 1976	Dynamic method using 50 mm cone without bentonite slurry (*first revision*)

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(15)	(Part 2) : 1976	Dynamic method using cone and bentonite slurry (*first revision*)
(16)	(Part 3) : 1976	Static cone penetration test
(17)	IS 10042 : 1981	Code of practice for site-investigations for foundation in gravel boulder deposits
(18)	IS 15736 : 2007	Geological exploration by geophysical method (electrical resistivity) — Code of practice
(19)	IS 3043 : 2018	Code of practice for earthing (*second revision*)
(20)	IS 15681:2006	Geological exploration by geophysical method (seismic refraction) — Code of practice
(21)	IS 13372	Seismic testing of rock mass — Code of practice
(22)	(Part 1) : 1992	Within a borehole
(23)	(Part 2) : 1992	Between the borehole
(24)	IS 5529 (Part 1) : 2013	*In-situ* permeability test: Part 1 Test in overburden
(25)	IS 2470 (Part 1) : 1985	Code of practice for installation of septic tanks: Part 1 Design criteria and construction (*second revision*)
(26)	IS 5529 (Part 2): 2006	*In-situ* permeability test: Part 2 Tests in bedrock (*second revision*)
(27)	IS 2720 (Part 31) : 1990	Methods of tests for soils field determination of California bearing ratio (*first revision*)
(28)	IS 2720	Methods of tests for soils:
	(Part 1) : 1983	Preparation of dry soil samples for various tests (*second revision*)
	(Part 2) : 1973	Determination of water content (*second revision*)
	(Part 3/Sec 1) : 1980	Determination of specific gravity, Section 1 Fine grained soils (*first revision*)
	(Part 3/Sec 2) : 1980	Determination of specific gravity, Section 2 Fine, medium and coarse grained soils (*first revision*)
	(Part 4) : 1985	Grain size analysis (*second revision*)
	(Part 5) : 1985	Determination of liquid and plastic limit (*second revision*)
	(Part 10) : 1991	Determination of unconfined compressive strength (*second revision*)
	(Part 11) : 1993	Determination of the shear strength parameters of a specimen tested in unconsolidated undrained triaxial compression without the measurement of pore water pressure(*first revision*)
	(Part 12) : 1981	Determination of shear strength parameters of soil from consolidated undrained triaxial compression test with measurement of pore water pressure (*first revision*)
	(Part 13) : 1986	Direct shear test (*second revision*)
	(Part 15) : 1986	Determination of consolidation properties (*first revision*)
	(Part 28) : 1974	Determination of dry density of soils, in-place, by the sand replacement method (*first revision*)
	(Part 29) : 1975	Determination of dry density of soils in-place, by the core cutter method (*first revision*)

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	(Part 33) : 1971	Determination of the density in-place by the ring and water replacement method
	(Part 34):1972	Determination of density of soils in-place by rubber-balloon method
	(Part 39/Sec 1) : 1977	Direct shear test for soils containing gravel, Section 1 Laboratory test
(29)	IS 1498 : 1970	Classification and identification of soils for general engineering purposes (*first revision*)
(30)	IS 401 : 2001	Preservation of timber — Code of practice (*fourth revision*)
(31)	IS 15180 : 2002	Guidelines for use in prediction of subsidence and associated parameters in coal mines having nearly horizontal single seam workings
(32)	IS 14243	Selection and development of site for building in hill areas — Guidelines
	(Part 1) : 1995	Microzonation of urban centres
	(Part 2) : 1995	Selection and development
(33)	IS 11134 : 1984	Code of practice for setting out of buildings
(34)	IS 3764 : 1992	Excavation work — Code of safety (*first revision*)
(35)	IS 1904 : 2021	General requirements for design and construction of foundations in soils — Code of practice (*fourth revision*)
(36)	IS 6403 : 1981	Code of practice for determination of bearing capacity of shallow foundations (*first revision*)
(37)	IS 12070 : 1987	Code of practice for design and construction of shallow foundations on rocks
(38)	IS 1080 : 1985	Code of practice for design and construction of shallow foundations in soils (other than raft, ring and shell) (*second revision*)
(39)	IS 11089 : 1984	Code of practice for design and construction of ring foundations
(40)	IS 9456 : 1980	Code of practice for design and construction of conical and hyperbolic paraboloidal types of shell foundations
(41)	IS 2974 (Part 1) : 1982	Code of practice for design and construction of machine foundations: Part 1 Foundations for reciprocating type machines (*second revision*)
(42)	IS 2911 (Part 4) : 2013	Design and construction of pile foundations — Code of practice: Part 4 Load test on piles (*second revision*)
(43)	IS 14593 : 1998	Design and construction of bored cast *in-situ* piles founded on rocks — Guidelines
(44)	IS 4968	Method for subsurface sounding for soils:
	(Part 1) : 1976	Dynamic method using 50 mm cone without bentonite slurry (*first revision*)
	(Part 2) : 1976	Dynamic method using cone and bentonite slurry (*first revision*)
	(Part 3) : 1976	Static cone penetration test (*first revision*)
(45)	IS 2911	Design and construction of pile foundations — Code of practice
	(Part 1/Sec 1) : 2010	Concrete piles, Section 1 Driven cast *in-situ* concrete piles (*second revision*)
	(Part 1/Sec 2) : 2010	Concrete piles, Section 2 Bored cast *in-situ* concrete piles (*second revision*)

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(46)	IS 14893 : 2021	Low strain non-destructive integrity testing of piles — Guidelines (*first revision*)
(47)	IS 2911 (Part 1/Sec 3) : 2010	Design and construction of pile foundations — Code of practice, Concrete piles, Section 3 Driven precast concrete piles (*second revision*)
(48)	IS 2911 (Part 1/Sec 4) : 2010	Design and construction of pile foundations — Code of practice, Concrete piles, Section 4 Precast concrete piles in prebored holes (*first revision*)
(49)	IS 456 : 2000	Plain and reinforced concrete — Code of practice (*fourth revision*)
(50)	IS 2911 (Part 3) : 2021	Design and construction of pile foundations — Code of practice, Under-reamed piles (*second revision*)
(51)	IS 2911 (Part 2) : 2021	Design and construction of pile foundations — Code of practice, Timber piles (*second revision*)
(52)	IS 1892 : 2021	Subsurface investigation for foundations — Code of practice (*second revision*)
	IS 2720	Methods of test for soils:
	(Part 1) : 1983	Preparation of dry soil samples for various tests (*second revision*)
	(Part 2) : 1973	Determination of water content (*second revision*)
	(Part 3/Sec 1) : 1980	Determination of specific gravity, Section 1 Fine grained soils (*first revision*)
	(Part 3/Sec 2) : 1980	Determination of specific gravity, Section 2 Fine, medium and coarse grained soils (*first revision*)
	(Part 4) : 1985	Grain size analysis (*second revision*)
	(Part 5) : 1985	Determination of liquid and plastic limits (*second revision*)
	(Part 6) : 1972	Determination of shrinkage factors (*first revision*)
	(Part 7) : 1980	Determination of water content — Dry density relation using light compaction (*second revision*)
	(Part 8) : 1983	Determination of water content — Dry density relation using heavy compaction (*second revision*)
	(Part 9) : 1992	Determination of dry density — Moisture content relation by constant mass of soil method (*first revision*)
	(Part 10) : 1991	Determination of unconfined compressive strength (*second revision*)
	(Part 11) : 1993	Determination of the shear strength parameters of a specimen tested in unconsolidated undrained triaxial compression without the measurement of pore water pressure (*first revision*)
	(Part 12) : 1981	Determination of shear strength parameters of soil from consolidated undrained triaxial compression test with measurement of pore water pressure (*first revision*)
	(Part 13) : 1986	Direct shear test (*second revision*)
	(Part 14) : 1983	Determination of density index (relative density) of cohesionless soils (*first revision*)
	(Part 15) : 1986	Determination of consolidation properties (*first revision*)
	(Part 16) : 1987	Laboratory determination of cbr (*second revision*)
	(Part 17) : 1986	Laboratory determination of permeability (*first revision*)
	(Part 18) : 1992	Determination of field moisture equivalent (*first revision*)
	(Part 19) : 1992	Determination of centrifuge moisture equivalent (*first revision*)
	(Part 20) : 1992	Determination of linear shrinkage (*first revision*)

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	(Part 21) : 1977	Determination of total soluble solids (*first revision*)
	(Part 22) : 1972	Determination of organic matter (*first revision*)
	(Part 23) : 1976	Determination of calcium carbonate
	(Part 24) : 1976	Determination of cation exchange capacity (*first revision*)
	(Part 25) : 1982	Determination silica sesquioxide ratio (*first revision*)
	(Part 26) : 1987	Determination of pH value (*second revision*)
	(Part 27) : 1977	Determination of total soluble sulphates (*first revision*)
	(Part 28) : 1974	Determination of dry density of soils in place, by the sand replacement method (*first revision*)
	(Part 29) : 1975	Determination of dry density of soils in place, by the core cutter method (*first revision*)
	(Part 30) : 1980	Laboratory vane shear test
	(Part 31) : 1990	Field determination of california bearing ratio (*first revision*)
	(Part 33) : 1971	Determination of the density in-place by the ring and water replacement method
	(Part 34) : 1972	Determination of density of soils in-place by rubber-balloon method
	(Part 35) : 1974	Measurement of negative pore water pressure
	(Part 36) : 1987	Laboratory determination of permeability of granular soils (constant head) (*first revision*)
	(Part 37) : 1976	Determination of sand equivalent values of soils and fine aggregates
	(Part 38) : 1976	Compaction control test (hilf method)
	(Part 39/Sec 1) : 1977	Direct shear test for soils containing gravel, Section 1 Laboratory test
	(Part 39/Sec 2) : 1979	Direct shear test for soils containing gravel, Section 2 In-situ shear test
	(Part 40) : 1977	Determination of free swell index of soils
	(Part 41) : 1977	Measurement of swelling pressure of soils
(53)	IS 2911	Code of practice for design and construction of pile foundations:
	(Part 1/Sec 1) : 2010	Concrete piles, Section 1 Driven cast in-situ concrete piles (*second revision*)
	(Part 1/Sec 2) : 2010	Concrete piles, Section 2 Bored cast in-situ concrete piles (*second revision*)
	(Part 1/Sec 3) : 2010	Concrete piles, Section 3 Precast driven concrete piles (*second revision*)
	(Part 1/Sec 4) : 2010	Concrete piles, Section 4 Precast concrete piles in prebored holes (*first revision*)
	(Part 2) : 1980	Timber piles (*first revision*)
	(Part 3) : 1980	Under-reamed pile foundation (*first revision*)
	(Part 4) : 2013	Load test on piles (*second revision*)
(54)	IS 2911	Code of practice for design and construction of pile foundations:
	(Part 1/Sec 1) : 2010	Concrete piles, Section 1 Driven cast in-situ concrete piles (*second revision*)
	(Part 1/Sec 2) : 2010	Concrete piles, Section 2 Bored cast in-situ concrete piles (*second revision*)
	(Part 1/Sec 3) : 2010	Concrete piles, Section 3 Precast driven concrete piles (*second revision*)

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	(Part 1/Sec 4) : 2010	Concrete piles, Section 4 Precast concrete piles in prebored holes (*first revision*)
(55)	IS 2950 (Part 1):1981	Code of practice for design and construction of raft foundations: Part 1 design (*second revision*)
(56)	IS 8009 (Part 1) : 1976	Code of practice for calculation of settlement of foundations: Part 1 Shallow foundations subjected to symmetrical static vertical loads
(57)	IS 2974	Code of practice for design and construction of machine foundation:
	(Part 1) : 1982	Foundations for reciprocating type machine (*second revision*)
	(Part 2) : 1980	Foundations for impact type machines (hammer foundations) (*first revision*)
	(Part 3) : 1992	Foundations for rotary type machines (medium and high frequency) (*second revision*)
	(Part 4) : 1979	Foundations for rotary type machines of low frequency (*first revision*)
	(Part 5) : 1987	Foundations for impact machines other than hammers (forging and stamping press; pig breakers, drop crusher and jolter) (*first revision*)
	IS 13301 : 1992	Vibration isolation for machine foundations — Guidelines
	IS 9556 : 1980	Code of practice for design and construction of diaphragm walls
(58)	IS 13094 : 2021	Selection of ground improvement techniques for weak soils — Guidelines (*first revision*)
(59)	IS 15284	Design and construction for ground improvement — Guidelines:
	(Part 1) : 2003	Stone columns
(60)	(Part 2) : 2004	Preconsolidation using vertical drains
(61)	IS 13162 (Part 2) : 2024	Geotextiles — Methods of test: Part 2 Determination of resistance to exposure of ultra-violet light and water (Xenon arc type apparatus) (*first revision*)
	IS 13321 (Part 1) : 2022	Geosynthetics: Part 1 Terms and definitions
	IS 13325 : 1992	Determination of tensile properties of extruded polymer geogrids using the wide strip — Test method
	IS 13326 (Part 1) : 2025	Geosynthetics — Determination of friction characteristics: Part 1 Direct shear test (*first revision*)
	IS 14293 : 2024	Geotextiles — Method for determination of trapezoid tearing strength (*first revision*)
	IS 14294 : 2024	Geotextiles — Method for determination of apparent opening size by dry sieving technique (*first revision*)
	IS 14324 : 2025	Geotextiles and geotextile-related products — Determination of water permeability characteristics normal to the plane, without load (*first revision*)
	IS 14706 : 2024	Geosynthetics — Sampling and preparation of test specimens (*first revision*)
	IS 14714 : 2024	Geotextiles — Determination of abrasion resistance (*first revision*)
	IS 14715	Jute geotextiles:
	(Part 1) : 2016	Strengthening of sub-grade in roads — Specification (*second revision*)
	(Part 2) : 2016	Control of bank erosion in rivers and waterways — Specification (*second revision*)

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	IS 14716 : 2021	Geosynthetics test method for the determination of mass per unit area of geotextiles and geotextile-related products (*first revision*)
	IS 14739 : 2021	Geotextiles and geotextile-related products determination of tensile creep and creep rupture behaviour (*first revision*)
	IS 14986 : 2001	Jute geo-grid for rain water erosion control in road and railway embankments and hill slopes
	IS 15060 : 2018	Geosynthetics — Tensile test for joint seams by wide-width strip method (*first revision*)
(62)	IS 2720	Methods of tests for soils:
	(Part 11) : 1993	Determination of the shear strength parameters of a specimen tested in unconsolidated undrained triaxial compression without the measurement of pore water pressure (*first revision*)
	(Part 12) : 1981	Determination of shear strength parameters of soil from consolidated undrained triaxial compression test with measurement of pore water pressure (*first revision*)

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 3 TIMBER AND BAMBOO: 3A TIMBER

BUREAU OF INDIAN STANDARDS

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National Building Code Sectional Committee, CED 46

FOREWORD

This NBCS (Part C/Subsection 3A) deals with the structural design aspect of timber structures. In this Subsection, the various species of Indian timber, classified into three groups depending on the structural properties influencing the design most, are included.

This Section was first published in 1970 which was subsequently revised in 1983, 2005 and 2016 as a part of National Building Code of India. In the 1983 version of this code, timber was covered under Section 3 of Part 6 under the title 'Wood', which did not cover bamboo. In the version of 2005, the scope of Section 3 was enlarged as Section 3 Timber and Bamboo, which was subdivided into Subsection 3A 'Timber' and Subsection 3B 'Bamboo'. This Subsection pertains to 3A 'Timber'.

In the first revision of 1983, provisions of this Subsection were updated and design of nailed laminated timber beams, were included and information on bolted construction joints was added. In the second revision of 2005, a number of terminologies related to timber for structural purpose were added, strength data of additional species of timber were included, requirements for structural timber and preferred cut sizes thereof were modified, requirements for glued laminated construction and finger joints and also for laminated veneer lumber were introduced. Brief details were included for structural sandwiches, glued laminated beams, lamella roofing, nail and screw holding power of timber, structural use of plywood and trussed rafter. Further, guidelines for protection against termite attack in buildings were also incorporated in the last revision of this Subsection. Also, keeping into consideration the dwindling forest resources, brief details on use of plantation timbers including certain fast growing species and suitable guidelines in terms of their seasoning, sawing, treatment, etc and also use of finger jointing and glued laminated timber was introduced in the last revision so as to effect judicious use of timber.

The third revision of 2016 addressed several significant changes, including modifications to definitions of various terms to align with prevailing engineering practices and the addition of new terminologies related to structural timber. The permissible lateral strength (in double shear) of mild steel common wire was updated to include various new timber species, with modified strength values for certain existing species based on the latest research. Guidance on increased permissible lateral strength for mild steel nails clenched across the grain, as opposed to nails with points cut flush, was incorporated. Design provisions for timber connector joints were further updated, along with data on block shear test results of glued timber joints based on indigenous studies. Additionally, information on the strength properties of glued finger joints from indigenous research was included for design guidance. A new figure illustrating possible orientations of planks in glue-laminated beams (glulam) was added, as well as a design outline for horizontally laminated beams. Updates were also made to the design provisions for trussed rafters, including data on lateral load-bearing strength of common wire nails in plywood-to-wood joints. Provisions for lamella roofing design were detailed and the composite nail/screw-holding power of specific Indian timbers was provided for design guidance.

In this revision, this publication has been renamed as National Building Construction Standards (NBCS).

As a result of experience gained in implementation of 2016 version and feedback received as well as formulation of new standards in the field and revision/updation of some of the existing standards, a need to revise this Subsection was felt. This revision has, therefore, been brought out to take care of these aspects.

The significant changes incorporated in this revision include the following:

a) Cross-laminated timber components including material specifications, dimensions, and fire resistance requirements;

b) Mass timber construction and its components given in **21**;

c) Fire safety enhancements and fire-resistant assembly methods for mass timber, ensuring compliance with structural and fire safety standards;

d) Focus on sustainability with introduction of life cycle assessment (LCA) to evaluate the environmental impact of timber from production to disposal; and

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e) Introduction of mechanical properties of foreign timber species, to support informed design, specification and safe application.

Reference may be made to Part F 'Fire and Life Safety' of this NBCS for fire protection of the timber buildings.

The information contained in this Subsection is largely based on the following Indian Standards:

IS 399 : 1963	Classification of commercial timbers and their zonal distribution (*revised*)
IS 883 : 2016	Design of structural timber in building — Code of practice for (*fifth revision*)
IS 1150 : 2000	Trade names and abbreviated symbols for timber species (*third revision*)
IS 2366 : 1983	Code of practice for nail-jointed timber construction (*first revision*)
IS 4891 : 1988	Specification for preferred cut sizes of structural timber (*first revision*)
IS 4983 : 1968	Code of practice for design and construction of nailed laminated timber beams
IS 11096 : 1984	Code of practice for design and construction of bolt jointed timber construction
IS 14616 : 1999	Laminated veneer lumber — Specification

As the regulation of land development and buildings is a State subject and is dealt by the States and local bodies such as urban local bodies within their jurisdiction through building regulatory documents like building regulations, building byelaws and fire regulations, this document is voluntary in nature and is non-binding. The implementation depends on adoption by concerned parties or stipulations in a contract or by appropriate adoption by the concerned authorities.

For the purpose of deciding whether a particular requirement of this Subsection is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 2022 'Rules for rounding off numerical values (*second revision*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this Subsection.

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Important Explanatory Note for Users of this NBCS

In any Part/Section of this NBCS, where reference is made to **'good practice'** in relation to design, constructional procedures or other related information, and where reference is made to **'accepted standard'** in relation to material specification, testing, or other related information, the Indian Standards listed at the end of the Part/Section shall be used as a guide to the interpretation.

At the time of publication, the editions indicated in the standards were valid. All standards are subject to revision and parties to agreements based on any Part/Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

In the list of standards given at the end of a Subsection, the number appearing within parentheses in the first column indicates the number of the reference of the standard in the Part/Section.

For example:

a) Good practices [C-3A(1)] refers to the Indian Standard(s) give at serial number (1) of the list of standards given at the end of this Part C/Subsection 3A, that is, IS 707 : 2011 'Timber technology and utilization of wood, bamboo and cane — Glossary of terms (*third revision*)'.

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 3 TIMBER AND BAMBOO: 3A TIMBER

1 SCOPE

1.1 This NBCS (Part C/Subsection 3A) covers the general principles involved in the design of structural timber in buildings, including elements of structures connected by fasteners/fastening techniques.

1.2 This shall not be interpreted to prevent the use of material or methods of design or construction not specifically mentioned herein; and the methods of design may be based on analytical and engineering principles, or reliable test data, or both, that demonstrate the safety and serviceability of the resulting structure. The classification of timber into strength groups should not be interpreted as preventing the use of design data desired for a particular timber or grade of timber on the basis of reliable tests.

2 TERMINOLOGY

For the purpose of this Subsection, the following definitions and those given in the accepted standard [C-3A(1)], shall apply.

2.1 Structural Purpose Definitions

2.1.1 *Beam*, *Built-up Laminated* — A beam made by joining layers of timber together with mechanical fastenings, so that the grain of all layers is essentially parallel.

2.1.2 *Beam*, *Glued-Laminated* — A beam made by bonding layers of veneers or timber with an adhesive, so that grain of all laminations is essentially parallel.

2.1.3 *Bressummer* — A beam, across broad opening, sustaining super structure.

2.1.4 *Diaphragm*, *Structural* — A structural element of large extent placed in a building as a wall, or roof, and made use of to resist horizontal forces such as wind or earthquakes-acting parallel to its own plane.

2.1.5 *Duration of Load* — Period during which a member or a complete structure is stressed as a consequence of the loads applied.

NOTE — Timber is stronger under loads of brief duration than under permanently applied loads.

2.1.6 *Edge Distance* — The distance measured perpendicular to grain from the centre of the connector to the edge of the member.

2.1.7 *End Distance* — The distance measured parallel to grain of the member from the centre of the connector to the closest end of timber.

2.1.8 *Finger Joint* — End joint formed by machining a number of similar, tapered, symmetrical fingers in the ends of timber member, which are then bonded together.

2.1.9 *Finger Jointed Timber* — Timber containing adhesively bonded finger joints spaced at manufacturer specified minimum spacing.

2.1.10 *Fundamental or Ultimate Stress* — The stress which is determined on small clear specimen of timber, in accordance with accepted standards [C-3A(2)]; and does not take into account the effect of naturally occurring characteristics and other factors.

2.1.11 *Glue* — A substance capable of holding materials together by surface attachment.

2.1.12 *Glued Laminated* (*Glulam*) — A wood structural element comprising of sawn wood lamination bonded together with adhesives and in which the grains of all laminations are parallel longitudinally.

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2.1.12.1 *Glued laminated*, *horizontally* — Glued laminated timber where gluing planes are perpendicular to the largest dimension of the transverse section, that is, in cross-wise direction.

2.1.12.2 *Glued laminated*, *vertically* — Glued laminated timber where gluing planes are perpendicular to the smallest dimension of the transverse section.

2.1.13 *Inside Location* — Position in buildings in which timber remains continuously dry or protected from weather.

2.1.14 *Isoptera* — An order of social insects consisting of the termites.

2.1.15 *Lamella Roof* — A roof or frame, made up of relatively short members, called lamellas, forming a network of framing timbers. This network forms a barrel-like structure of mutually braced and stiffened units, arching over the building between supports, and together with the sheathing forms a diaphragm for resistance of vertical and lateral loads.

2.1.16 *Laminations* — Thin elements of wood of appreciable width and length which, when superimposed on each other and fastened together with their grains essentially parallel make up cross section of considerable depth or width.

2.1.17 *Laminated Veneer Lumber* — A structural composite made by laminating veneers, 1.5 mm to 4.2 mm thick, with suitable adhesive and with the grain of veneers in successive layers aligned along the longitudinal (length) dimension of the composite.

2.1.18 *Latewood* — The portion of the wood that is formed in the later part of growth season and consists of thicker walled cells.

2.1.19 *Loaded Edge Distance* — The distance measured from the centre to the edge towards which the load induced by the connector acts, and the unloaded edge distance is the one opposite to the loaded edge.

2.1.20 *Location* — A term generally referred to as exact place where a timber is used in building.

2.1.21 *Mechanically Laminated* — A wood structural element comprising of laminations which are not glued but which are held together with mechanical fastenings such as nails or bolts and in which all laminations have the grains approximately parallel longitudinally.

2.1.22 *Outside Location* — Position in buildings in which timbers are occasionally subjected to wetting and drying as in the case of open sheds and outdoor exposed structures.

2.1.23 *Permissible Stress* — Stress obtained by applying factor of safety to the ultimate stress.

2.1.24 *Sandwich*, *Structural* — A layered construction comprising a combination or relatively high-strength facing material intimately bonded to and acting integrally with a low density core material.

2.1.25 *Spaced Column* — Two column sections adequately connected together by glue, bolts, screws or otherwise.

2.1.26 *Stressed Skin* — A form of construction in which the outer skin, in addition to its normal functioning of providing a surface covering, acts integrally with the framing members contributing to the strength of the unit as a whole.

2.1.27 *Structure*, *Permanent* — Structural units in timber which are constructed for a long duration and wherein adequate protection and design measures have initially been incorporated to render the structure serviceable for the required life.

2.1.28 *Structure*, *Temporary* — Structures which are erected for a short period, such as hutments at project sites, for rehabilitation, temporary defence constructions, exhibition structures, etc.

2.1.29 *Structural Element* — The component timber members and joints which make up a resulting structural assembly.

2.1.30 *Structural Grades* — Grades defining the maximum size of strength reducing natural characteristics (knots, sloping grain, etc) deemed permissible in any piece of structural timber within designated structural grade classification.

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2.1.31 *Structural Timber* — Timber in which strength is related to the anticipated in-service use as a controlling factor in grading and selection and/or stiffness.

2.1.32 *Stud* — One of a series of slender wood structural members placed vertically as supporting members in a wall partition, or similar structural unit.

2.1.33 *Termite* — An insect of the order *Isoptera* which may burrow in the wood or wood products of a building for food or shelter.

2.1.34 *Wet Location* — Position in buildings in which timbers are almost continuously damp or wet in contact with the earth or water, such as piles and timber foundations.

2.2 Definitions of Defects in Timber

2.2.1 *Check* — A separation of fibres extending along the grain which is confined to one face of a piece of wood.

2.2.2 *Compression Wood* — Abnormal wood which is formed on the lower sides of branches and inclined stems of coniferous trees. It is darker and harder than normal wood but relatively low in strength for its weight. It can be usually identified by wide eccentric growth rings with abnormally high proportion of growth latewood.

2.2.3 *Dead Knot* — A knot in which the layers of annual growth are not completely intergrown with those of the adjacent wood. It is surrounded by pitch or bark. The encasement may be partial or complete.

2.2.4 *Decay or Rot* — Disintegration of wood tissue caused by fungi (wood destroying) or other micro-organisms.

2.2.5 *Decayed Knot* — A knot softer than the surrounding wood and containing decay.

2.2.6 *Diameter of Knot* — The maximum distance between the two points farthest apart on the periphery of a round knot, on the face on which it becomes visible. In the case of a spike or a splay knot, the maximum width of the knot visible on the face on which it appears shall be taken as its diameter.

2.2.7 *Discolouration* — A change from the normal colour of the wood which does not impair the strength of the wood.

2.2.8 *Knot* — A branch base or limb embedded in the tree or timber by natural growth.

2.2.9 *Knot Hole* — A hole left as a result of the removal of a knot.

2.2.10 *Live Knot* — A knot free from decay and other defects, in which the fibres are firmly intergrown with those of the surrounding wood. (*Synonyms:* 'Intergrown knot'; *cf.* 'Dead knot')

2.2.11 *Loose Grain* (*Loosened Grain*) — A defect on a flat sawn surface caused by the separation or raising of wood fibres along the growth rings; *cf.* 'Raised grain'.

2.2.12 *Loose Knot* — A knot that is not held firmly in place by growth or position, and that cannot be relied upon to remain in place; *cf.* 'Tight knot'.

2.2.13 *Mould* — A soft vegetative growth that forms on wood in damp, stagnant atmosphere. It is the least harmful type of fungus, usually confined to the surface of the wood.

2.2.14 *Pitch Pocket* — Accumulation of resin between growth rings of coniferous wood as seen on the cross-section.

2.2.15 *Sap Stain* — Discolouration of the sapwood mainly due to fungi.

2.2.16 *Sapwood* — The outer layer of log, which in the growing tree contain living cells and food material. The sapwood is usually lighter in colour and is readily attacked by insects and fungi.

2.2.17 *Shake* — A partial or complete separation between adjoining layers of tissues as seen in end surfaces.

2.2.18 *Slope of Grain* — The inclination of the fibres to the longitudinal axis of the member.

2.2.19 *Sound Knot* — A tight knot free from decay, which is solid across its face and at least as hard as the surrounding wood.

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2.2.20 *Split* — A crack extending from one face of a piece of wood to another and runs along the grain of the piece.

2.2.21 *Tight Knot* — A knot so held by growth or position as to remain firm in position in the piece of wood; *cf.* 'Loose knot'.

2.2.22 *Wane* — The original rounded surface of a tree remaining on a piece of converted timber.

2.2.23 *Warp* — A deviation in sawn timber from a true plane surface or distortion due to stresses causing departure from a true plane.

2.2.24 *Worm Holes* — Cavities caused by worms.

3 SYMBOLS

For the purpose of this Subsection, the following letter symbols shall have the meaning indicated against each:

a	=	Projected area of bolt in main member (t' x d_3), mm^2
B	=	Width of the beam, mm
C	=	Concentrated load, N
D	=	Depth of beam, mm
D_1	=	Depth of beam at the notch, mm
D_2	=	Depth of notch, mm
d	=	Dimension of least side of column, mm
d_1	=	Least overall width of box column, mm
d_2	=	Least overall dimension of core in box column, mm
d_3	=	Diameter of bolt, mm
d_f	=	Bolt-diameter factor
E	=	Modulus of elasticity in bending, N/mm^2
e	=	Length of the notch measured along the beam span from the inner edge of the support to the farthest edge of the notch, mm
F	=	Load acting on a bolt at an angle to grain, N
f_{ab}	=	Calculated bending stress in extreme fibre, N/mm^2
f_{ac}	=	Calculated average axial compressive stress, N/mm^2
f_{at}	=	Calculated axial tensile stress, N/mm^2
f_b	=	Permissible bending stress on the extreme fibre, N/mm^2
f_c	=	Permissible stress in axial compression, N/mm^2
f_{cn}	=	Permissible stress in compression normal (perpendicular) to grain, N/mm^2
f_{cp}	=	Permissible stress in compression parallel to grain, N/mm^2
$f_{c\theta}$	=	Permissible compressive stress in the direction of the line of action of the load, N/mm^2
f_t	=	Permissible stress in tension parallel to grain, N/mm^2 [extreme fibre stress in bending-tension (f_b) is safe practical estimate of tensile strength due to mode of failure in beams]
H	=	Horizontal shear stress, N/mm^2
I	=	Moment of inertia of a section, mm^4
K	=	Coefficient in deflection depending upon type and criticality of loading on beam
K_1	=	Modification factor for change in slope of grain
K_2	=	Modification factor for change in duration of loadings

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K_3, K_4, K_5, K_6	=	Form factors
K_7	=	Modification factor for bearing stress
K_8	=	Constant equal to $0.584\sqrt{\frac{E}{f_{cp}}}$
K_9	=	Constant equal to $\frac{\pi}{2}\sqrt{\frac{UE}{5q\,f_{cp}}}$
K_{10}	=	Constant equal to $0.584\sqrt{\frac{2.5E}{f_{cp}}}$
l or L	=	Span of a beam or truss, mm
M	=	Maximum bending moment in beam, N-mm
N	=	Total number of bolts in the joint
n	=	Shank diameter of the nail, mm
P	=	Load on bolt parallel to grain, N
p_1	=	Ratio of the thickness of the compression flange to the depth of the beam
Q	=	Static moment of area above or below the neutral axis about neutral axis, mm^3
q	=	Constant for particular thickness of plank
q_1	=	Ratio of the total thickness of web or webs to the overall width of the beam
R	=	Load on bolt perpendicular (normal) to grain, N
S	=	Unsupported overall length of column, mm
t	=	Nominal thickness of planks used in forming box type column, mm
t'	=	Thickness of main member, mm
U	=	Constant for a particular thickness of the plank
V	=	Vertical end reaction or shear at a section, N
W	=	Total uniform load, N
x	=	Distance from reaction to load, mm
y	=	Factor determining the value of form factor K_4
δ	=	Deflection at middle of beam, mm
θ	=	Angle of load to grain direction
Z	=	Section modulus of beam, mm^3
λ_1	=	Percentage factor for t'/d_3 ratio, parallel to grain
λ_2	=	Percentage factor for t'/d_3 ratio, perpendicular to grain

4 MATERIALS

4.1 Species of Timber

The species of timber recommended for structural purposes are given in Table 1 along with their permissible stresses and properties. For information on mechanical properties of certain foreign timber species, reference may be made to Annex A.

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Table 1 Safe Permissible Stresses for the Species of Timber

[*Clauses* 4.1, 4.2, 4.7.2(b), 5.4.1, 5.4.2, 5.4.2.2, 6.5.8.3.1(b), 10.2.3 *and* 15.3]

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content Kg/m^3	Modulus of Elasticity (All Grades and All Locations) $\times 10^3$ N/mm^2	Permissible Stress for Grade I N/mm^2												Preservative Characters		Refracteriness to Air Seasoning[3)]
	Botanical Name	Trade Name				Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1)]	Treat-ability Grade[2)]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
i)	**GROUP A:**																		
	a) *Acacia catectou*	Khair (KHA)	Uttar Pradesh	1 009	13.44	20.1	16.8	13.4	1.6	2.2	13.8	12.3	10.1	7.7	6.0	4.9	I	–	A
	b) *Acacia chundra*	Red kutch	Madhya Pradesh	1 086	16.79	26.5	22.0	17.6	2.2	3.2	17.9	15.9	13.0	10.9	8.4	6.9	–	–	A
	c) *Albizia odoratissima*	Kala siris (KSI)	Chennai	737	13.54	18.7	15.6	12.5	1.5	2.2	13.3	11.8	9.6	7.3	5.6	4.6	I	e	B
	d) *Bruguiera spp.*	Bruguiera (BSV) (Mangrove)	Andaman	897	17.68	21.9	18.3	14.6	1.2	1.7	14.3	12.7	10.4	5.5	4.3	3.5	III	–	–
	e) *Grewia tiliifolia*	Dhaman (DHA)	Chennai	788	14.82	18.3	15.2	12.2	1.3	1.9	12.0	10.7	8.7	6.0	4.7	3.8	III	d	B
	f) *Hopea utilis* (*Balano carpus utilis*)	Karung	Chennai	987	16.91	25.1	20.9	16.7	1.5	2.2	16.4	14.6	11.9	9.3	7.3	5.9	–	–	–

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) × 10^3	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3]
	Botanical Name	Trade Name		Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1]	Treat-ability Grade[2]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	g) *Hopea glabra*	Hopea (HOP)	Chennai	1 081	14.79	21.3	17.8	14.2	1.5	2.2	14.5	12.9	10.6	9.9	7.7	6.3	I	–	A
	h) *Hopea parviflora*	Hopea (HOP)	Chennai	923	13.03	18.6	15.5	12.4	1.3	1.8	13.2	11.8	9.6	9.2	7.3	6.0	I	e	A
	j) *Manilota polyandra* (*Syn. Cynometra polyandra*)	Ping (PIG)	Assam	903	13.20	19.1	15.9	12.7	1.3	1.8	1.2	10.4	8.5	5.7	4.4	3.6	III	b	A
	k) *Mesua ferrea*	Mesua (MES)	Assam	965	16.30	23.3	19.4	15.5	1.2	1.8	15.5	13.8	11.3	5.9	4.6	3.7	I	–	A
	m) *Mimusops littoralis*	Bullet-wood (BUL)	Southern Andaman	1 103	17.39	22.7	18.9	15.1	1.5	2.1	14.2	12.7	10.4	11.3	8.8	7.2	I	–	A
	n) *Pesciloneuron indicum*	Ballagi (BAL)	Chennai	1 139	16.29	22.4	18.7	15.0	1.5	2.2	14.7	13.1	10.7	8.7	6.8	5.5	I	e	A
	p) *Pterocarpus Scantalinus*	Red sanders (MA)	Chennai	1 121	12.73	25.0	20.9	16.7	1.7	2.5	18.1	16.1	13.2	11.8	9.2	7.5	–	–	A
	q) *Sageraea elliptica*	Chooi (COC)	Andaman	869	15.06	21.5	17.9	14.3	1.1	1.5	12.5	11.1	9.1	5.3	4.1	3.4	–	–	A

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content Kg/m^3	Modulus of Elasticity (All Grades and All Locations) × 10^3 N/mm^2	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3]
	Botanical Name	Trade Name				Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1]	Treat-ability Grade[2]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	r) *Stereospermun celonoides*	Padri (PAD)	Chennai	731	12.94	19.0	15.8	12.7	1.1	1.6	11.9	10.6	8.7	4.0	3.1	2.6	III	–	B
	s) *Vitex altissima*	Milla (MIL)	Maharashtra	937	13.01	18.2	15.2	12.1	1.2	1.7	12.6	11.2	9.2	9.5	7.4	6.1	I	e	A
ii)	**GROUP B:**																		
	a) *Albizzia lebbeck*	Kokko (KOK)	Andaman	642	11.17	13.4	11.2	9.0	1.1	1.5	9.0	8.0	6.5	4.4	3.4	2.8	I	e	B
	b) *Anogeissus latifolia*	Dhaura, Axle wood (AXL) (Bakli)	Uttar Pradesh	892	10.55	16.1	13.4	10.7	1.1	1.6	9.1	8.1	6.6	4.7	3.7	3.0	I	e	A
	c) *Artocarpus hirsulus*	Aini (AIH)	Chennai	600	10.45	15.0	12.5	10.0	0.7	1.1	10.4	9.2	7.5	3.3	2.6	2.1	I	–	B
	d) *Acacia nilotica*	Babul (BAB)	Uttar Pradesh	797	-	-	12.9	10.3	1.4	2.1	8.9	7.9	6.4	5.2	4.0	3.3	I	b	B
	e) *Acacia ferruginea*	Safed khair	Maharashtra	993	12.28	23.0	19.2	15.3	1.7	2.4	13.9	12.4	10.1	9.9	7.7	6.3	–	–	–

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) × 10^3	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3)]
	Botanical Name	Trade Name		Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1)]	Treat-ability Grade[2)]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	f) *Acrocarpus fraxinifolius*	Mundani (MUN)	Chennai	690	12.59	16.1	13.4	10.8	1.2	1.8	10.5	9.4	7.7	4.6	3.6	2.9	III	c	B
	g) *Aglaia odulis*	Aglaia (AGL)	Assam	815	12.56	18.2	15.2	12.1	1.4	2.0	10.1	8.9	7.3	4.4	3.4	2.8	–	–	A
	h) *Anogeissus acuminota*	Yon	Orissa	844	11.67	17.6	14.7	11.7	1.3	1.8	10.8	9.6	7.9	5.1	4.0	3.3	–	–	A
	j) *Atalanlia monophylla*	Jungli-nimbu (JHI)	Orissa	897	10.31	16.7	13.9	11.1	1.5	2.1	11.3	10.0	8.2	6.3	4.9	4.0	–	–	–
	k) *Altingia excelsa*	Jutili (JUT)	Assam	795	11.37	17.1	14.3	11.4	1.2	1.8	11.0	9.8	8.0	6.8	5.3	4.4	II	e	A
	m) *Amoora spp.*	Amari (AMA)	West Bengal	625	1.05	13.4	1.1	9.2	0.9	1.3	8.4	7.4	6.0	3.7	2.9	2.4	II	d	B
	n) *Bucklandia populnea* (*Syn. Exbucklandia populnea*)	Pipli (PIP)	West Bengal	672	9.89	12.8	10.7	8.6	1.1	1.5	7.9	7.0	5.7	3.5	2.7	2.2	III	e	C
	p) *Cassia fistula*	Amaltas (AMT)	Uttar Pradesh	865	11.80	19.2	16.0	12.8	1.4	2.0	12.3	10.9	8.9	7.2	5.6	4.6	I	–	A

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) × 10^3	Permissible Stress for Grade I N/mm^2												Preservative Characters		Refracteriness to Air Seasoning[3]
	Botanical Name	Trade Name		Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain				Durability Class[1]	Treat-ability Grade[2]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)		(18)	(19)	(20)
	q) *Carallia lucida*	Maniawaga	Assam	748	12.60	18.4	15.3	12.3	1.2	1.7	11.4	10.1	8.3	5.9	4.6	3.8		–	–	–
	r) *Canarium strictum*	Dhup	Chennai	655	11.86	13.3	11.1	8.9	0.9	1.2	8.1	7.2	5.9	2.8	2.2	1.8		III	–	C
	s) *Cassia sienea*	Kasod	Madhya Pradesh	820	10.50	15.4	12.8	10.9	1.0	1.4	10.8	9.6	7.9	5.5	4.3	3.5		–	–	-
	t) *Casuerina equisetifolia*	Casuarina (CAS)	Orissa	769	11.44	14.6	12.2	9.8	1.3	1.8	8.2	7.3	5.9	4.0	3.1	2.5		III	e	A
	u) *Celophyllum temculosum*	Poon (POO)	Maharashtra	657	9.77	13.4	11.2	9.0	0.8	1.1	8.6	7.7	6.3	2.8	2.2	1.8		II	–	B
	v) *Chloroxylon swielenia*	Satin wood (CFI)	Madhya Pradesh	865	11.69	18.2	15.1	12.1	1.4	2.0	10.9	9.7	8.0	6.3	4.9	4.0		III	–	A
	w) *Cullenia resayoana* (*Syn. C. execelsa*)	Karani (KAP)	Chennai	625	12.43	14.7	12.3	9.8	0.6	0.9	9.0	8.0	6.6	2.7	2.1	1.7		III	b	C
	y) *Diploknema butyracea* (*Syn. Bassia butyrance*)	Hill mahua (HMA)	Southern Andaman	780	10.64	15.3	12.8	10.2	1.0	1.5	9.9	8.8	7.2	6.6	5.2	4.2		–	–	–
	z) *Dyscxylum malebaricum*	White ceda (WCE)	Chennai	745	10.92	13.2	11.0	8.8	1.0	1.4	8.0	7.1	5.8	3.1	2.4	1.9		I	–	B

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content Kg/m³	Modulus of Elasticity (All Grades and All Locations) × 10³ N/mm²	Permissible Stress for Grade I N/mm²											Preservative Characters		Refracteriness to Air Seasoning[3]
	Botanical Name	Trade Name				Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1]	Treat-ability Grade[2]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	aa) *Dipterocarpus grandiflorus*	Gurjan (GUR)	Northern Andaman	758	11.71	12.5	10.5	8.4	0.8	1.1	7.9	7.1	5.8	2.7	2.1	1.7	I	–	B
	ab) *Dipterocarpus macrocarpus*	Hollong (HOL)	Assam	726	13.34	14.5	12.0	9.6	0.8	1.1	8.8	7.9	6.4	3.5	2.7	2.2	III	a	B
	ac) *Dichopsis polyantha* (*Syn. Palaquium polyanthum*)	Tali (TAL)	Assam	734	11.24	14.9	12.4	10.0	1.1	1.6	9.9	8.8	7.2	4.7	3.7	3.0	–	–	B
	ad) *Dichopsis elliptica* (*Syn. Palaquium ellipticum*)	Pali (PAL)	Chennai	606	11.86	13.9	11.6	9.3	0.7	1.0	8.5	7.5	6.2	2.9	2.2	1.8	I	e	B
	ae) *Diospyros micropylla*	Ebony (EBO)	Maharashtra	776	12.15	14.2	11.9	9.5	0.9	1.3	8.3	7.3	6.0	3.3	2.6	2.1	–	–	A
	af) *Diospyros pyrrhocarpus*	Ebony (EBO)	Northern Andaman	843	9.93	13.5	11.2	9.0	1.0	1.4	7.9	7.0	5.7	4.0	3.1	2.5	III	–	A
	ag) *Dipterocarpus bourdilloni*	Gurjan (GUR)	Kerala	699	12.71	13.6	11.3	9.0	0.7	1.0	7.8	6.9	5.7	2.5	1.9	1.6	–	–	B

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content Kg/m^3	Modulus of Elasticity (All Grades and All Locations) × 10^3 N/mm^2	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3]
	Botanical Name	Trade Name				Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1]	Treat-ability Grade[2]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	ah) *Eucalyptus globulus*	Eucalyptus (Blue gum) (BLN)	Chennai	912	14.83	15.9	13.2	10.6	10.3	1.5	9.0	8.0	6.5	3.4	2.6	2.1	I	e	A
	aj) *Eucalyptus ougenioides*	Eucalyptus	Chennai	853	11.47	16.4	13.6	10.9	1.2	1.7	11.3	10.0	8.2	7.6	5.9	4.8	–	–	–
	ak) *Eugenia gardnery*	Jaman (JAM)	Chennai	952	11.94	14.8	12.3	9.8	1.1	1.6	9.2	8.2	6.7	5.8	4.5	3.7	III	d	–
	am) *Eugenia jambolana*	Jaman (JAM)	Uttar Pradesh	778	10.94	16.0	13.3	10.6	1.2	1.7	9.7	8.6	7.1	4.7	3.7	3.0	–	–	–
	an) *Gluta travancorice*	Gluta (GLU)	Chennai	726	12.73	13.5	11.3	9.0	0.9	1.3	9.0	8.0	6.6	4.0	3.1	2.5	I	–	A
	ap) *Grewia veslita*	Dhaman (DHA)	West Bengal	758	12.00	15.4	12.6	10.3	1.4	2.0	9.1	8.1	6.6	4.1	3.2	2.6	III	d	B
	aq) *Heritiera spp.*	Sundri (SUN)	Assam	872	13.37	17.9	14.9	11.9	1.3	1.8	11.0	9.8	8.0	6.5	5.0	4.1	I	–	A
	ar) *Kingiodendron pinnatum* (*Syn. Hardwickia pinnata*)	Piney (PIN)	Chennai	617	10.62	13.2	11.0	8.8	0.4	1.3	8.2	7.3	6.0	2.9	2.2	1.8	–	–	B

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) $\times 10^3$	Permissible Stress for Grade I N/mm^2												Preservative Characters		Refracteriness to Air Seasoning[3]
	Botanical Name	Trade Name		Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain				Durability Class[1]	Treat-ability Grade[2]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)		(18)	(19)	(20)
	as) *Kayea floribund*	Karal	Assam	813	10.88	16.8	14.0	1.1	1.1	1.6	10.1	9.0	7.3	4.4	3.4	2.8		III	–	–
	at) *Lagerstromia lanceolata*	Benteak (BEN)	Chennai	617	10.76	12.7	10.6	8.5	0.8	1.2	8.2	7.3	5.9	3.4	2.6	2.2		I	e	B
	au) *Lagerstromia parviflora*	Lendi (LEN)	Uttar Pradesh	734	10.97	14.3	11.9	9.5	1.1	1.6	8.7	7.7	6.3	3.7	2.9	2.4		I	e	A
	av) *Mimusops elengi*	Bakul (BKL)	Chennai	885	12.39	17.3	14.4	11.5	1.3	1.8	11.0	9.8	8.0	5.6	4.3	3.6		I	–	A
	aw) *Machilus macrantha*	Machilus (MAC)	West Bengal	692	10.00	12.4	10.3	8.3	1.0	1.5	8.2	7.3	6.0	3.5	2.7	2.2		III	e	B/C
	ay) *Miliuse tyomentosa* (*Syn. Saccopapetalum tomentosum*)	Hoom (HOO)	Maharashtra	745	11.06	14.8	12.3	9.9	0.9	1.3	9.7	8.6	7.0	3.5	2.7	2.2		III	–	B
	az) *Pommetia pinnata*	–	Andaman	788	12.90	14.3	11.9	9.5	1.1	1.6	9.1	8.0	6.6	4.0	3.1	2.5		–	–	–
	ba) *Pterocarpus dolbergioides*	Padauk (PAD)	Northern Andaman	721	11.24	17.1	14.3	11.4	1.0	1.5	12.0	10.7	8.7	5.5	4.3	3.5		I	c	B
	bb) *Mesua assamica*	Kayea	Assam	842	12.83	17.4	14.5	11.6	1.0	1.4	11.7	10.4	8.5	5.3	4.1	3.3		II	e	–

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) × 10^3	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3)]
	Botanical Name	Trade Name		Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1)]	Treat-ability Grade[2)]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	bc) *Pterocarpus marsupium*	Bijasal (BIJ)	Maharashtra	803	10.25	14.9	12.4	9.9	0.9	1.3	9.1	8.1	6.6	4.1	3.2	2.6	I	e	B
	bd) *Fraxlnus macrantha*	Ash (ASH)	Uttar Pradesh	712	10.69	15.0	12.5	10.0	1.2	1.7	8.5	7.6	6.2	4.3	3.3	2.7	III	–	B
	be) *Fraxlnus execlsior*	Ash (ASH)	Punjab	719	10.41	14.8	12.3	9.8	1.2	1.7	8.1	7.2	5.8	3.3	2.6	2.1	III	–	B
	bf) *Planchonia valida* (*Syn. P. andamanica*)	Red bombwe (RBO)	Andaman	913	13.10	16.1	13.4	10.7	1.0	1.4	10.8	9.6	7.9	4.9	3.8	3.1	III	–	–
	bg) *Quercus lamellosa*	Oak	West Bengal	87	12.44	14.5	12.1	9.7	1.2	1.7	8.7	7.8	6.4	3.8	2.9	2.4	II	c	A
	bh) *Quercus griffithii*	Oak	Meghalaya	974	10.06	13.1	10.9	8.8	1.1	1.6	8.0	7.1	5.8	4.6	3.6	2.9	–	–	A
	bj) *Quercus incane*	Oak	Punjab	1 008	10.82	15.8	13.1	10.5	1.2	1.8	8.7	7.8	6.3	5.0	3.9	3.2	–	–	A
	bk) *Quercus lineate*	Oak	West Bengal	874	12.63	15.2	12.7	10.1	1.2	1.7	9.6	8.6	7.0	5.3	4.1	3.4	II	c	A

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content Kg/m^3	Modulus of Elasticity (All Grades and All Locations) × 10^3 N/mm^2	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3]
	Botanical Name	Trade Name				Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1]	Treat-ability Grade[2]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	bm) *Quercus semecarpifolia*	–	Punjab	834	11.58	15.8	13.1	10.5	1.3	1.8	8.3	7.3	6.0	3.8	2.9	2.4	–	–	A
	bn) *Shorea robusta*[4]	Sal (SAL)	Madhya Pradesh	805	12.67	16.9	14.0	11.2	0.9	1.3	10.6	9.4	7.7	4.6	3.5	2.9	I	e	A
	bp) *Soymida fabrifuga*	Rohini (ROH)	Chennai	1 116	12.22	21.5	17.9	14.4	1.6	2.3	15.0	13.3	10.9	12.9	10. 0	8.2	I	–	A
	bq) *Shorea talura*	–	Maharashtra	721	12.20	16.8	14.0	11.2	1.1	1.6	12.6	11.2	9.2	6.8	5.3	4.3	–	–	–
	br) *Plerygota alata* (*Syn. Sterculia alata*)	Narikel (NAR)	Assam	593	10.95	13.4	11.8	8.9	0.8	1.2	8.2	7.3	6.0	2.7	2.1	1.7	III	–	C
	bs) *Syzygium cumini*	Jaman (JAM)	Assam	841	10.55	14.8	12.4	9.9	1.1	1.6	9.0	8.0	6.5	6.9	5.4	4.4	II	e	A
	bt) *Terminalia bellirica*	Bahera (BAH)	Uttar Pradesh	729	10.19	13.6	11.3	9.0	1.0	1.4	8.4	.7.5	6.1	3.7	2.8	2.3	III	b	B
	bu) *Terminalia chebula*	Myrobalan (MYR)	–	918	12.37	17.1	14.2	11.4	1.1	1.6	1.2	10.4	8.5	6.7	5.2	4.3	II	c	A

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) × 10^3	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3)]
	Botanical Name	Trade Name		Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1)]	Treat-ability Grade[2)]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	bv) *Terminalia citrina*	–	Assam	755	11.89	17.1	14.3	11.4	1.1	1.6	10.8	9.6	7.9	5.0	3.9	3.2	–	–	–
	bw) *Terminalia manii*	Black-chuglam (BCH)	Southern Andaman	822	12.66	16.8	14.0	11.2	1.1	1.6	10.3	9.2	7.5	5.1	4.0	3.2	II	a	B
	by) *Tectona grandis*	Teak (TEA)	Uttar Pradesh	660	9.97	15.5	12.9	10.3	1.2	1.6	9.4	8.3	6.8	4.5	3.5	2.8	I	e	B
	bz) *Terminalia paniculate*	Kindal (KIN)	Maharashtra	765	10.57	13.1	10.9	8.7	0.9	1.3	8.6	7.7	6.3	3.6	2.8	2.3	I	c	A
	ca) *Alreminalia alata*	Laurel (LAU), Sain	Chennai	906	10.54	15.1	12.5	10.0	1.1	1.6	9.4	8.4	6.8	6.2	4.8	4.0	I	b	A
	cb) *Terminalia bilata*	White-chuglam (WCH)	Southern Andaman	690	12.38	15.5	13.0	10.4	0.9	1.2	9.8	8.7	7.1	3.6	2.8	2.3	III	e	B
	cc) *Thespesia populnea*	Bhendi (BHE)	Maharashtra	766	10.36	18.9	15.8	12.6	1.3	1.9	11.3	10.0	8.2	4.4	3.4	2.8	–	–	B
	cd) *Xylia xylocarpa*	Irul (IRU)	Maharashtra	839	11.63	16.2	13.5	10.8	1.3	1.8	10.9	9.7	7.9	7.8	6.0	4.9	I	e	A

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) × 10^3	Permissible Stress for Grade I N/mm^2												Preservative Characters		Refracteriness to Air Seasoning[3)]
				Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain				Durability Class[1)]	Treat-ability Grade[2)]	
	Botanical Name	Trade Name				Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	
	ce) *Zanthoxylum budranga*	Mullilam (MUL)	West Bengal	587	10.65	14.7	12.2	9.8	0.9	1.2	9.5	8.4	6.9	3.4	2.6	2.1	I	e	B	
	cf) *Adina oligocephala*	-	Arunachal	715	11.17	15.2	12.7	10.1	1.2	1.7	10.3	9.2	7.5	4.0	3.1	2.4	–	–	–	
	cg) *Castanopsis indica*	Chestnut	Meghalaya	688	12.54	14.8	12.3	9.9	1.0	1.4	9.8	8.7	7.1	3.4	2.7	2.2	–	–	B	
	ch) *Eucalyptus citriodara*	Eucalyptus	Nilgiri	831	12.12	17.3	14.4	11.5	1.4	2.0	11.0	9.8	8.0	4.2	3.3	2.7	–	–	–	
	cj) *Eucalyptus citriodata*	Eucalyptus	Ooty	725	9.35	15.4	12.9	10.3	1.0	1.4	8.6	7.6	6.3	3.0	2.4	2.0	–	–	–	
	ck) *Eucalytus tereticornis*	Eucalyptus	Chennai	777	11.05	16.7	13.9	11.1	1.0	1.4	9.7	8.6	7.1	3.4	2.6	2.2	III	e	–	
iii)	**GROUP C:**																			
	a) *Tbizia procera*	White siris	Uttar Pradesh	643	9.02	13.4	11.2	8.9	1.0	1.4	8.5	7.6	6.2	4.3	3.3	2.7	I	c	B	
	b) *Artocarpus lakocha*	Lakooch (LAK)	Uttar Pradesh	647	6.14	10.0	8.3	6.7	1.0	1.4	5.3	4.7	3.8	2.8	2.2	1.8	I	–	B	

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content Kg/m^3	Modulus of Elasticity (All Grades and All Locations) × 10^3 N/mm^2	Permissible Stress for Grade I N/mm^2												Preservative Characters		Refracteriness to Air Seasoning[3)]
	Botanical Name	Trade Name				Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain				Durability Class[1)]	Treat-ability Grade[2)]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)		(18)	(19)	(20)
	c) *Artocarpus hetarophyllus* (*Syn. A. Integrifolia*)	Jack, kathal (KAT)	Chennai	617	9.46	13.9	11.6	9.2	1.0	1.5	9.3	8.3	6.8	4.5	3.5	2.9		I	d	B
	d) *Aphanamixis polystachya* (*Syn. Amoora rehituka*)	Pitraj (PIT)	West Bengal	668	8.98	12.3	10.2	8.2	1.1	1.5	8.0	7.1	5.8	4.0	3.1	2.6		I	–	B
	e) *Adina cordifolia*[4)]	Haldu (HAL)	Uttar Pradesh	663	8.54	13.3	11.1	8.9	1.0	1.4	8.7	7.7	6.3	4.4	3.4	2.8		III	a	B
	f) *Anthocephyalus chinensis* (*Syn. A. Cadamba)*	Kadam (KAD)	–	485	1.88	9.7	8.1	5.4	0.7	1.0	5.9	5.3	4.3	1.9	1.5	1.2		III	a	–
	g) *Arlocarpus chaplasha*	Chaplash (CHP)	Assam	515	9.11	13.2	11.0	8.8	0.9	1.2	8.5	7.5	6.2	3.6	2.8	2.3		III	d	B
	h) *Acacia leucophloea*	Hiwar (HIW)	Madhya Pradesh	737	7.85	13.4	11.2	9.0	1.0	1.5	7.5	6.7	5.4	4.5	3.5	2.8		–	–	A
	j) *Acacia melanoxylone*	Black wood	Chennai	630	9.45	13.0	10.8	8.7	1.1	1.5	7.6	6.8	5.5	3.2	2.5	2.0		–	–	–

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content Kg/m^3	Modulus of Elasticity (All Grades and All Locations) × 10^3 N/mm^2	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3]
	Botanical Name	Trade Name				Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1]	Treat-ability Grade[2]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	k) *Acacia mearnsii* (*Syn. A. mollissima*)	Black wattle	Chennai	669	6.10	10.4	8.6	6.9	0.8	1.2	6.0	5.4	4.4	2.3	1.8	1.5	–	–	–
	m) *Accer spp.*	Maple (MAP)	Punjab, Uttar Pradesh	551	7.35	9.9	8.2	6.5	0.9	1.3	5.5	4.9	4.0	2.1	1.7	1.4	III	–	B
	n) *Aegla marmalos* (*Syn. Intsia bijuga*)	Bael (BEL)	Uttar Pradesh	890	8.81	13.5	11.2	9.0	1.4	2.0	8.8	7.8	6.4	6.8	5.3	4.3	III	–	B
	p) *Afzelia bijuga*	-	Andaman	705	9.16	13.2	11.0	8.8	1.1	1.5	7.9	7.1	5.8	4.0	3.1	2.6	–	–	–
	q) *Ailanthus grandis*	Gokul (GOK)	West Bengal	404	7.94	8.3	6.9	5.5	0.6	0.8	5.3	4.7	3.9	1.1	0.9	0.7	III	–	C
	r) *Anogeissus pendula*	Kardhai (KAH)	Uttar Pradesh	929	9.75	17.0	14.2	11.4	1.3	1.8	9.8	8.7	7.1	6.5	5.1	4.2	III	–	A
	s) *Areca nut*	-	Kerala	833	9.48	15.2	12.7	10.2	1.2	1.6	10.8	9.6	7.8	7.3	5.7	4.7	–	–	–
	t) *Albizia lucida*	-	Arunachal Pradesh	566	8.51	10.7	8.9	7.1	8.2	1.2	7.3	6.3	5.3	2.3	1.8	1.5	–	–	–
	u) *Azadirachta indica*	Neem (NEE)	Uttar Pradesh	836	8.52	14.6	12.1	9.7	1.3	1.8	10.0	8.9	7.3	5.0	3.9	3.2	–	–	–

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) $\times 10^3$	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3]
	Botanical Name	Trade Name		Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1]	Treat-ability Grade[2]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	v) *Boswellia seriata*	Salai (SAA)	Bihar	551	7.21	9.4	7.9	6.3	0.7	1.1	5.5	4.9	4.0	2.1	1.6	1.3	I	e	C
	w) *Bridelia retusa*	Kassi (KAS)	Bihar	584	9.42	11.6	9.7	7.7	0.9	1.3	7.1	6.3	5.1	4.0	3.1	2.6	I	e	B
	y) *Betula lnoides*	Birch (BIR)	West Bengal	625	9.23	9.6	8.0	6.4	0.8	1.1	5.7	5.0	4.1	2.2	1.7	1.4	–	–	B
	z) *Bischofia javanica*	Uriam Bishopwood (URI)	Chennai	769	8.84	9.6	8.2	6.5	0.8	1.1	5.9	5.3	4.3	3.6	2.8	2.3	III	–	A
	aa) *Burserra serrata* (*Syn. Protium serratum*)	Muntenga (MUR)	Arunachal Pradesh	756	1.17	15.5	13.3	10.5	0.9	1.3	10.1	9.0	7.4	5.3	4.1	3.4	II	c	–
	ab) *Careya arbersa*	Kumbi (KUM)	Uttar Pradesh	889	8.37	13.1	10.9	8.8	1.0	1.5	7.7	6.8	5.6	5.3	4.1	3.4	I	e	A
	ac) *Cedrus deodara*	Deodar (DEO)	Himachal Pradesh	557	9.48	10.2	8.7	7.2	0.7	1.0	7.8	6.9	5.7	2.7	2.1	1.7	I	c	C
	ad) *Cupressus torulosa*	Cypress (CYP)	Uttar Pradesh	506	8.41	8.8	7.6	6.2	0.6	0.8	6.9	6.2	5.0	2.4	1.8	1.5	I	e	C

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) $\times 10^3$	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3]
	Botanical Name	Trade Name		Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1]	Treat-ability Grade[2]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	ae) *Castanopsis hystrix*	Indian chestnut (ICH)	West Bengal	624	9.85	10.6	8.8	7.0	0.8	1.2	6.4	5.7	4.6	2.7	2.1	1.7	II	b	B
	af) *Chukrasia vclutina* (*Syn. C. Tabularis*)	Chickrassy (CHI)	West Bengal	666	8.35	11.8	9.8	7.9	1.1	1.5	7.1	6.3	5.2	3.9	3.1	2.5	II	c	B
	ag) *Calophyllum wightianum*	Poon (POO)	Maharashtra	689	8.68	13.5	11.2	9.0	1.0	1.4	8.7	7.8	6.4	4.0	3.1	2.5	II	–	B
	ah) *Canarium strictum*	White dhup	Assam	569	10.54	10.1	8.4	6.7	0.7	1.1	6.2	5.5	4.5	2.1	1.6	1.3	III	–	C
	aj) *Chlorophora excelsa*	-	Chennai	471	6.57	10.2	8.5	6.8	0.5	0.7	6.4	5.6	4.6	2.0	1.6	1.3	–	–	–
	ak) *Cocosnucifera*	Coconut (COC)	Kerala	761	7.34	9.2	7.7	6.1	0.7	1.1	9.5	8.4	6.9	3.9	3.0	2.5	–	–	–
	am) *Dalbergia latifolia*	Rosewood (ROS)	Madhya Pradesh	884	8.39	12.9	10.8	8.6	1.1	1.6	8.0	7.1	5.8	4.2	3.3	2.7	I	–	B
	an) *Dalbergia sissee*	Sisso (SIS)	Punjab	799	7.14	12.8	10.7	8.5	1.3	1.8	8.2	7.3	6.0	4.2	3.3	2.7	I	e	B
	ap) *Dillemia indica*	Dillenia (DIL)	West Bengal	617	8.61	12.1	10.0	8.0	0.8	1.2	7.3	6.5	5.3	2.7	2.1	1.7	III	a	B

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content Kg/m^3	Modulus of Elasticity (All Grades and All Locations) × 10^3 N/mm^2	Permissible Stress for Grade I N/mm^2												Preservative Characters		Refracteriness to Air Seasoning[3)]
	Botanical Name	Trade Name				Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain				Durability Class[1)]	Treat-ability Grade[2)]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)		(18)	(19)	(20)
	aq) *Dillenia pentagyne*	Dillenia (DIL)	West Bengal	622	7.56	11.8	9.9	7.9	0.9	1.3	7.1	6.3	5.2	3.5	2.7	2.2		III	d	B
	ar) *Diospyres melanoxylon*	Ebony (EBO)	Maharashtra	818	7.69	10.9	9.1	7.3	0.9	1.2	7.0	6.2	5.1	3.3	2.6	2.1		II	–	A
	as) *Duabanga grandiflora* (*Syn. D. Sonneratioides*)	Lampati (LAP)	West Bengal	485	8.38	9.8	8.2	6.5	0.6	0.9	6.4	5.7	4.7	1.8	1.4	1.1		III	c	C
	at) *Elesocarpus tuberculatus*	Rudrak (RUD)	Chennai	466	8.74	9.7	8.1	6.4	0.7	1.0	6.3	5.6	4.6	2.0	1.5	1.3		–	–	C
	au) *Eucalyptus hybrid*	Mysore gum (MGU)	Chennai	753	6.00	10.2	8.5	6.8	0.9	1.2	7.3	6.5	5.3	4.0	3.1	2.5		III	e	–
	av) *Calitres rhomboidea* (*Syn. Frenela rhomboidea*)	–	Chennai	607	6.48	9.2	7.7	6.1	0.7	1.0	6.9	6.1	5.0	4.0	3.1	2.6		–	–	–
	aw) *Garuga pinnata*	Garuga (GAU)	Uttar Pradesh	571	7.58	11.7	9.7	7.8	1.0	1.5	7.2	6.4	5.3	3.4	2.6	2.1		I	e	B
	ay) *Gmelina arborea*	Gamari (GAM)	Uttar Pradesh	501	7.02	9.8	8.2	6.6	0.8	1.2	5.7	5.0	4.1	4.2	3.2	2.7		I	e	B

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content Kg/m^3	Modulus of Elasticity (All Grades and All Locations) × 10^3 N/mm^2	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3)]
	Botanical Name	Trade Name				Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1)]	Treat-ability Grade[2)]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	az) *Gardonia latifolia*	Gardenia (GAI)	Madhya Pradesh	705	7.13	14.1	11.7	9.4	1.2	1.7	8.4	7.4	6.1	4.6	3.6	3.0	–	–	–
	ba) *Hardwickis binata*	Anjan (ANJ)	Madhya Pradesh	852	6.64	14.1	11.8	9.4	1.3	1.8	9.0	8.0	6.5	7.4	5.6	4.7	I	e	–
	bb) *Heloptelea integrifolia*	Kanju (KAN)	Uttar Pradesh	592	7.46	12.0	10.0	8.0	0.9	1.3	6.7	6.0	4.9	2.8	2.2	1.8	III	b	B
	bc) *Heterrophragma rexburghii*	Palang (PAL)	Madhya Pradesh	616	8.69	12.3	10.2	8.2	0.7	1.0	7.9	7.0	5.7	3.4	2.6	2.1	–	–	–
	bd) *Juglans spp.*	Walnut (WAL)	Uttar Pradesh	565	9.00	9.9	8.3	6.6	0.9	1.2	5.8	5.2	4.2	2.2	1.7	1.4	III	–	B
	be) *Lagerstrosmia speciosa* (*Syn. L. flesregihal*)	Jarul (JAAR)	Northern Andaman	622	8.53	12.1	10.1	8.1	0.8	1.8	7.7	6.8	5.6	3.4	2.6	2.2	II	e	B
	bf) *Lannea grandis* (*Syn. L.coromandelica*)	Jhingan (JHI)	Uttar Pradesh	557	5.63	8.5	7.1	5.7	0.6	0.9	4.9	4.4	3.6	2.2	1.7	1.4	III	e	B
	bg) *Leucanena leucocephala*	Subabul (SUB)	Uttar Pradesh	673	6.32	11.6	9.7	7.8	1.0	1.5	7.4	6.6	5.4	3.8	3.0	2.4	–	–	–

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content Kg/m^3	Modulus of Elasticity (All Grades and All Locations) × 10^3 N/mm^2	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3]
	Botanical Name	Trade Name				Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1]	Treat-ability Grade[2]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	bh) *Lophopatalum wightianum*	Banati (BAN)	Chennai	460	7.33	8.5	7.5	5.6	0.5	0.8	5.3	4.7	3.8	1.8	1.4	1.1	III	–	C
	bj) *Madhuca longifolia varlatifolia* (*Syn. Bassia latifolia*)	Mahua (MAU)	Madhya Pradesh	936	8.82	13.0	10.8	8.7	1.0	1.4	7.5	6.7	5.5	6.3	4.9	4.0	I	e	A
	bk) *Mangifera indica*	Mango, Aam (MAN)	Orissa	661	9.12	12.2	10.2	8.2	1.0	1.4	7.3	6.5	5.3	3.1	2.4	2.0	III	a	C
	bm) *Machilus macrantha*	Machilus (MAC)	Chennai	521	7.63	10.2	8.5	6.8	0.7	1.0	6.3	5.6	4.6	2.4	1.9	1.5	III	e	B
	bn) *Mallotus philippinensis*	Raini (RAI)	Uttar Pradesh	662	7.51	10.8	9.0	7.2	1.0	1.4	6.0	5.4	4.4	2.9	2.3	1.8	III	–	B
	bp) *Manglietia insignia*	-	Assam	449	10.37	10.9	9.1	7.3	0.7	1.0	8.0	7.1	5.8	3.4	2.6	2.1	–	–	–
	bq) *Michelia montana*	Champ (CHM)	West Bengal	512	8.25	10.9	9.1	7.3	0.7	1.0	6.6	5.9	4.8	2.8	2.2	1.8	I	–	B

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) $\times 10^3$	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3]
	Botanical Name	Trade Name		Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1]	Treat-ability Grade[2]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	br) *Mitragyna pervifolia* (*Syn. Stephagyne pervifolia*)	Kaim (KAI)	Uttar Pradesh	651	7.82	12.6	10.5	8.4	1.0	1.5	7.9	7.0	5.7	3.7	2.9	2.4	III	b	B
	bs) *Michelia excelsa*	Champ (CHM)	West Bengal	513	10.12	9.8	8.2	6.5	0.7	1.0	6.1	5.5	4.5	1.6	1.3	1.0	II	e	B
	bt) *Miliusa velutnia*	Domsal (DOM)	Uttar Pradesh	747	7.92	11.7	9.7	7.8	1.1	1.6	7.0	6.3	5.1	3.7	2.9	2.4	III	–	–
	bu) *Morus alba*	Mulberry (MUL)	Uttar Pradesh	743	8.20	11.8	9.8	7.9	1.0	1.4	6.6	5.8	4.8	3.8	2.9	2.4	II	–	B
	bv) *Morus serrata*	Mulberry (MUL)	Himachal Pradesh	657	7.03	10.2	8.5	6.8	0.9	1.3	5.6	5.0	4.1	2.6	2.0	1.6	III	–	B
	bw) *Morus laevigata*	Bola (BOL)	Andaman	588	8.61	12.3	10.2	8.2	1.0	1.5	7.2	6.4	5.3	3.3	2.5	2.1	–	–	B
	by) *Ougeinia eejeinensis* (*Syn. O. delbergioides*)	Sandan (SAD)	Madhya Pradesh	784	8.54	13.3	11.1	8.9	1.2	1.7	8.5	7.5	6.2	5.1	3.9	3.2	I	–	B
	bz) *Phoebe hainesiana*	Bonsum (BOH)	Assam	566	9.50	13.2	11.0	8.8	0.8	1.2	8.8	7.8	6.4	2.8	2.1	1.8	II	c	B

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) × 10^3	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3]
	Botanical Name	Trade Name		Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1]	Treat-ability Grade[2]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	ca) *Pinus roxburghii* (*Syn. P. longifolia*)	Chir (CHR)	Uttar Pradesh	525	9.82	8.5	7.3	6.0	0.6	0.9	6.0	5.3	4.4	2.0	1.5	1.3	III	b	C
	cb) *Pinus wallichiana*	Kail (KAL)	–	515	6.80	6.6	5.6	5.0	0.6	0.8	5.2	4.6	3.8	1.7	1.3	1.0	II	c	C
	cc) *Phoebe goalperansis*	Bonsum (BOH)	Assam	511	7.65	9.7	8.1	6.5	0.7	1.0	6.6	5.9	4.8	2.2	1.7	1.4	II	c	B
	cd) *Parretiopsis jacquementiena*	Rohu Parrotia	Himachal Pradesh	761	5.77	12.5	10.4	8.3	1.2	1.7	6.8	6.1	5.0	4.0	3.1	2.5	III	–	B
	ce) *Pinus kesia* (*Syn. Pinus insularis*)	Khasi pine (KPI)	North East	513	7.38	8.9	7.4	5.9	0.6	0.7	5.8	5.2	4.3	1.5	1.2	1.0	III	a	B
	cf) *Pistacia integerrima*	Kikar singhi	Jammu and Kashmir	881	7.32	13.1	10.9	8.7	1.2	1.7	8.0	7.1	5.8	4.3	3.4	2.8	–	–	–
	cg) *Podocarpus nerrifolius*	Thitmin (THT)	Southern Andaman	533	9.41	12.5	10.4	8.3	6.1	0.9	8.0	7.1	5.8	2.6	2.0	1.6	II	–	–
	ch) *Polyalthia fragrances*	Debdaru (DEB) (Nedunar)	Maharashtra	752	9.15	11.9	9.9	7.9	0.8	1.2	6.7	6.0	4.9	3.0	2.3	1.9	III	–	B

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) $\times 10^3$	Permissible Stress for Grade I N/mm^2											Preservative Characters		Refracteriness to Air Seasoning[3)]
	Botanical Name	Trade Name		Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1)]	Treat-ability Grade[2)]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	cj) *Polyalthia coreoides*	–	Madhya Pradesh	700	9.29	13.2	11.0	8.8	1.0	1.4	7.1	6.3	5.2	3.2	2.5	2.0	–	–	–
	ck) *Prunus napeulensis*	Arupati	West Bengal	548	9.41	104.4	8.7	69.6	0.9	1.2	6.7	6.0	4.9	2.4	1.9	1.6	–	–	–
	cm) *Pterespermum acerifolium*	Hattipaila (HAT)	West Bengal	607	9.55	13.5	11.3	9.0	0.9	1.2	8.7	7.7	6.3	3.2	2.5	2.0	III	c	B
	cn) *Qucrcus spp.*	Oak	North East	657	11.65	11.4	9.5	7.6	0.8	1.2	6.7	5.9	4.8	2.0	1.6	1.3	II	c	B
	cp) *Raderomachera xylocarpe* (*Syn. Sterosperam xylocarpum*)	Vedankonnai	Chennai	696	8.52	13.2	11.0	8.8	1.1	1.5	9.0	8.0	6.6	4.3	3.3	2.7	II	a	–
	cq) *Schleichera oleosa* (*Syn. S. trijuga*)	Kusum (KUS)	Bihar	1 032	12.12	15.5	13.0	10.4	1.5	2.1	10.9	9.7	7.9	6.1	4.2	3.9	II	a	A
	cr) *Schima wallichii*	Chilauni (CHL)	West Bengal	693	9.57	11.1	9.3	7.4	0.9	1.3	6.6	5.9	4.8	2.3	1.8	1.4	III	d	B
	cs) *Shotea assamica*	Makai (MAK)	Assam	548	9.27	11.1	9.2	7.4	0.9	1.3	7.1	6.3	5.2	2.9	2.2	1.8	III	c	B

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) × 10^3	Permissible Stress for Grade I N/mm^2												Preservative Characters		Refracteriness to Air Seasoning[3)]
	Botanical Name	Trade Name		Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain				Durability Class[1)]	Treat-ability Grade[2)]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)		(18)	(19)	(20)
	ct) *Sonneralia apetale*	Keora (KEO)	West Bengal	617	8.63	12.8	10.7	8.5	0.9	1.3	7.4	6.6	5.4	4.8	3.7	3.0		II	–	B
	cu) *Stereospermum suaveolans*	Padri (PAD)	Uttar Pradesh	721	8.86	13.3	11.1	8.9	0.9	1.3	7.3	7.0	5.7	3.5	2.7	2.2		III	–	B
	cv) *Tactona grandis*	Teak (TEA)	Madhya Pradesh	617	8.49	12.8	10.7	8.5	0.8	1.3	7.9	7.0	5.7	4.0	3.1	2.6		I	e	B
	cw) *Terminalia arjuna*	Arjun (ARJ)	Bihar	794	7.71	12.2	10.2	8.2	1.1	1.6	7.4	6.6	5.4	5.2	4.1	3.3		II	b	B
	cy) *Terminalia myriocarpa*	Hollock (HOC)	Assam	615	9.62	11.9	9.9	8.0	0.9	1.2	7.6	6.7	5.5	2.9	2.2	1.8		III	a	B
	cz) *Terminalia procera*	White bombwae (WBO)	Northern Andaman	626	8.99	11.8	9.8	7.9	0.9	1.3	7.2	6.4	5.3	3.0	2.3	1.9		III	b	B
	da) *Taxus buccata*	Yew (YEW)	West Bengal	705	7.79	14.3	11.9	9.5	1.2	1.7	8.7	7.8	6.4	4.7	3.7	3.0		–	–	–
	db) *Tamarindus indica*	Imli (IML)	Chennai	913	5.63	11.4	9.5	7.6	1.2	1.7	7.0	6.2	5.1	5.3	4.1	3.4		–	–	B

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Table 1 (*Continued*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content Kg/m^3	Modulus of Elasticity (All Grades and All Locations) × 10^3 N/mm^2	Permissible Stress for Grade I N/mm^2												Preservative Characters		Refracteriness to Air Seasoning[3)]
	Botanical Name	Trade Name				Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			Durability Class[1)]	Treat-ability Grade[2)]	
						Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
	dc) *Toena ciliata*	Toon (TOO)	Uttar Pradesh	487	6.40	8.7	7.3	5.8	0.7	1.0	5.4	4.8	3.9	2.4	1.8	1.5	II	c	B
	dd) *Vateria indica*	Vellapine (VEL)	Chennai	535	10.95	11.5	9.6	7.6	0.7	1.1	7.5	6.7	5.5	2.3	1.8	1.4	III	e	C
	de) *Aeculas indica*	Horse chestnut (HCH)	Uttar Pradesh	484	7.55	8.5	7.1	5.7	0.8	1.1	4.8	4.2	3.5	1.8	1.4	1.1	–	–	B
	df) *Borassus flabelsfer*	Tad (Palmyra) (TAD)	Arunachal Pradesh	838	8.79	10.5	8.8	7.0	0.7	1.0	10.0	8.8	7.2	4.7	3.6	2.7	–	–	–
	dg) *Eucalyptus cemaldulensis*	Eucalyptus	Karnataka	804	9.53	12.8	10.6	8.5	0.8	1.1	7.2	6.4	5.2	3.5	2.7	2.2	–	–	A
	dh) *Eucalyptus camaldulenis*	Eucalyptus	Uttar Pradesh	781	7.03	12.4	10.4	8.3	1.1	1.6	7.9	7.0	5.7	3.5	2.8	2.3	–	–	A
	dj) *Eucalyptus pilularia*	Eucalyptus	Tamil Nadu	713	9.22	14.8	12.3	11.1	1.0	1.4	8.5	7.6	6.2	2.8	2.2	1.8	–	–	A
	dk) *Eucalyptus propingus*	Eucalyptus	Tamil Nadu	584	7.93	12.8	10.7	8.5	0.8	1.2	8.0	5.4	4.4	2.5	1.9	1.6	–	–	A

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Table 1 (*Concluded*)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) × 10^3	Permissible Stress for Grade I N/mm^2												Preservative Characters		Refracteriness to Air Seasoning[3)]
				Kg/m^3	N/mm^2	Bending-Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain				Durability Class[1)]	Treat-ability Grade[2)]	
	Botanical Name	Trade Name				Inside Location	Outside Location	Wet Location	Horizontal	Along Grain	Inside Location	Outside Location	Wet Location	Inside Location	Outside Location	Wet Location				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	
	dm) *Eucalyptus saligna*	Eucalyptus	Uttar Pradesh	819	8.24	11.5	9.6	7.6	1.5	2.1	8.2	7.3	6.0	6.2	4.8	4.0	–	–	A	
	NOTE — Any other species (including foreign species) of timber may be classified into Group A, B, or C based on their safe permissible stresses and other relevant properties, corresponding to the classification table provided for given Indian timber species. The assignment of a timber species to a specific group may be determined through standardized testing and evaluation of mechanical properties, including strength, durability and density, in accordance with the applicable test methods and criteria specified in accordance with accepted standards [C-3A(2)].																			

[1)] Classification for preservation based on durability tests, etc.

Class:

I Average life more than 120 months;

II Average life 60 months and above but less than 120 months; and

III Average life less than 60 months.

[2)] Treatability Grades:

a Heartwood easily treatable;

b Heartwood treatable, but complete penetration not always obtained; in case where the least dimension is more than 60 mm;

c Heartwood only partially treatable;

d Heartwood refractory to treatment; and

e Heartwood very refractory to treatment, penetration of preservative being practically nil even from the ends.

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[3] Classifications based on seasoning behaviour of timber and refractoriness with respect to cracking, splitting and drying rate:

A Highly refractory (slow and difficulty to season free from surface and end cracking);

B Moderately refractory (may be seasoned free from surface and end cracking within reasonably short periods, given a little protection against rapid drying conditions); and

C Non-refractory may be rapidly seasoned free from surface and end-cracking even in the open air and sun. If not rapidly dried, they develop blue stain and mould on the surface.

[4] Species thus marked and tested from other localities show higher strength to enable their categorization in higher group.

For Example:

i) Sal tested from West Bengal, Bihar, Uttar Pradesh and Assam can be classified as Group `A' species.

ii) Haldu tested from Bihar can be classified as Group `B' species.

iii) Morus laevigate (Bole) of Assam can be classified in Group `B' species.

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4.1.1 *Grouping*

Species of timber recommended for constructional purposes are classified in three groups on the basis of their strength properties, namely, modulus of elasticity (E) and extreme fibre stress in bending-tension (f_b).

The characteristics of these groups are as given below:

a) *Group A* — E above 12.6 kN/mm^2 and f_b above 18.0 N/mm^2.

b) *Group B* — E above 9.8 kN/mm^2 and up to 12.6 kN/mm^2 and f_b above 12.0 N/mm^2 and up to 18.0 N/mm^2.

c) *Group C* — E above 5.6 kN/mm^2 and up to 9.8 kN/mm^2 and f_b above 8.5 N/mm^2 and up to 12.0 N/mm^2.

NOTE — Modulus of elasticity given above is applicable for all locations and extreme fibre stress in bending is for inside location.

4.1.2 Timber species may be identified in accordance with good practice [C-3A(3)].

4.2 The general characteristics like durability, treatability and seasoning of the species are also given in Table 1. Species of timber other than those recommended in Table 1 may be used, provided the basic strength properties are determined and found in accordance with **4.1.1**.

NOTE — For obtaining basic stress figures of the unlisted species, reference may be made to the Forest Research Institute, Dehradun.

4.3 The permissible lateral strength (in double shear) of mild steel common wire nail shall be as given in Table 2 and Table 3 for different species of timber.

4.4 Moisture Content in Timber

The permissible moisture content of timber for various positions in buildings shall be as given in Table 4.

Table 2 Permissible Lateral Strengths (in Double Shear) of Nails 3.55 mm Dia (9 SWG), 80 mm Long

(*Clauses* 4.3, 7.8.2 *and* 8.3.2)

Sl No.	**Species of Wood**		**For Permanent Construction Strength per Nail**		**For Temporary Structures Strength per Nail (for Both Lengthening Joints and Node Joints)**
	Botanical Name	Trade Name	Lengthening Joints	Node Joints	
			N × 10^2	N × 10^2	N × 10^2
(1)	(2)	(3)	(4)	(5)	(6)
i)	*Albies pirdrow*	Fir	8	2	12
ii)	*Acacia catecha*	Khair	20	17.5	37.0
iii)	*Acacia nilotica*	Babul	15	11	32.5
iv)	*Acrocarpus fraxinifolius*	Mundani	18	9.5	19.5
v)	*Adina cordifolia*	Haldu	23.5	10	22
vi)	*Albizia lebbeck*	Kokko	20	7	24
vii)	*Albizia odoratissima*	Kala siris	14	5	22
viii)	*Anogeissus latifolia*	Axlewood	20	10	29
ix)	*Aphanamixis polystachya*	Pithraj	19	9	19

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Table 2 (*Continued*)

Sl No.	**Species of Wood**		**For Permanent Construction Strength per Nail**		**For Temporary Structures Strength per Nail (for Both Lengthening Joints and Node Joints)**
	Botanical Name	Trade Name	Lengthening Joints	Node Joints	
			$N \times 10^2$	$N \times 10^2$	$N \times 10^2$
(1)	(2)	(3)	(4)	(5)	(6)
x)	*Azadiratchta indica*	Neem	22	11.5	19
xi)	*Balanocarpus utilis*	Karung	17.5	21	32
xii)	*Bischofia javnica*	Uriam	13	6.5	18
xiii)	*Betula alnoides*	Birch	13	12	22.5
xiv)	*Boswellia serrata*	Salai	12	5	14.5
xv)	*Calophyllum spp.*	Poon	16	9	21
xvi)	*Canarium euphyllum*	White dhup	9	8	10.5
xvii)	*Castanopsis hystrix*	Indian chestnut	18	10.5	23.5
xviii)	*Cedrus deodara*	Deodar	9	4	15
xix)	*Chukrasia tabularis*	Chikrassy	24	8	27
xx)	*Cinnamomum spp.*	Cinnomon	12	9	13
xxi)	*Cullenia rosaana* (*Syn. Cullenia excalsa*)	Karani	11.5	5	19
xxii)	*Cupressus torulosa*	Cypress	6	5	18
xxiii)	*Dalbergia latifohia*	Rosewood	19	8.5	23
xxiv)	*Dalberia sissoo*	Sissoo	14.5	7.5	19
xxv)	*Dipterocarpus macrocarpus*	Hollong	17	7	20
xxvi)	*Dipterocarpus spp.*	Gurjan	19	9	19.5
xxvii)	*Dillenia pertagyna*	Dillenia	16.5	12	16
xxviii)	*Diospyros melanoxylon*	Ebony	26.5	10	30.5
xxix)	*Duabanga grandiflora*	Lampati	15	5	12
xxx)	*Dysoxylum binectariferum*	White Cedar	7.5	5	14.5
xxxi)	*Eucalyptus eugenioides*	Eucalyptus	17	10	30
xxxii)	*Garuga pinnata*	Garuga	11.0	7.5	21.0
xxxiii)	*Gmelina arborea*	Gaman	8	4	9.5
xxxiv)	*Grerillea robusta*	Silver oak	12.0	7.0	10.5
xxxv)	*Grewia tilifolia*	Dhaman	13	5	24
xxxvi)	*Lagerstroemia spp.*	Jarul	24.5	21.5	22.5

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Table 2 (*Continued*)

Sl No.	**Species of Wood**		**For Permanent Construction Strength per Nail**		**For Temporary Structures Strength per Nail (for Both Lengthening Joints and Node Joints)**
	Botanical Name	Trade Name	Lengthening Joints	Node Joints	
			$N \times 10^2$	$N \times 10^2$	$N \times 10^2$
(1)	(2)	(3)	(4)	(5)	(6)
xxxvii)	*Holoptelea integrifolia*	Kanju	12.5	9.5	20.5
xxxviii)	*Hopea parviflora*	Hopea	31.5	13	28.5
xxxix)	*Lagerstroemia spp.*	Lendi	19	5	26
xl)	*Madhuka Logifera* (*Syn. Barfia Latifolia*)	Mahua	23	7.5	26
xli)	*Mangifera indica*	Mango	11	9	16
xlii)	*Maniltoa polyandra*	Ping	26	23.5	32
xliii)	*Mesua ferrea*	Mesua	26	8	41
xliv)	*Michelia spp.*	Champ	13	9	20
xlv)	*Millingtonia spp.*	Phulsopa	10.5	6	17
xlvi)	*Morus alba*	Mulberry	13	10.5	22.5
xlvii)	*Melia azedarach*	Persian lilac (bakain)	10.5	2.5	10.5
xlviii)	*Morus alba*	Mulberry	13	10.5	22.5
xlix)	*Ougeinia oojeinensis*	Sandan	17	11	18
l)	*Phoebe spp.*	Bonsum	12	6	13
li)	*Pinus roxburghii*	Chir	11	10	16
lii)	*Pinus wallichiana*	Kail	7	3	9
liii)	*Planchonia andamanica*	Red bombwe	14	13	29
liv)	*Planchonia valida* (*Syn. P. andamonica*)	Red bombwe	14.5	13	29.5
lv)	*Poeciloneuron indicum*	Ballagi	16	7	15.5
lvi)	*Populus deltoides*	Poplar	16	11.5	17.5
lvii)	*Protium serratum* (*Syn. Burserra Serrata*) A. P.	–	25.5	7.0	22.5
lviii)	*Pterocarpus marsupium*	Bijasal	15	12	27
lix)	*Pterocarpus dalbergiodes*	Pauduak	19	14	23
lx)	*Quercus spp.*	Oak (Assam)	21	10	37.5

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Table 2 (*Concluded*)

Sl No.	Species of Wood		For Permanent Construction Strength per Nail		For Temporary Structures Strength per Nail (for Both Lengthening Joints and Node Joints)
	Botanical Name	Trade Name	Lengthening Joints	Node Joints	
			$N \times 10^2$	$N \times 10^2$	$N \times 10^2$
(1)	(2)	(3)	(4)	(5)	(6)
lxi)	*Quercus spp.*	Oak (Uttar Pradesh)	11	10.5	26.5
lxii)	*Scheichera cleosa*	Kusum	23	16	39.5
lxiii)	*Schima wallachi*	Chillauni	19.5	9.5	25.5
lxiv)	*Shorea assamica*	Makai	12.5	7.5	15.0
lxv)	*Shorea robusta*	Sal (Madhya Pradesh)	23	15.5	19.5
lxvi)	*Shorea robusta*	Sal (Uttar Pradesh)	10	5	19
lxvii)	*Stereospenmum chelonoides*	Padriwood	16	8	19.5
lxviii)	*Syzygium spp.*	Jamum	15	12	25
lxix)	*Tectona grandis*	Teak	14	8	13
lxx)	*Terminalia alata*	Sain	16	16	29
lxxi)	*Terminalia arjuna*	Arjun	8.5	7	16.5
lxxii)	*Terminalia bellirica*	Bahera	10	10	14
lxxiii)	*Terminalia biolata*	White chuglam	18	9	21
lxxiv)	*Terminalia chebula*	Myrobalan	16.5	5.5	22.5
lxxv)	*Terminalia procera*	Badam	18	10.5	20
lxxvi)	*Terminalia manii*	Black chuglam	23	10	33
lxxvii)	*Terminalia myriocarpa*	Hollock	13	10	19.5
lxxviii)	*Toona spp.*	Toona	10	8	21
lxxix)	*Xylia xylacarpa*	Irul	23	6	33
lxxx)	*Toona ciliata*	Toon	16	9	21

NOTES

1 Nails of 3.55 mm (9 SWG) diameter are most commonly used. The above values can also be used for 4 mm (8 SWG) diameter 100 mm long nails.

2 The values in Newton (N) are approximate converted values from kgf. For exact conversion the value is 1 kgf = 9.806 65 N.

3 Preboring of nail-jointed timber construction is desirable and shall be done in accordance with good practice as in, **6.2.2** and Table 3 of [C-3A(7)].

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Table 3 Permissible Lateral Strengths (In Double Shear) of Nails 5.00 mm Dia (6 SWG), 125 mm and 150 mm Long

(*Clauses* 4.3, 7.8.2 *and* 8.3.2)

Sl No.	Species of Wood		For Permanent Construction Strength per Nail		For Temporary Structures Strength per Nail (for Both Lengthening Joints and Node Joints)
	Botanical Name	Trade Name	Lengthening Joints	Node Joints	
			$N \times 10^2$	$N \times 10^2$	$N \times 10^2$
(1)	(2)	(3)	(4)	(5)	(6)
i)	*Abies pindrow*	Fir	16.5	4.5	21
ii)	*Acacia catechu*	Khair	42	25	71.5
iii)	*Acacia nilotica*	Babul	27	13.5	53
iv)	*Alibizia procera*	Safed siris	35	18	-
v)	*Alibizia odoratissima*	Kala siris	27.5	17.5	45
vi)	*Alstonia scholaris*	Chatian	9.5	5.5	27
vii)	*Anogeissus latifolia*	Axlewood	22.5	13	46.5
viii)	*Azadirachata indica*	Neem	23.5	16	47
ix)	*Cupressus torulosa*	Cypress	20	7	27
x)	*Cullenia rosayroana*	Karani	11	9.5	30
xi)	*Dalbergia sissoo*	Sissoo	17	15	43
xii)	*Diptrocarous spp.*	Gurjan	19.5	9.5	33
xiii)	*Eucalyptus Prob. E. Tereticornis*	–	37.5	28.5	47
xiv)	*Hardwickia binata*	Anjan	32	19	59
xv)	*Hopea perviflora*	Hopea	60.5	25	61.5
xvi)	*Holoptelea integrifolia*	kanju	18	12.5	37.5
xvii)	*Mangifera indica*	Mango	22.5	15	32
xviii)	*Mesua ferrea*	Mesua	24	15.5	57.5
xix)	*Michelia champaca*	Champ	26	12.5	39
xx)	*Pterocarpus marsupium*	Bijasal	20.5	15	43
xxi)	*Pinus roxburghii*	Chir	9	6	24
xxii)	*Shorea robusta* (Uttar Pradesh)	Sal	19.5	17	37
xxiii)	*Shorea robusta*	Sal	30.5	20	41
xxiv)	*Schleichera cleosa*	Kusum	15	14	55
xxv)	*Stereospimum personatum*	Padriwood	22	8	34
xxvi)	*Syzygium cumini*	Jamum	18	14.5	38.5
xxvii)	*Terminalia myriocarpa*	Hollock	27.5	9	41
xxviii)	*Tectona grandis*	Teak	28	13	30
xxix)	*Toona ciliata*	Toon	19	7	32.5
xxx)	*Hopea utilis*	Karung kangoo	31	10	58
xxxi)	*Phoebe spp.*	Bonsum	20	7.5	30
xxxii)	*Pterocarpus dalbergiodes*	Padauk	27.4	13.4	37.7
xxxiii)	*Populus deltoides*	Poplar	21	15	38.5

NOTES

1 Nails of 5.00 mm diameter are most commonly used.

2 The values in Newton (N) are approximate converted values from kgf. For exact conversion the value is 1 kgf = 9.806 65 N.

3 Preboring of nail-jointed timber construction is desirable and shall be done in accordance with good practice as in, **6.2.2** and Table 3 of [C-3A(7)].

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Table 4 Permissible Percentage Moisture Content Values

(*Clause* 4.4)

Sl No.	Use	Zones (*see* Note)			
		I	II	III	IV
(1)	(2)	(3)	(4)	(5)	(6)
i)	Structural elements	12	14	17	20
ii)	Doors and windows:				
	a) 50 mm and above in thickness	10	12	14	16
	b) Thinner than 50 mm	8	10	12	14
iii)	Flooring strips for general purposes	8	10	10	12
iv)	Flooring strips for tea gardens	12	12	14	16

NOTE — The country has been broadly divided into the following four zones based on the humidity variations in the country:

a) Zone I — Average annual relative humidity less than 40 percent;
b) Zone II — Average annual relative humidity 40 percent to 50 percent;
c) Zone III — Average annual relative humidity 50 percent to 67 percent; and
d) Zone IV — Average annual relative humidity more than 67 percent.

For detailed zonal classification in the country, tolerances, etc, reference may be made to good practice [C-3A(4)].

4.5 Sawn Timber

4.5.1 *Sizes*

Preferred cut sizes of timber for use in structural components shall be as given in Table 5, Table 6 and Table 7.

Table 5 Preferred Cut Sizes of Structural Timber for Roof Trusses (Span from 3 m to 20 m)

(*Clause* 4.5.1)

Sl No.	Thickness mm	Width mm							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
i)	20	40	50	60	80	100	–	–	–
ii)	25	40	50	60	80	100	120	140	160
iii)	30	40	50	60	80	100	120	140	160
iv)	35	–	–	60	80	100	120	140	160
v)	40	–	–	60	80	100	120	140	160
vi)	50	–	–	60	80	100	120	140	160
vii)	60	–	–	–	80	100	120	140	160
viii)	80	–	–	–	–	100	120	140	160

NOTES

1 For truss spans marginally above 20 m, preferred cut sizes of structural timber may be allowed.

2 Preferred lengths of timber — 1 m, 1.5 m, 2 m, 2.5 m and 3 m.

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Table 6 Preferred Cut Sizes of Structural Timber for Roof Purlins, Rafters, Floor Beams, etc

(*Clause* 4.5.1)

Sl No.	Thickness mm	Width mm						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	50	80	100	120	140	–	–	–
ii)	60	80	100	120	140	160	–	–
iii)	80	–	100	120	140	160	–	–
iv)	100	–	–	–	140	160	180	200

NOTE — Preferred lengths of timber — 1.5 m, 2 m, 2.5 m and 3 m.

Table 7 Preferred Cut Sizes of Structural Timber for Partition Framing and Covering and for Centering

(*Clause* 4.5.1)

Sl No.	Thickness mm	Width mm								
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
i)	10	40	50	60	80	–	–	–	–	–
ii)	15	40	50	60	80	100	–	–	–	–
iii)	20	40	50	60	80	100	120	160	200	–
iv)	25	40	50	60	80	100	120	160	200	240
v)	30	40	50	60	80	100	120	160	200	240
vi)	40	40	–	60	80	100	120	160	200	240
vii)	50	–	50	–	80	100	120	160	200	240
viii)	60	–	–	60	80	100	120	160	200	240
ix)	80	–	–	–	80	100	120	160	200	240

NOTE — Preferred lengths of timber: 0.5 m, 1 m, 1.5 m, 2 m, 2.5 m and 3 m.

4.5.2 *Tolerances*

Permissible tolerances in measurements of cut sizes of structural timber shall be as follows:

a) For width and thickness:

1) Up to and including 100 mm : $+3\ -\ 0\ mm$

2) Above 100 mm : $+6\ -\ 3\ mm$

b) For length : $+10\ -\ 0\ mm$

4.6 Grading of Structural Timber

4.6.1 Cut sizes of structural timber shall be graded, after seasoning, into three grades based on permissible defects given in Table 8:

a) Select grade;

b) Grade I; and

c) Grade II.

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Table 8 Permissible Defects for Cut Sizes of Timber for Structural Use

(*Clauses* 4.6.1 *and* 4.6.2.2)

All dimensions in millimetres.

Sl No.	Defects	Select Grade	Grade I	Grade II
(1)	(2)	(3)	(4)	(5)
i)	Wane	Shall be permissible at its deepest portion up to a limit of 1/8 of the width of the surface on which it occurs	Shall be permissible at its deepest portion up to a limit of 1/6 of the width of the surface on which it occurs	Shall be permissible at its deepest portion up to a limit of 1/4 of the width of the surface on which it occurs
ii)	Worm holes	Other than those due to powder post beetles are permissible	Other than those due to powder post beetles are permissible	Other than those due to powder post beetles are permissible
iii)	Slope of grain	Shall not be more than 1 in 20	Shall not be more than 1 in 15	Shall not be more than 1 in 12
iv)	Live knots:			

Sl No.	*Width of Wide Faces of Cut Sizes of Timber, Max*	*Permissible Maximum Size of Live Knot on*		*Permissible Maximum Size of Live Knot on*		*Permissible Maximum Size of Live Knot on*	
		Narrow Faces and ¼ of the Width Face Close to Edges of Cut Size of Timber	Remaining Central Half of the Width of the Wide Faces	Narrow Faces and ¼ of the Width Face Close to Edges of Cut Size of Timber	The Remaining Central Half of the Width of the Wide Faces	Narrow Faces and ¼ of the Width Face Close to Edges of Cut Size of Timber	Remaining Central Half of the Width of the Wide Faces
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	75	10	10	19	19	29	30
ii)	100	13	13	25	25	38	39
iii)	150	19	19	38	38	57	57
iv)	200	22	25	44	50	66	75
v)	250	25	29	50	57	66	87
vi)	300	27	38	54	75	81	114
vii)	350	29	41	57	81	87	123
viii)	400	32	44	63	87	96	132
ix)	450	33	47	66	93	99	141
x)	500	35	50	69	100	105	150
xi)	550	36	52	72	103	108	156
xii)	600	38	53	75	106	114	159

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Table 8 (*Concluded*)

v) Checks and shakes:

Sl No.	*Width of the Face of the Timber Max*	*Permissible Depth Max*	*Permissible Depth Max*	*Permissible Depth Max*
(1)	(2)	(3)	(4)	(5)
i)	75	12	25	36
ii)	100	18	35	54
iii)	150	25	50	75
iv)	200	33	65	99
v)	250	40	81	120
vi)	300	50	100	150
vii)	350	57	115	171
viii)	400	66	131	198
ix)	450	76	150	225
x)	500	83	165	249
xi)	550	90	181	270
xii)	600	100	200	300

4.6.2 The prohibited defects given in **4.6.2.1** and permissible defects given in **4.6.2.2** shall apply to structural timber.

4.6.2.1 *Prohibited defects*

Loose grains, splits, compression wood in coniferous species, heartwood rot, sap rot, crookedness, worm holes made by powder post beetles and pitch pockets shall not be permitted in all the three grades.

4.6.2.2 *Permissible defects*

Defects to the extent specified in Table 8 shall be permissible.

NOTE — Wanes are permitted provided they are not combined with knots and the reduction in strength on account of the wanes is not more than the reduction with maximum allowable knots.

4.6.3 *Location of Defects*

The influence of defects in timber is different for different locations in the structural element. Therefore, these should be placed during construction in such a way so that they do not have any adverse effect on the members, in accordance with the accepted standard [C-3A(5)].

4.7 Suitability

4.7.1 *Suitability in Respect of Durability and Treatability for Permanent Structures*

There are two choices as given in **4.7.1.1** and **4.7.1.2**.

4.7.1.1 *First choice*

The species shall be any one of the following:

a) Untreated heartwood of high durability. Heartwood if containing more than 15 percent sap wood, may need chemical treatment for protection;

b) Treated heartwood of moderate and low durability and Class ‘a’ and Class ‘b’ treatability;

c) Heartwood of moderate durability and Class ‘c’ treatability after pressure impregnation; and

d) Sapwood of all classes of durability after thorough treatment with preservative.

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4.7.1.2 *Second choice*

The species of timber shall be heartwood of moderate durability and Class 'd' treatability.

4.7.2 *Choice of Species for Load-Bearing Temporary Structures or Semi-Structural Components at Construction Site*

The species shall be any one of the following:

a) Heartwood of low durability and Class 'e' treatability; or

b) The species whose durability and/or treatability is yet to be established, as listed in Table 1.

4.8 Fastenings

All structural members shall be framed, anchored, tied and braced to develop the strength and rigidity necessary for the purposes for which they are used. Allowable stresses or loads on joints and fasteners shall be determined in accordance with recognized principles. Common mechanical fastenings are of bar type such as nails and spikes, wood screws and bolts, and timber connectors including metallic rings or wooden disc-dowels. Chemical fastenings include synthetic adhesives for structural applications.

NOTE — Synthetic adhesives are able to achieve for timber what welding has achieved for steel.

4.9 Requirements of Structural Timber

The various other requirements of structural timber for use in building shall be in accordance with the accepted standard [C-3A(5)].

5 PERMISSIBLE STRESSES

5.1 Fundamental stress values of different groups of timber are determined on small clear specimen according to good practices [C-3A(2)]. These values are then divided by the appropriate factors of safety to obtain the permissible stresses. Values of these factors of safety for different strength characteristics are as per the relevant table of the accepted standard [C-3A(5)].

5.2 The permissible stresses for Groups A, B and C for different locations applicable to Grade I structural timber shall be as given in Table 9 provided that the following conditions are satisfied:

a) The timbers should be of high or moderate durability and be given the suitable treatment where necessary;

b) Timber of low durability shall be used after proper preservative treatment to good practice [C-3A(6)]; and

c) The loads should be continuous and permanent and not of impact type.

5.3 The permissible stresses (excepting *E*) given in Table 9 shall be multiplied by the following factors to obtain the permissible stresses for other grades provided that the conditions laid down in **5.2** are satisfied:

a) For select grade timber : 1.16

b) For Grade II timber : 0.84

5.3.1 When low durability timbers are to be used [*see* **5.2(b)**] on outside locations, the permissible stresses for all grades of timber, arrived at by **5.2** and **5.3** shall be multiplied by 0.80.

5.4 Modification Factors for Permissible Stresses

5.4.1 *Due to Change in Slope of Grain*

When the timber has not been graded and has major defects like slope of grain, knots and checks or shakes but not beyond permissible value, the permissible stress given in Table 1 shall be multiplied by modification factor K_1 for different slopes of grain as given in Table 10.

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Table 9 Minimum Permissible Stress Limits (in N/mm^2) in Three Groups of Structural Timbers (For Grade I Material)

(*Clauses* 5.2 *and* 5.3)

Sl No.	Strength Character	Location of Use	Group A	Group B	Group C
(1)	(2)	(3)	(4)	(5)	(6)
i)	Bending and tension along grain	Inside[1)]	18.0	12.0	8.5
ii)	Shear[2)]:				
	a) Horizontal	All locations	1.05	0.64	0.49
	b) Along grain	All locations	1.5	0.91	0.70
iii)	Compression parallel to grain	Inside[1)]	11.7	7.8	4.9
iv)	Compression perpendicular to grain	Inside[1)]	4.0	2.5	1.1
v)	Modulus of elasticity ($\times 10^3$ N/mm^2)	All locations and grade	12.6	9.8	5.6

Table 10 Modifications Factor K_1 to Allow for Change in Slope of Grain

(*Clause* 5.4.1)

Sl No.	Slope	Modification Factor K_1	
		Strength of Beams, Joists and Ties	Strength of Posts or Columns
(1)	(2)	(3)	(4)
i)	1 in 10	0.80	0.74
ii)	1 in 12	0.90	0.82
iii)	1 in 14	0.98	0.87
iv)	1 in 15 and flatter	1.00	1.00

NOTE — For intermediary slopes of grains, values of modification factor may be obtained by interpolation.

[1)] For working stresses for other locations of use, that is, outside and wet, generally factors of 5/6 and 2/3 are applied.

[2)] The values of horizontal shear to be used only for beams. In all other cases shear along grain to be used.

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5.4.2 *Due to Duration of Load*

For different durations of design load, the permissible stresses given in Table 1 shall be multiplied by the modification factor K_2 given in Table 11.

NOTE — The strength properties of timber under load are time-dependent.

Table 11 Modification Factor K_2, for Change in Duration of Loading

(*Clause* 5.4.2)

Sl No.	Duration of Loading	Modification Factor K_2
(1)	(2)	(3)
i)	Continuous (normal)	1.0
ii)	Two months	1.15
iii)	Seven days	1.25
iv)	Wind and earthquake	1.33
v)	Instantaneous or impact	2.00

5.4.2.1 The factor K_2 is applicable to modulus of elasticity when used to design timber columns, otherwise they do not apply thereto.

5.4.2.2 If there are several duration of loads (in addition to the continuous) to be considered, the modification factor shall be based on the shortest duration load in the combination, that is, the one yielding the largest increase in the permissible stresses, provided the designed section is found adequate for a combination of other larger duration loads.

NOTE — In any structural timber design for dead loads, snow loads and wind or earthquake loads, members may be designed on the basis of total of stresses due to dead, snow and wind loads using modification factor $K_2 = 1.33$ for the permissible stresses (*see* Table 1) to accommodate the wind load, that is, the shortest of duration and giving the largest increase in the permissible stresses. The section thus found is checked to meet the requirements based on dead loads alone with modification factor $K_2 = 1.00$.

5.4.2.3 Modification factor K_2 shall also be applied to allowable loads for mechanical fasteners in design of joints, when the wood and not the strength of metal determines the load capacity.

6 DESIGN CONSIDERATIONS

6.1 All structural members, assemblies or framework in a building, in combination with the floors, walls and other structural parts of the building shall be capable of sustaining, with due stability and stiffness the whole dead and imposed loadings as per Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS, without exceeding the limits of relevant stresses specified in this Subsection.

6.2 Buildings shall be designed for all dead and imposed loads or forces assumed to come upon them during construction or use, including uplifts or horizontal forces from wind and forces from earthquakes or other loadings. Structural members and their connections shall be proportioned to provide a sound and stable structure with adequate strength and stiffness. Wooden components in construction generally include panels for sheathing and diaphragms, siding, beams, girder, columns, light framings, masonry wall and joist construction, heavy-frames, glued laminated structural members, structural sandwiches, prefabricated panels, lamella arches, portal frames and other auxiliary constructions.

6.3 Net Section

6.3.1 The net section is obtained by deducting from the gross sectional area of timber the projected area of all material removed by boring, grooving or other means at critical plane. In case of nailing, the area of the prebored hole shall not be taken into account for this purpose.

6.3.2 The net section used in calculating load carrying capacity of a member shall be at least net section determined as above by passing an imaginary plane or a series of connected planes transversely through the members.

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6.3.3 Notches shall be in no case remove more than one quarter of the section.

6.3.4 In the design of an intermediate or a long column, gross section shall be used in calculating load carrying capacity of the column.

6.4 Loads

6.4.1 The loads shall conform to those given in Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS.

6.4.2 The worst combination and location of loads shall be considered for design. Wind and seismic forces shall not be considered to act simultaneously.

6.5 Flexural Members

6.5.1 Such structural members shall be investigated for the following:

a) Bending strength;
b) Maximum horizontal shear;
c) Stress at the bearings; and
d) Deflection.

6.5.2 *Effective Span*

The effective span of beams and other flexural members shall be taken as the distance from face of supports plus one-half of the required length of bearing at each end except that for continuous beams and joists the span may be measured from centre of bearing at those supports over which the beam is continuous.

6.5.3 Usual formula for flexural strength shall apply in design:

$$f_{ab} = \frac{M}{Z} \leq f_b$$

6.5.4 *Form Factors for Flexural Members*

The following form factors shall be applied to the bending stress:

a) *Rectangular section* — For rectangular sections, for different depths of beams, the form factor K_3 shall be taken as:

$$K_3 = 0.81\left(\frac{D^2 + 89\,400}{D^2 + 55\,000}\right)$$

NOTE — Form factor (K_3) shall not be applied for beams having depth less than or equal to 300 mm.

b) *Box beams and I-beams* — For box beams and I-beams the form factor K_4 shall be obtained by using the formula:

$$K_4 = 0.8 + 0.8y\left(\frac{D^2 + 89\,400}{D^2 + 55\,000} - 1\right)$$

where

$$y = {p_1}^2\,(6 - 8p_1 + 3{p_1}^2)(1 - q_1) + q_1$$

c) *Solid circular cross sections* — For solid circular cross sections the form factor K_5 shall be taken as 1.18; and

d) *Square cross sections* — For square cross sections where the load is in the direction of diagonal, the form factor K_6 shall be taken as 1.414.

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6.5.5 *Width*

The minimum width of the beam or any flexural member shall not be less than 50 mm or 1/50 of the span, whichever is greater.

6.5.6 *Depth*

The depth of beam or any flexural member shall not be taken more than three times of its width without lateral stiffening.

6.5.6.1 *Stiffening*

All flexural members having a depth exceeding three times its width or a span exceeding 50 times its width or both shall be laterally restrained from twisting or buckling and the distance between such restraints shall not exceed 50 times its width.

6.5.7 *Shear*

6.5.7.1 The following formulae shall apply:

a) The maximum horizontal shear, when the load on a beam moves from the support towards the centre of the span, and the load is at a distance of three to four times the depth of the beam from the support, shall be calculated from the following general formula:

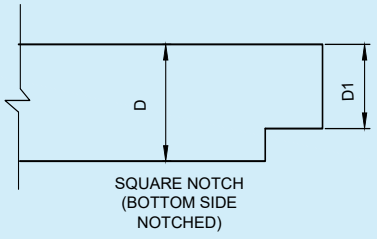
$$H = \frac{VQ}{Ib}$$

b) For rectangular beams:

$$H = \frac{3V}{2bD}$$

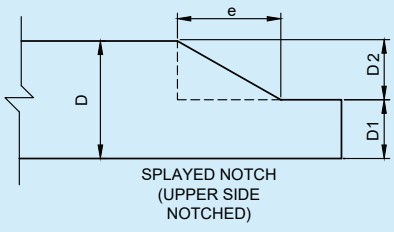
c) For notched beams, with tension notch at supports (*see* 6.5.7.3):

$$H = \frac{3VD}{2bD_1{}^2}$$



d) For notched at upper (compression) face, where e > D:

$$H = \frac{3VD}{2bD_1}$$



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e) For notched at upper (compression) face, where e < D:

$$H = \frac{3V}{2b\left[D - \left(\frac{D_2}{D}\right)e\right]}$$

6.5.7.2 For concentrated loads:

$$V = \frac{10C(l-x)(x/D)^2}{9l[2+(x/D)^2]} \quad and$$

For uniformly distributed loads:

$$V = \frac{W}{2}\left(1 - \frac{2D}{l}\right)$$

After arriving at the value of *V*, its value shall be substituted in the formula:

$$H = \frac{VQ}{Ib}$$

6.5.7.3 In determining the vertical reaction *V*, the following deductions in loads may be made:

a) Consideration shall be given to the possible distribution of load to adjacent parallel beams, if any;

b) All uniformly distributed loads within a distance equal to the depth of the beam from the edge of the nearest support may be neglected except in case of beam hanging downwards from a particular support; and

c) All concentrated loads in the vicinity of the supports may be reduced by the reduction factor applicable according to Table 12.

Table 12 Reduction Factor for Concentrated Loads in the Vicinity of Supports

[*Clause* 6.5.7.3(c)]

Sl No.	Distance of Load from the Nearest Support	1.5 D or Less	2 D	2.5 D	3 D or More
(1)	(2)	(3)	(4)	(5)	(6)
i)	Reduction factor	0.60	0.40	0.20	No reduction

NOTE — For intermediate distances, factor may be obtained by linear interpolation.

6.5.7.4 Unless the local stress is calculated and found to be within the permissible stress, flexural member shall not be cut, notched or bored except as follows:

a) Notches may be cut in the top or bottom neither deeper than one-fifth of the depth of the beam nor farther from the edge of the support than one-sixth of the span;

b) Holes not larger in diameter than one quarter of the depth may be bored in the middle third of the depth and length; and

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c) If holes or notches occur at a distance greater than three times the depth of the member from the edge of the nearest support, the net remaining depth shall be used in determining the bending strength (*see* Fig. 1):

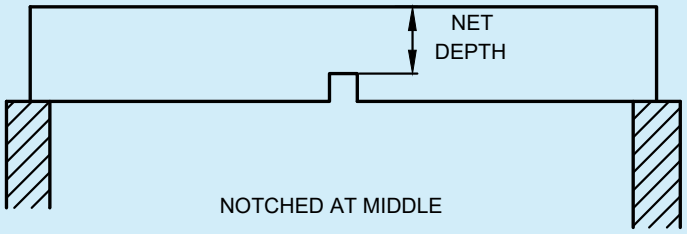

FIG. 1 NOTCHED BEAMS

6.5.8 *Bearing*

6.5.8.1 The ends of flexural members shall be supported in recesses which provide adequate ventilation to prevent dry rot and shall not be enclosed. Flexural members except roof timbers which are supported directly on masonry or concrete shall have a length of bearing of not less than 75 mm. Members supported on corbels, offsets and roof timbers on a wall shall bear immediately on and be fixed to wall-plate not less than 75 mm × 40 mm.

6.5.8.2 Timber joists or floor planks shall not be supported on the top flange of steel beams unless the bearing stress, calculated on the net bearing as shaped to fit the beam, is less than the permissible compressive stress perpendicular to the grain.

6.5.8.3 *Bearing stress*

6.5.8.3.1 *Length and position of bearing*

a) At any bearing on the side grain of timber, the permissible stress in compression perpendicular to the grain, f_{cn}, is dependent on the length and position of the bearing.

b) The permissible stresses given in Table 1 for compression perpendicular to the grain are also the permissible stresses for any length at the ends of a member and for bearings 150 mm or more in length at any other position.

c) For bearings less than 150 mm in length located 75 mm or more from the end of a member as shown in Fig. 2, the permissible stress may be multiplied by the modification factor K_7 given in Table 13.

d) No allowance need be made for the difference in intensity of the bearing stress due to bending of a beam.

e) The bearing area should be calculated as the net area after allowance for the amount of wane.

f) For bearings stress under a washer or a small plate, the same coefficient specified in Table 13 may be taken for a bearing with a length equal to the diameter of the washer or the width of the small plate.

g) When the direction of stress is at angle to the direction of the grain in any structural member, then the permissible bearing stress in that member shall be calculated by the following formula:

$$f_{c\theta} = \frac{f_{cp} \times f_{cn}}{f_{cp}\theta \ + f_{cn}\theta}$$

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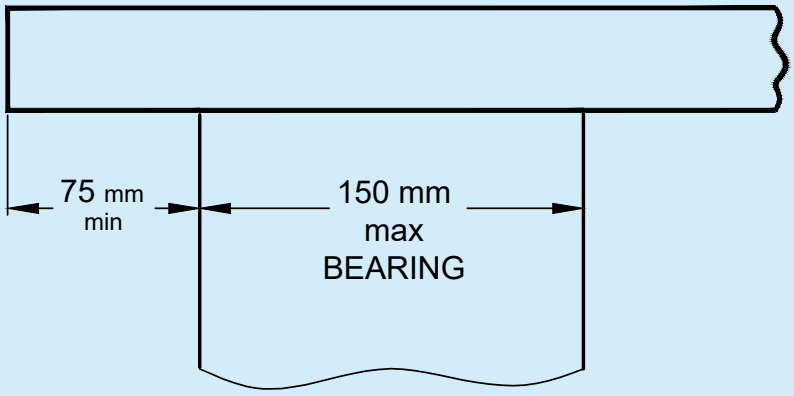

FIG 2. POSITION OF END BEARINGS

Table 13 Modification Factor K_7 for Bearing Stresses

[*Clauses* 6.5.8.3.1(c) *and* (f)]

Sl No.	Length of Bearing mm	15	25	40	50	75	100	150 or More
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	Modification factor, K_7	1.67	1.40	1.25	1.20	1.13	1.10	1.00

6.5.9 Purlin is a structural member, essentially a beam subjected to transverse loads and rests on the top chord of roof truss. Being under bi-axial bending, vertical loads are resolved into two components, that is, perpendicular to and parallel to roof slopes (*see* Fig. 3). The effect of normal and tangential components of the vertical load is worked out separately and the in fibre stress in bending, shall be the algebraic sum of both.

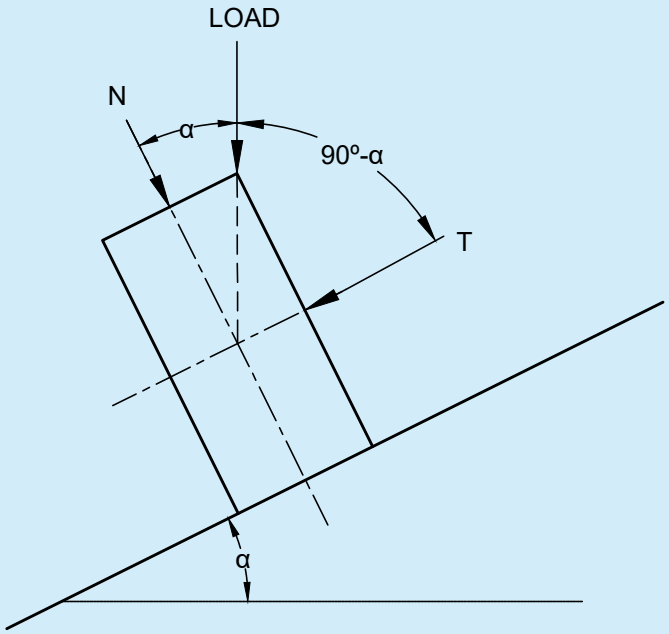

FIG. 3 PURLIN

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6.5.10 *Deflection*

The deflection in the case of all flexural members supporting brittle materials like gypsum ceilings, slates, tiles and asbestos sheets shall not exceed 1/360 of the span. The deflection in the case of other flexural members shall not exceed 1/240 of the span and 1/150 of the freely hanging length in the case of cantilevers.

6.5.10.1 Usual formula for deflection shall apply*:*

$$\delta = \frac{KWL^3}{EI}$$ (ignoring deflection due to shear strain)

K-value = 1/3 for cantilevers with load at free end;
= 1/8 for cantilevers with uniformly distributed load;
= 1/48 for beams supported at both ends with point load at centre; and
= 5/384 for beams supported at both ends with uniformly distributed load.

6.5.10.2 In order to allow the effect of long duration loading on *E*, for checking deflection in case of beams and joists the effective loads shall be twice the dead load if timber is initially dry.

NOTE — The tendency of flexural members to acquire larger permanent deflections under long continued loadings has to be taken care of either by considering double the dead load in calculations of deflection or by giving a factor of safety to MOE.

6.5.10.3 Self weight of beam shall be considered in design.

6.6 Columns

NOTE — The formulae given are for columns with pin end conditions and the length shall be modified suitably with other end conditions.

6.6.1 *Solid Columns*

Solid columns shall be classified into short, intermediate and long columns depending upon their slenderness ratio (*S*/*d*) as follows:

a) *Short columns* — where *S*/*d* does not exceed 11;

b) *Intermediate columns* — where *S*/*d* is between 11 and K_8; and

c) *Long columns* — where *S*/*d* is greater than K_8.

6.6.1.1 For short columns, the permissible compressive stress shall be calculated as follows:

$$f_c = f_{cp}$$

6.6.1.2 For intermediate columns, the permissible compressive stress is calculated by using the following formula:

$$f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_8 d}\right)^4\right]$$

6.6.1.3 For long columns, the permissible compressive stress shall be calculated by using the following formula:

$$f_c = \frac{0.329E}{\left(\frac{S}{d}\right)^2}$$

6.6.1.4 In case of solid columns of timber, *S*/*d* ratio shall not exceed 50.

6.6.1.5 The permissible load on a column of circular cross section shall not exceed that permitted for a square column of an equivalent cross sectional area, where side of square is equal to 0.886 times the diameter.

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6.6.1.6 For determining *S*/*d* ratio of a tapered column, such as for wooden poles, its least dimension shall be taken as the sum of the corresponding least dimensions at the small end of the column and one-third of the difference between this least dimension at the small end and the corresponding least dimension at the large end, but in no case shall the least dimension for the column be taken as more than one and a half times the least dimension at the small end. The induced stress at the small end of the tapered column shall not exceed the permissible compressive stress in the direction of grain.

6.6.2 *Built-up Columns*

6.6.2.1 *Box column*

Box columns shall be classified into short, intermediate and long columns as follows:

a) *Short columns* — where $\frac{S}{\sqrt{{d_1}^2+{d_2}^2}}$ is less than 8;

b) *Intermediate columns* — where $\frac{S}{\sqrt{{d_1}^2+{d_2}^2}}$ is between 8 and K_9; and

c) *Long columns* — where $\frac{S}{\sqrt{{d_1}^2+{d_2}^2}}$ is greater than K_9;

6.6.2.2 For short columns, the permissible compressive stress shall be calculated as follows:

$$f_c = q f_{cp}$$

6.6.2.3 For intermediate columns, the permissible compressive stress shall be obtained using the following formula:

$$f_c = q f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_9\sqrt{{d_1}^2 + {d_2}^2}}\right)^4\right]$$

6.6.2.4 For long columns, the permissible compressive stress shall be calculated by using the following formula:

$$f_c = \frac{0.329UE}{\left(\frac{S}{\sqrt{{d_1}^2 + {d_2}^2}}\right)^2}$$

6.6.2.5 The following values of *U* and *q*, depending upon plank thickness (*t*) in 6.6.2.3 and 6.6.2.4, shall be used for width of plank not greater than 5 times the plank thickness:

Sl No.	*t* mm	*U*	*q*
(1)	(2)	(3)	(4)
i)	25	0.80	1.00
ii)	50	0.60	1.00

6.6.3 *Spaced Columns*

6.6.3.1 The formulae for solid columns as specified in 6.6.1 are applicable to spaced columns with a restraint factor of 2.5 or 3, depending upon distances of end connectors in the column.

NOTE — A restrained factor of 2.5 for location of centroid group of fasteners at *S*/20 from end and 3 for location at *S*/10 to *S*/20 from end shall be taken.

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6.6.3.2 For intermediate spaced column, the permissible compressive stress shall be:

$$f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_{10}d}\right)^4\right]$$

6.6.3.3 For long spaced columns, the permissible compressive stress shall be:

$$f_c = \frac{0.329E \times 2.5}{(S/d)^2}$$

6.6.3.4 For individual members of spaced columns, *S*/*d* ratio shall not exceed 80.

6.6.4 Compression members shall not be notched. When it is necessary to pass services through such a member, this shall be effected by means of a bored hole provided that the local stress is calculated and found to be within the permissible stress specified. The distance from the edge of the hole to the edge of the member shall not be less than one quarter of width of the face.

6.7 Tension Member (Ties)

The stress is axial tension parallel to grain of wood shall be calculated on the basis of net-section area which shall not exceed the safe permissible value in tension parallel to grain:

$$f_{at} = \frac{Maximum\ force\ prevailing}{Area\ of\ cross\ section} \leq f_t$$

6.8 Structural Members Subject to Bending and Axial Stresses

6.8.1 Structural members subjected both to bending and axial compression shall be designed to comply with the following:

$$\frac{f_{ac}}{f_c} + \frac{f_{ab}}{f_b} \text{ is not greater than 1}$$

6.8.2 Structural members subjected both to bending and axial tension shall be designed to comply with the following:

$$\frac{f_{at}}{f_t} + \frac{f_{ab}}{f_b} \text{ is not greater than 1}$$

6.9 Timber Roof Truss

Trusses are framed structures in which separate straight members are so arranged and connected at their ends that members form triangles. The external loads cause direct stresses in the members. Essentially a plane structure which is very stiff in the plane of the members but very flexible in every other direction. For members subjected to reversal of stresses, design has to be for one stress and tested for the other applied at the panel points. Eccentricity of meeting members at joint may result in the increased stresses in design.

7 DESIGN OF COMMON STEEL WIRE NAIL JOINTS

7.1 General

Nail jointed timber construction is suitable for light and medium timber framings (trusses, etc) up to 15 m spans. With the facilities of readily available materials and simpler workmanship in mono-chord and split-chord constructions, this type of fabrication has a large scope. Preboring of nail-jointed timber construction is desirable and shall be done in accordance with good practice as in, **6.2.2** and Table 3 of good practice [C-3A(7)].

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7.2 Dimensions of Members

The dimension of an individual piece of timber (that is any single member) shall be within the range given below:

a) The minimum thickness of the main members in mono-chord construction shall be 30 mm;

b) The minimum thickness of an individual piece of members in split-chord construction shall be 20 mm for web members and 25 mm for chord members; and

c) The space between two adjacent pieces of timber shall be restricted to a maximum of 3 times the thickness of the individual piece of timber of the chord member. In case of web members, it may be greater for joining facilities.

7.3 No lengthening joint shall preferably be located at a panel point. Generally, not more than two, but preferably one, lengthening joint shall be permitted between the two panel points of the members.

7.4 Specification and Diameter of Nails

7.4.1 The nails used for timber joints shall conform to Part B 'Building Materials' of this NBCS. The nails shall be diamond pointed.

7.4.2 The diameter of nail shall be within the limits of one-eleventh to one-sixth of the least thickness of members being connected.

7.4.3 Where the nails are exposed to be saline conditions, common wire nails shall be galvanized.

7.5 Arrangement of Nails in the Joints

The end distances, edge distances and spacing of nails in a nailed joint should be such as to avoid undue splitting of the wood and shall not be less than those given in **7.5.1** and **7.5.2**.

7.5.1 *Lengthening Joints*

The requirement of spacing of nails in a lengthening joint shall be as given in Table 14 (*see also* Fig. 4):

Table 14 Spacing of Nails

(*Clause* 7.5.1)

Sl No.	Spacing of Nails	Type of Stress in the Joint	Requirement *Min*
(1)	(2)	(3)	(4)
i)	End distance	Tension compression	12 *n* 10 *n*
ii)	In direction of grain	Tension compression	10 *n* 5 *n*
iii)	Edge distance	–	5 *n*
iv)	Between row of nails perpendicular to the grain	–	5 *n*

NOTES

1 *n* is shank diameter of nails.

2 The 5*n* distance between rows perpendicular to the grain may be increased subject to the availability of width of the member keeping edge distance constant.

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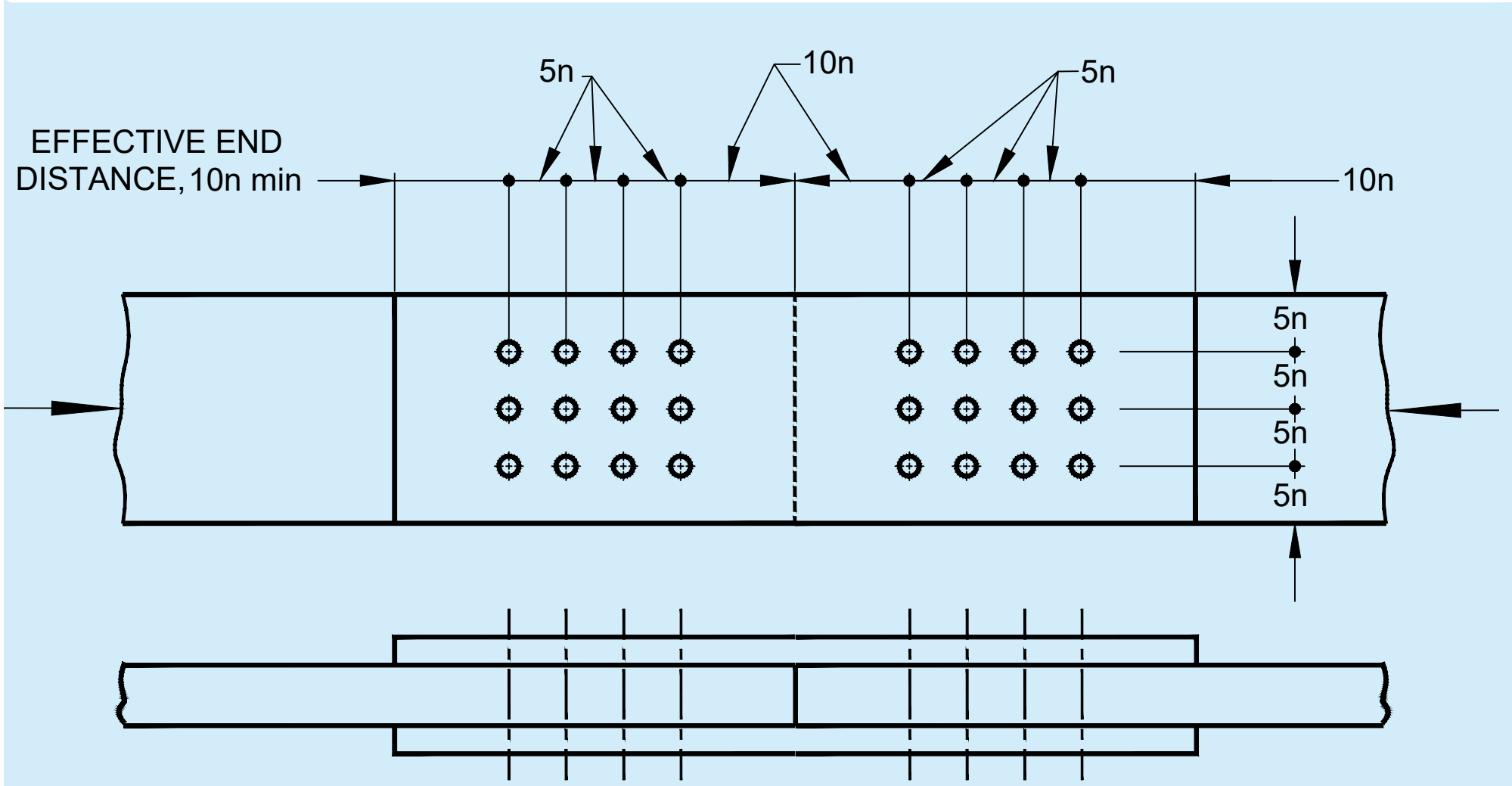

FIG. 4A MONOCHORD TYPE BUTT JOINT SUBJECT TO COMPRESSION

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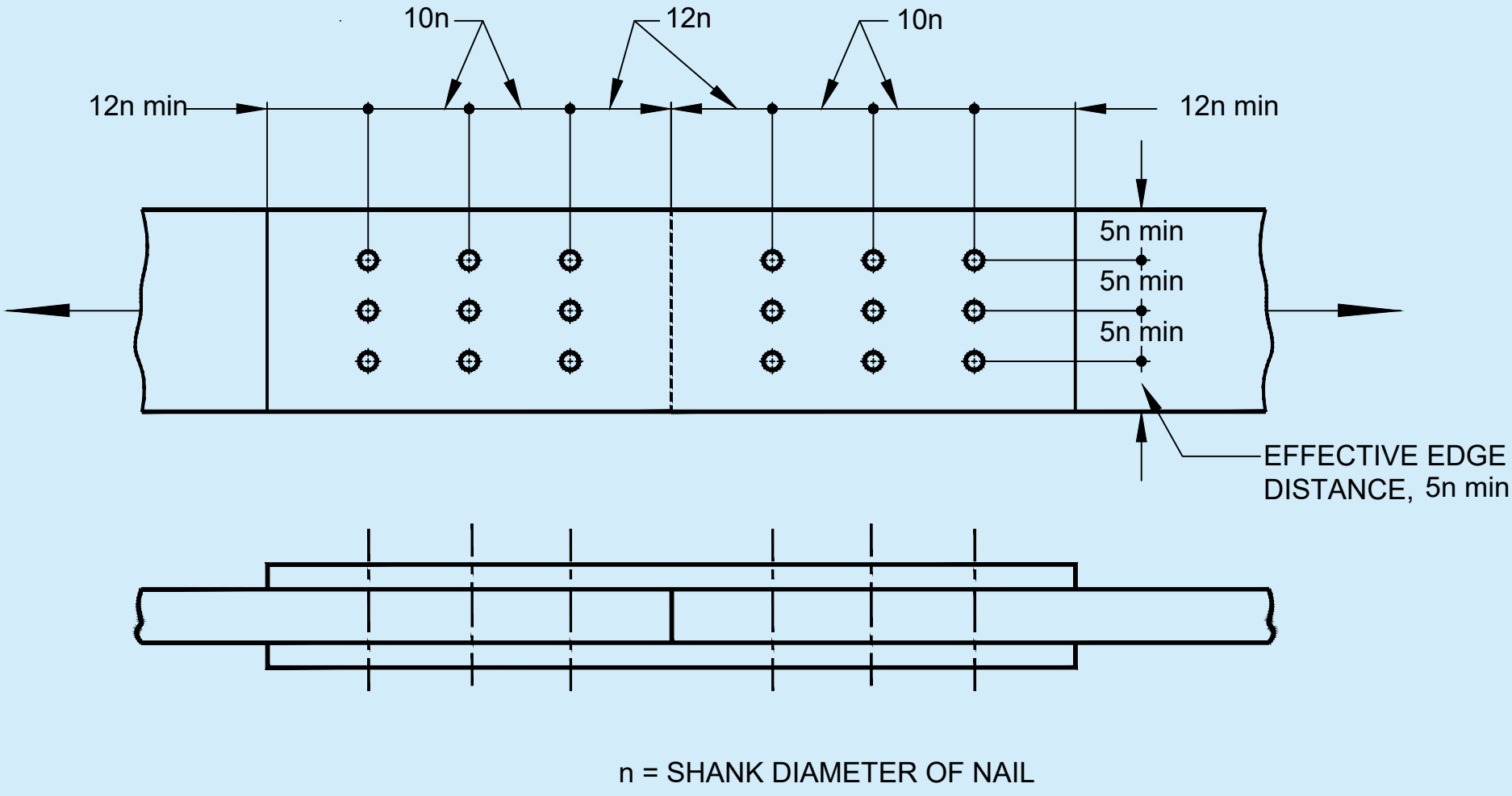

FIG. 4B MONOCHORD TYPE BUTT JOINT SUBJECT TO TENSION

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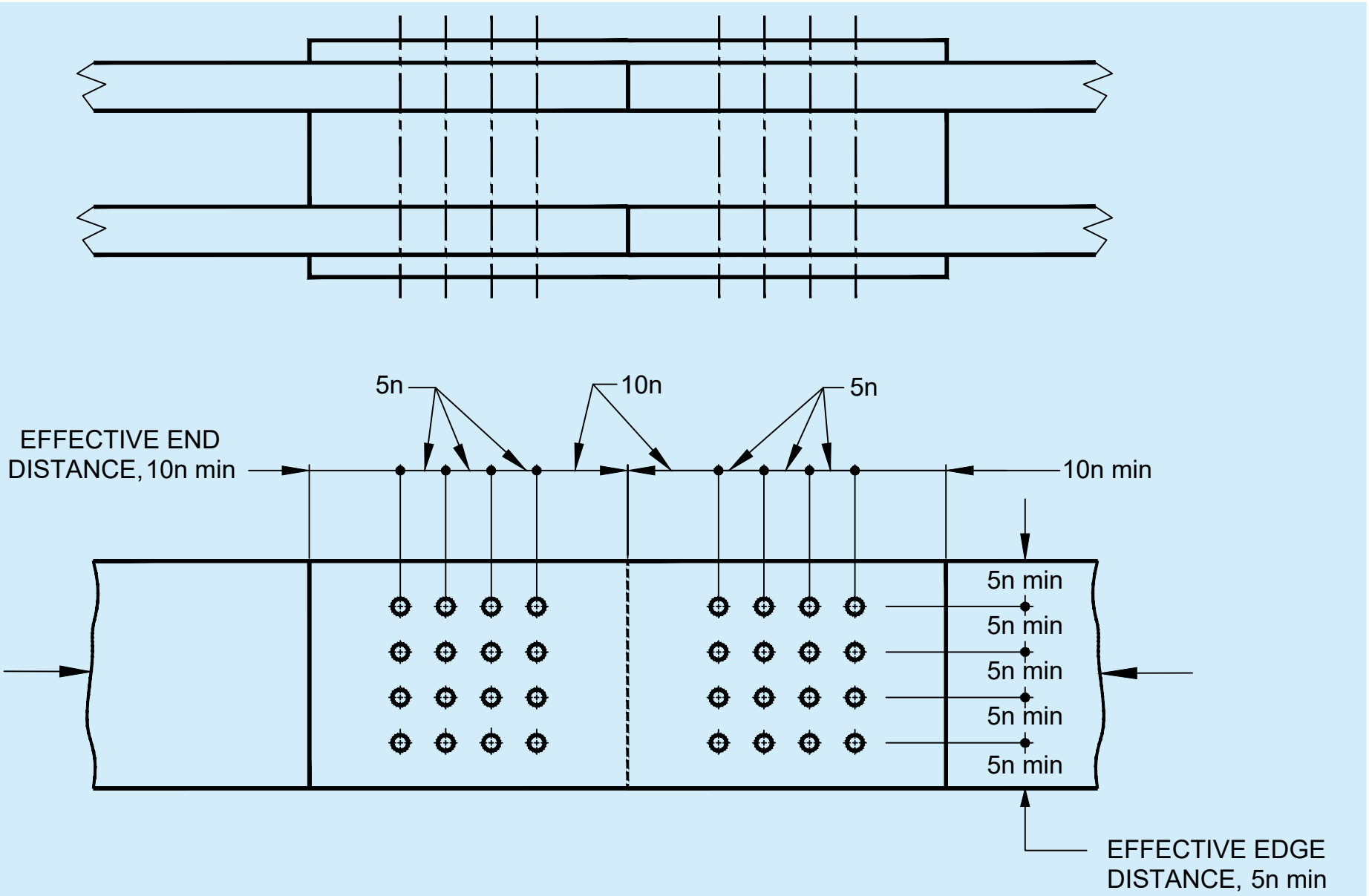

FIG. 4C SPLIT - CHORD TYPE BUTT JOINT SUBJECT TO COMPRESSION

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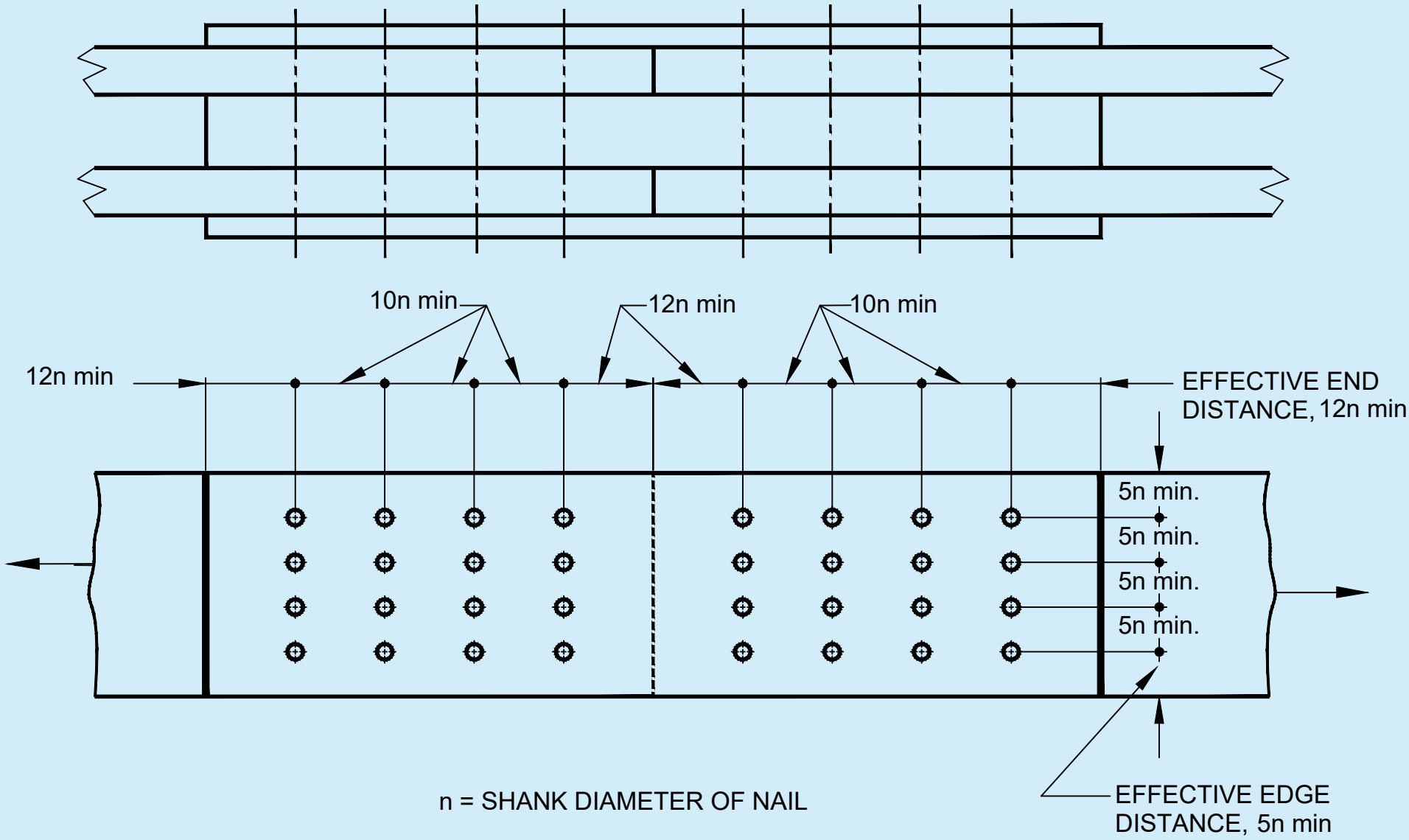

FIG. 4D SPLIT - CHORD TYPE BUTT JOINT SUBJECT TO TENSION

FIG. 4 SPACING OF NAILS IN A LENGHTHENING JOINT

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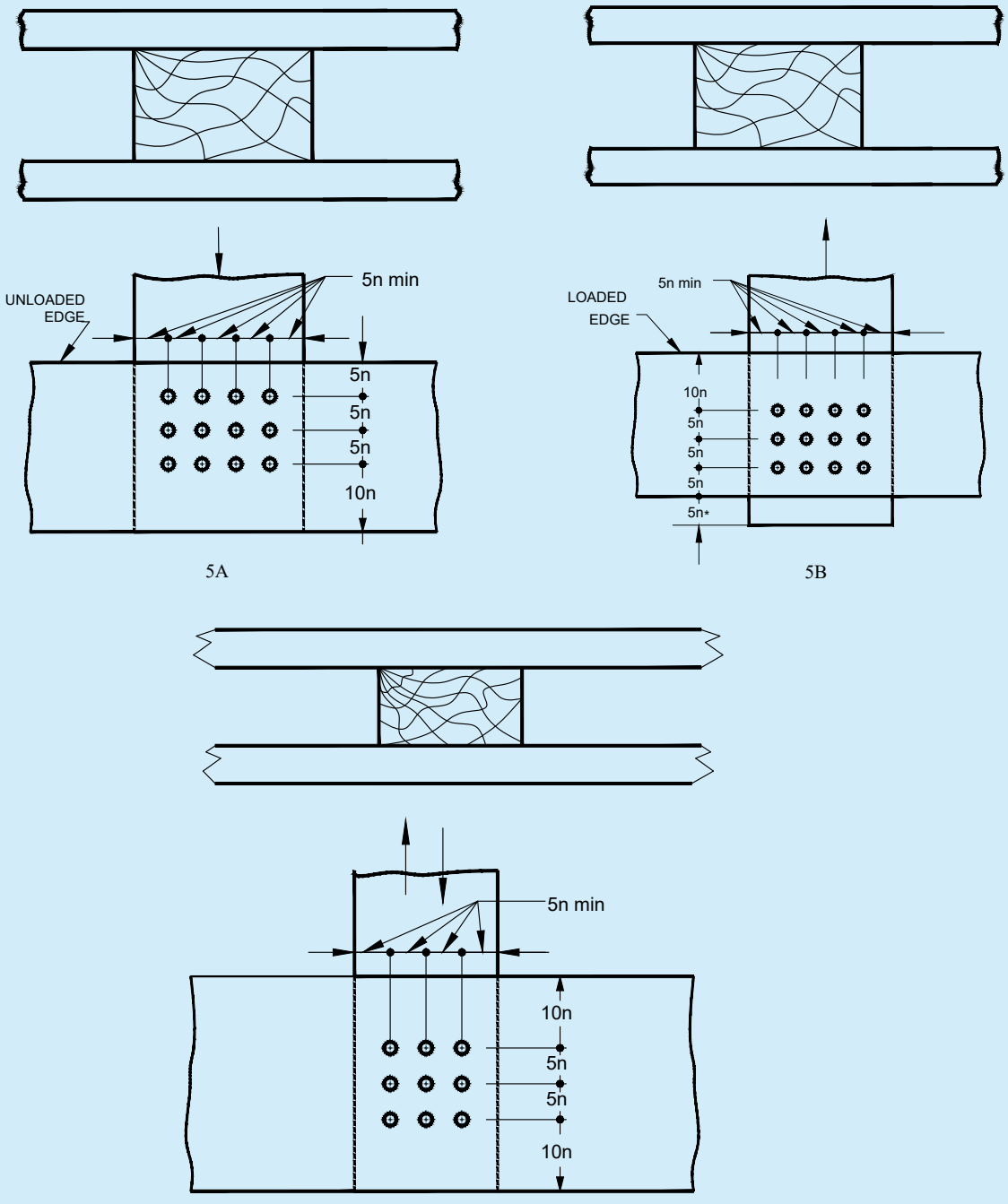
7.5.2 *Node Joints*

The requirement for spacing of nails in node joints shall be as specified in Fig. 5 where the members are at right angle and as in Fig. 6 where the members are inclined to one another at angles other than 90° and subjected to either pure compression or pure tension.



* 5n MAY BE INCREASED TO 10n, IF THE DESIGNED WIDTH OF CHORD MEMBER PERMITS. OTHERWISE THE END OF THE LOADED WEB MEMBER MAY BE EXTENDED BY 5n min

FIG. 5 SPACING OF NAILS WHERE MEMBERS ARE AT RIGHT ANGLES TO ONE ANOTHER

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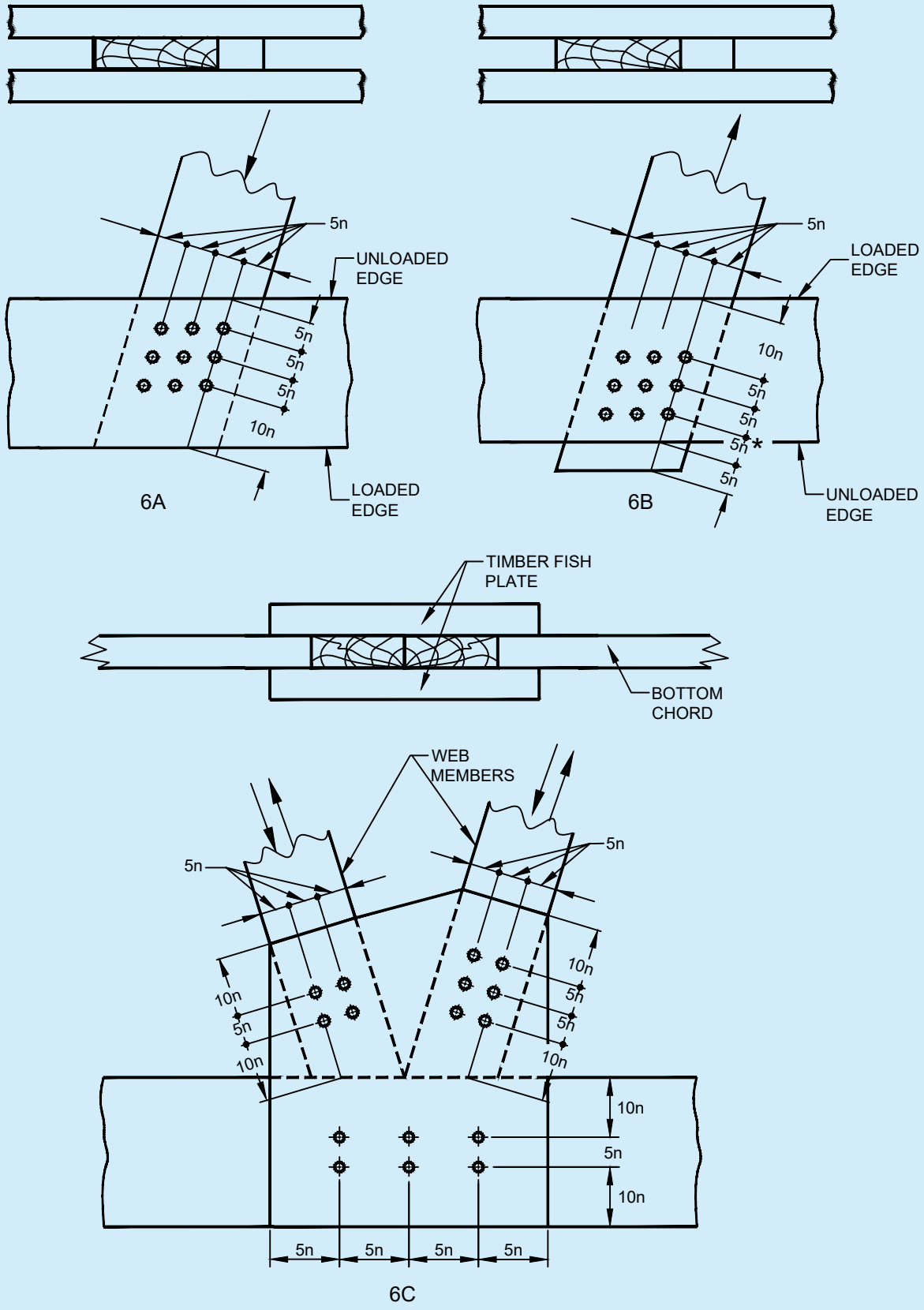

*5n MAY BE INCREASED TO 10n, IF THE DESIGNED WIDTH OF CHORD MEMBER PERMITS. OTHERWISE THE END OF THE LOADED WEB MEMBER MAY BE EXTENDED BY 5n min
n = SHANK DIAMETER OF NAIL

FIG. 6 SPACING OF NAILS AT NODE JOINTS WHERE MEMBERS ARE INCLINED TO ONE ANOTHER

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7.6 Penetration of Nails

7.6.1 For a lap joint when the nails are driven from the side of the thinner member, the length of penetration of nails in the thicker member shall be one and a half times the thickness of the thinner member subject to maximum of the thickness of the thicker member.

7.6.2 For butt joints the nails shall be driven through the entire thickness of the joint.

7.7 Design Considerations

7.7.1 Where a number of nails are used in a joint, the allowable load in lateral resistance shall be the sum of the allowable loads for the individual nails, provided that the centroid of the group of these nails lies on the axis of the member and the spacings conform to **7.5**. Where a large number of nails are to be provided at a joint, they should be so arranged that there are more of rows rather than more number of nails in a row.

7.7.2 Nails shall, as far as practicable, be arranged so that the line of force in a member passes through the centroid of the group of nails. Where this is not practicable, allowance shall be made for any eccentricity in computing the maximum load on the fixing nails as well as the loads and bending moment in the member.

7.7.3 Adjacent nails shall preferably be driven from opposite faces, that is, the nails are driven alternatively from either face of joint.

7.7.4 For a rigid joint, a minimum of 2 nails for nodal joints and 4 nails for lengthening joint shall be driven.

7.7.5 Two nails in a horizontal row are better than using the same number of nails in a vertical row.

7.7.6 The permissible lateral strength (in double shear) of mild steel common wire for different species of timber shall be as per **4.3**.

7.8 Special Consideration in Nail-Jointed Truss Construction

7.8.1 The initial upward camber provided at the centre of the lower chord of nail-jointed timber trusses shall be not less than 1/200 of the effective span for timber structures using seasoned wood and 1/100 for unseasoned or partially seasoned wood.

7.8.2 The total combined thickness of the gusset or splice plates on either side of the joint in a mono-chord type construction shall not be less than one and a half times the thickness of the main members subject to a minimum thickness of 25 mm of individual gusset plate.

NOTES

1 The allowable load or lateral strength values of nails shall be those as given in Table 2 and Table 3.

2 The strength data for joints given in the Subsection apply to gusset or splice or fish plates of solid wood; however, materials other than solid wood may be used for gusset when field tests are made and their strength requirements have been established.

7.8.3 The total combined thickness of all spacer blocks or plates or both including outer splice plates, at any joint in a split-chord type construction shall not be less than one and a half times the total thickness of all the main members at that joint.

7.9 Fabrication

The fabrication of nail-jointed timber construction shall be done in accordance with good practice [C-3A(7)].

8 DESIGN OF NAIL LAMINATED TIMBER BEAMS

8.1 Method of Arrangement

8.1.1 The beam shall be made up of 20 mm to 30 mm thick planks placed vertically with joints staggered in the adjoining planks with a minimum distance of 300 mm. The planks are laminated with the help of wire nails at regular intervals to take up horizontal shear developed in the beam besides keeping the planks in position (*see* Fig. 7).

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8.1.2 The advantage in laminations lies in dimensional stability, possible dispersal of defects and better structural performance.

8.2 Sizes of Planks and Beams

8.2.1 The plank thickness for fabrication of nailed laminated beams recommended are 20 mm, 25 mm and 30 mm.

8.2.2 In case of nailed laminated timber beam the maximum depth and length of planks shall be limited to 250 mm and 2 000 mm, respectively.

8.2.3 In order to obtain the overall width of the beam, the number and thickness of planks to form vertical nailed laminated beams, and also type and size of wire nail shall be as mentioned in Table 15. The protruding portion of the nail shall be cut off or clenched across the grains.

8.3 Design Considerations

8.3.1 Nail laminated beams shall be designed in accordance with **6** and provisions in the good practice [C-3A(8)].

8.3.1.1 The deflection in the case of nailed laminated timber beams, joists, purlins, battens and other flexural members supporting brittle materials like gypsum, ceiling slates, tiles and asbestos sheets shall not exceed 1/480 of the span. The deflection in case of other flexural members shall not exceed 1/360 of the span in the case of beams and joists and 1/225 of the freely hanging length in case of cantilevers.

8.3.2 Permissible lateral strength of diamond pointed mild steel wire nails shall be as given in Table 2 and Table 3 for Indian species of timber, which shall apply to nails that have their points cut flush with the faces. For nails clenched across the grains the strength may be increased by 20 percent over the values for nails with points cut flush.

Table 15 Number and Size of Planks and Nails for Nailed Laminated Beams

(*Clause* 8.2.3)

Sl No.	Overall Width of Beam mm	No. of Planks	Thickness of Each Plank mm	Type and Size of Nail to be Used mm
(1)	(2)	(3)	(4)	(5)
i)	50	2	25	80 long 3.55 dia
ii)	60	3	20	- do -
iii)	70	3	(2 × 25) (1 × 20)	- do -
iv)	80	4	20	100 long 4.0 dia
v)	90	3	30	- do -
vi)	100	4	25	125 long 5.0 dia
vii)	110	4	(3 × 30) (1 × 20)	- do -
viii)	120	4	30	- do -
ix)	150	5	30	150 long 5.0 dia

NOTE — A number of combinations of the different thickness of planks may be adopted as long as the minimum and maximum thickness of the planks are adhered to.

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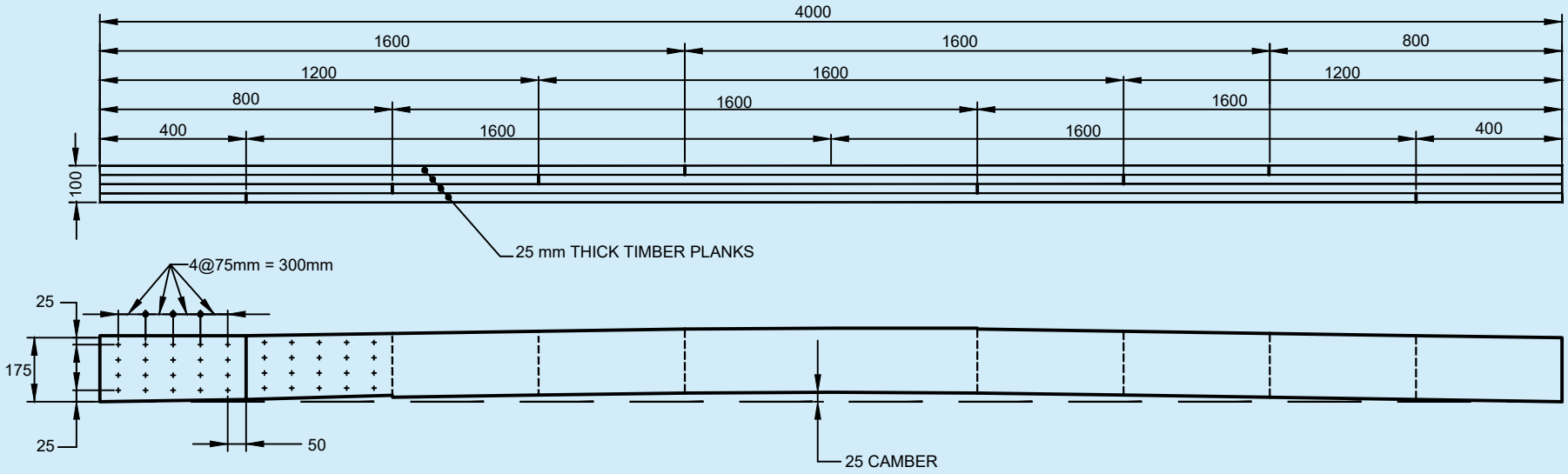

All dimensions in millimetres.

FIG. 7 PLAN AND ELEVATION OF A TYPICAL NAILED LAMINATED TIMBER BEAM

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8.3.3 *Arrangement of Nails*

8.3.3.1 A minimum number of four nails in a vertical row at regular interval not exceeding 75 mm to take up horizontal shear as well as to keep the planks in position shall be used. Near the lengthening joints of the planks this distance may, however, be limited to 50 mm instead of 75 mm.

8.3.3.2 Shear shall be calculated at various points of the beam and the number of nails required shall be accommodated within the distance equal to the depth of the beam, with a minimum of 4 nails in a row at a standard spacing as shown in Fig. 8.

8.3.3.3 If the depth of the beam is more, then the vertical intermediate spacing of nails may be increased proportionately.

8.3.3.4 If the nails required at a point are more than that can be accommodated in a row, then these shall be provided lengthwise of the beam within the distance equal to the depth of the beam at standard lengthwise spacing.

8.3.3.5 For nailed laminated beam minimum depth of 100 mm for 3.55 mm and 4 mm diameter nails and 125 mm for 5 mm diameter nails shall be provided.

8.4 Fabrication

The fabrication of nailed laminated timber beams shall be done in accordance with good practice [C-3A(8)].

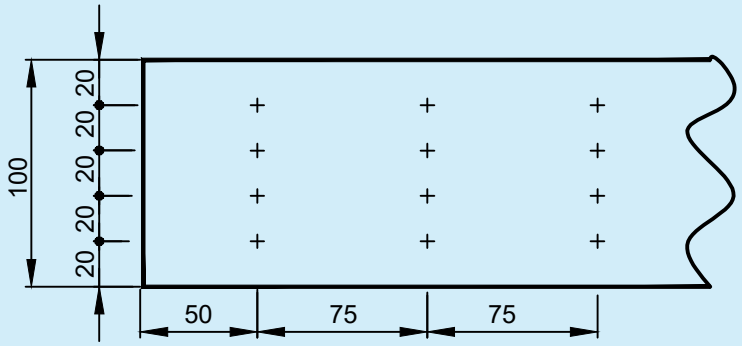

FIG. 8A FOR 3.55 mm AND 4 mm DIAMETER NAILS

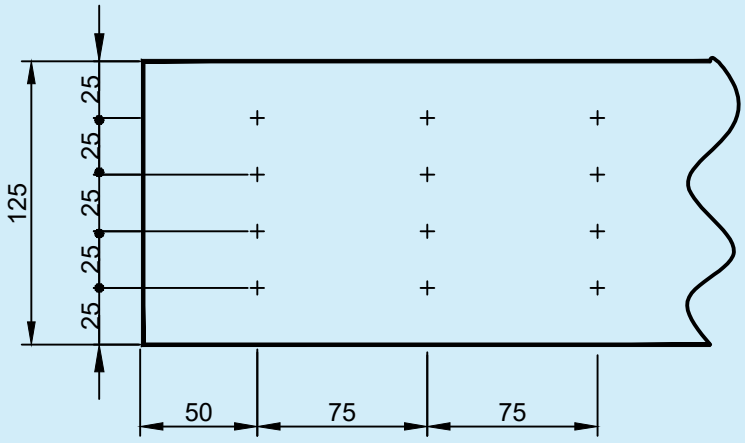

FIG. 8B FOR 5 mm DIAMETER NAILS

All dimensions in millimetres.

FIG. 8 STANDARD LENGTHWISE SPACING IN NAILED LAMINATED BEAM

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9 DESIGN OF BOLTED CONSTRUCTION JOINTS

9.1 General

Bolted joints suit the requirements of prefabrication in small and medium span timber structures for speed and economy in construction. Bolt jointed construction units offer better facilities as regards to workshop ease, mass production of components, transport convenience and re-assembly at site of work particularly in defence sector for high altitudes and far off situations. Designing is mainly influenced by the species, size of bolts, moisture conditions and the inclination of loadings to the grains. In principle, bolted joints follow the pattern of riveted joints in steel structures.

9.2 Design Considerations

9.2.1 Bolted timber construction shall be designed in accordance with **6**. The concept of critical section, that is, the net section obtained by deducting the projected area of bolt-holes from the cross-sectional area of member is very important for the successful design and economy in timber.

9.2.2 *Bolt Bearing Strength of Wood*

The allowable load for a bolt in a joint consisting of two members (single shear) shall be taken as one half the allowable loads calculated for a three member joint (double shear) for the same t'/d_3 ratio. The percentage of safe working compressive stress of timber on bolted joints for different t'/d_3 ratios shall be as given in Table 16.

Table 16 Percentage of Safe Working Compressive Stress of Timber for Bolted Joints in Double Shear

(*Clause* 9.2.2)

Sl No.	t'/d_3 Ratio	Stress Percentage	
		Parallel to Grain λ_1	Perpendicular to Grain λ_2
(1)	(2)	(3)	(4)
i)	1.0	100	100
ii)	1.5	100	96
iii)	2.0	100	88
iv)	2.5	100	80
v)	3.0	100	72
vi)	3.5	100	66
vii)	4.0	96	60
viii)	4.5	90	56
ix)	5.0	80	52
x)	5.5	72	49
xi)	6.0	65	46
xii)	6.5	58	43
xiii)	7.0	52	40
xiv)	7.5	46	39
xv)	8.0	40	38
xvi)	8.5	36	36
xvii)	9.0	34	34
xviii)	9.5	32	33

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Table 16 (*Concluded*)

Sl No.	t'/d_3 Ratio	Stress Percentage	
		Parallel to Grain λ_1	Perpendicular to Grain λ_2
(1)	(2)	(3)	(4)
xix)	10.0	30	31
xx)	10.5	–	31
xxi)	11.0	–	30
xii)	11.5	–	30
xiii)	12.0	–	28

9.2.2.1 Where a number of bolts are used in a joint, the allowable loads shall be the sum of the allowable loads for the individual bolts.

9.2.2.2 The factors for different bolt diameter used in calculating safe bearing stress perpendicular to grain in the joint shall be as given in Table 17.

9.2.3 *Dimensions of Members*

a) The minimum thickness of the main member in mono-chord construction shall be 40 mm;

b) The minimum thickness of side members shall be 20 mm and shall be half the thickness of main members; and

c) The minimum individual thickness of spaced member in split-chord construction shall be 20 mm and 25 mm for webs and chord members, respectively.

9.2.4 *Bolts and Bolting*

a) The diameter of bolt in the main member shall be so chosen to give larger slenderness (t'/d_3) ratio of bolt;

b) There shall be more number of small diameter bolts rather than small number of large diameter bolts in a joint;

c) A minimum of two bolts for nodal joints and four bolts for lengthening joints shall be provided;

d) There shall be more number of rows rather than more bolts in a row;

e) The bolt holes shall be of such diameter that the bolt can be driven easily; and

f) Washers shall be used between the head of bolt and wood surface as also between the nut and wood.

Table 17 Bolt Diameter Factor

(*Clause* 9.2.2.2)

Sl No.	Diameter of Bolt mm	Diameter Factor (d_f)
(1)	(2)	(3)
i)	6	5.70
ii)	10	3.60
iii)	12	3.35
iv)	16	3.15
v)	20	3.05
vi)	22	3.00
vii)	25	2.90

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9.3 Arrangement of Bolts

9.3.1 The following spacings in bolted joints shall be followed (*see* Fig. 9):

a) *Spacing of bolts in a row* — For parallel and perpendicular to grain loading = 4 d_3;

b) *Spacing between rows of bolts*:

1) *For perpendicular to grain loading* — 2.5 d_3 to 5 d_3 (2.5 d_3 for t'/d_3 ratio of 2 and 5 d_3 for t'/d_3 ratio of 6 or more. For ratios between 2 and 6 the spacing shall be obtained by interpolation.

2) *For parallel to grain loading* — At least (N - 4) d_3 with a minimum of 2.5 d_3. Also governed by net area at critical section which should be 80 percent of the total area in bearing under all bolts.

c) *End distance* — 7d_3 for softwoods in tension, 5 d_3 for hardwoods in tension and 4 d_3 for all species in compression; and

d) *Edge distance*:

1) For parallel to grain loading 1.5 d_3 or half the distance between rows of bolts, whichever is greater.

2) For perpendicular to grain loading, the loaded edge distance shall be at least 4 d_3.

9.3.2 For inclined members, the spacing given above for perpendicular and parallel to grain of wood may be used as a guide and bolts arranged at the joint with respect to loading direction.

9.3.3 The bolts shall be arranged in such a manner so as to pass the centre of resistance of bolts through the intersection of the gravity axis of the members.

9.3.4 Staggering of bolts shall be avoided as far as possible in case of members loaded parallel to grain of wood. For loads acting perpendicular to grain of wood, staggering is preferable to avoid splitting due to weather effects.

9.3.5 *Bolting*

The bolt holes shall be bored or drilled perpendicular to the surface involved. Forcible driving of the bolts shall be avoided which may cause cracking or splitting of members. A bolt hole of 1.0 mm oversize may be used as a guide for preboring.

9.3.5.1 Bolts shall be tightened after one year of completion of structure and subsequently at an interval of two to three years.

9.4 Outline for Design of Bolted Joints

Allowable load on one bolt (unit bearing stress) in a joint with wooden splice plates shall not be greater than value of P, R, F as determined by one of the following equations:

a) *For loads parallel to grain*:

$$P = f_{cp}\ a\ \lambda_1$$

b) *For loads perpendicular to grain*:

$$R = f_{cn}\ a\ \lambda_2\ d_f$$

c) *For loads at an angle to grain*:

$$F = \frac{PR}{P\sin^2\theta + R\cos^2\theta}$$

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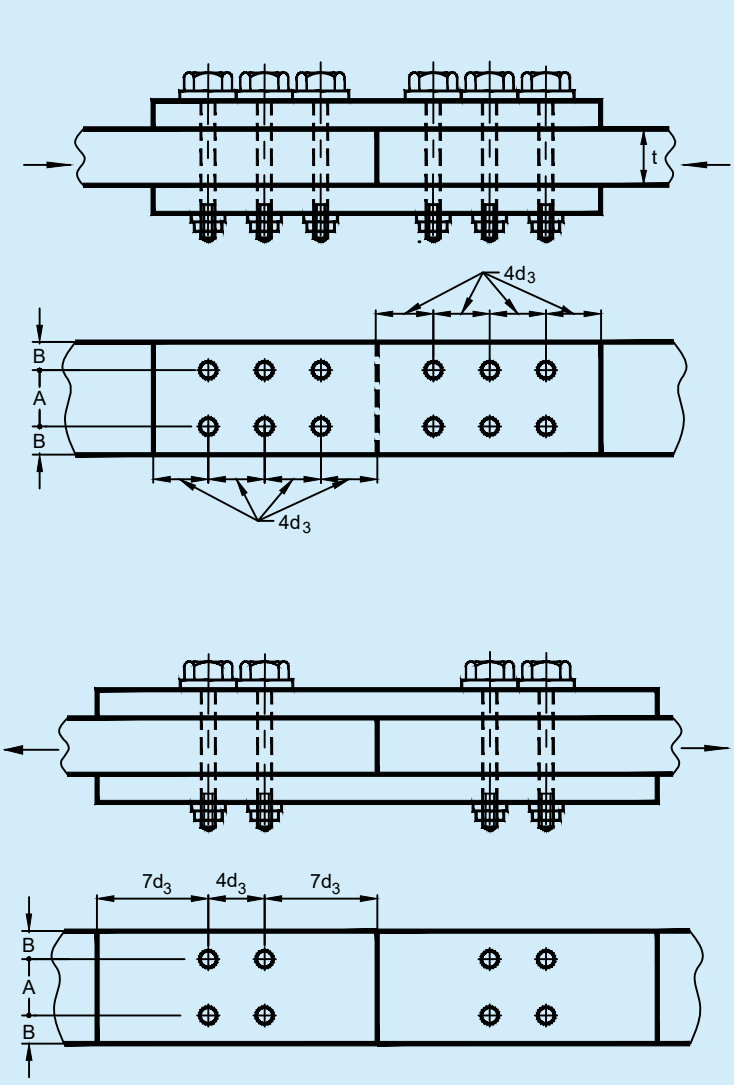

FIG. 9A SPACING OF BOLTS IN LENTHENING JOINTS
(JOINTS LOADED PARALLEL TO GRAIN)

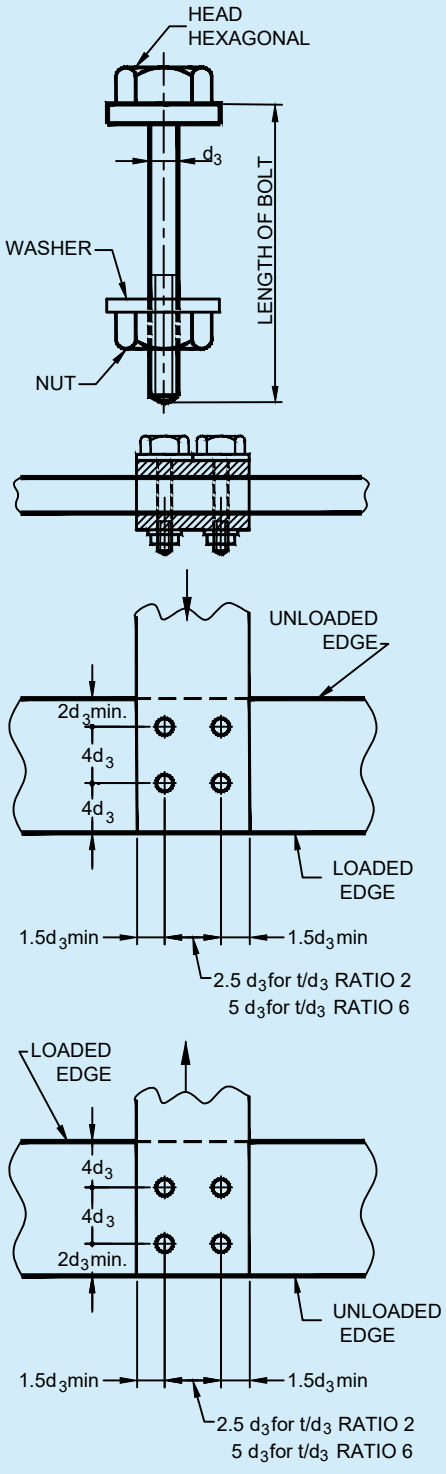

FIG. 9B SPACING OF BOLTS AT NODE JOINTS

FIG. 9 TYPICAL SPACING OF BOLTS IN STRUCTURAL JOINTS

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9.5 Fabrication

The fabrication of bolt jointed construction shall be in accordance with good practice [C-3A(9)].

To meet the specific requirements of 'all-timber construction' for structures where chemical corrosion of steel elements is a problem, and in order to erect anti-magnetic laboratories, sheds, etc, design should be made on the basis of mild steel bolt joints with the following reductions for timber bolts and bamboo pins:

Allowable safe working load per hardwood timber bolt	= 66 percent of the safe working load per mild steel bolt
Safe working load per solid bamboo pin	= 56 percent of the working load per mild bamboo per steel bolt

10 DESIGN OF TIMBER CONNECTOR JOINTS

10.1 In large span structures, the members have to transmit very heavy stresses requiring stronger jointing techniques with metallic rings or wooden disc-dowels. Improvised metallic ring connector is a split circular band of steel made from mild steel pipes. This is placed in the grooves cut into the contact faces of the timber members to be joined, the assembly being held together by means of a connecting bolt (*see* Fig. 10).

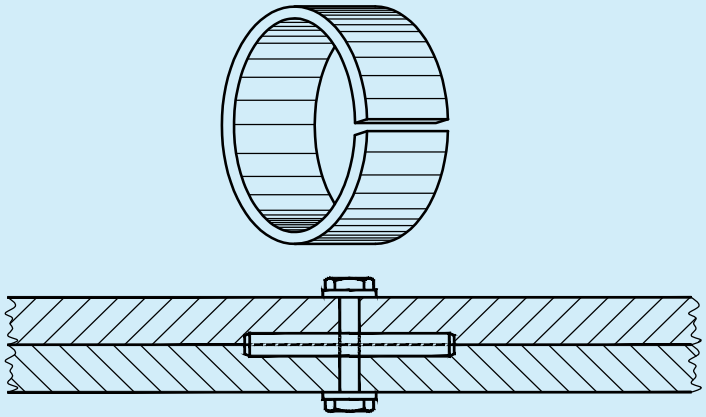

FIG. 10 SPLIT RING CONNECTOR

10.1.1 *Dimensions of Members*

Variation of thickness of central (main) and side members affect the load carrying capacity of the joint. The thickness of main member shall be at least 57 mm and that of side member 38 mm with length and width of members governed by placement of connector at joint.

The metallic connector shall be so placed that the loaded edge distance is not less than the diameter of the connector and the end distance not less than 1.75 times the diameter on the loaded side.

10.1.2 *Design Considerations*

The primary stresses in a split ring connector joint under tension are illustrated in Fig. 11. The shaded areas represent the part of wood in shear, compression and tension. Related formulae for the same are given in **10.1.2.1**.

For fabrication of structural members, a hole of the required size of the bolt is drilled into the member and a groove is made on the contact faces of the joint.

Theoretical safe loads in design shall be corroborated with sample tests done in accordance with a [C-3A(10)].

NOTE — A pilot study on determination of strength of improvised split ring connector joint in a compression test for a specific design problem conducted using *Mesua ferrea* (Mesua), yielded the data which is given below, for guidance:

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Sl No.	*No. and Diameter of Ring Used in a Joint*		*No. and Size of Bolt Used in a Joint*		*Side Member*		*Central Member*		*Load Direction with respect to Grains of Wood*	*End Distance*	*Inter-mediate Distance*	*Load Per Pair of Connector*
	No.	Size mm	No.	Size mm	Thickness mm	Width mm	Thickness mm	Width mm		mm	mm	kgf
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
i)	2	63	1	12 × 125	31	92	38	92	Parallel	63	–	3 930
ii)	2	63	1	12×125	31	92	38	92	Parallel	75	–	4 185
iii)	2	63	1	12 × 150	38	92	63	92	Parallel	75	–	4 010
iv)	2	63	1	12 × 150	38	117	63	117	Parallel	75	–	4 450
v)	2	63	1	12 × 125	31	138	38	92	Perpendicular	69	–	2 520
vi)	2	63	1	12 × 125	38	138	38	92	Perpendicular	69	–	3 515
vii)	2	100	1	19 × 175	38	138	66	138	Parallel	100	–	7 075
viii)	2	100	1	19 × 175	38	138	66	138	Parallel	125	–	7 370
ix)	2	100	1	19 × 175	41	138	75	138	Parallel	100	–	7 220
x)	2	100	1	19 × 175	41	138	75	138	Parallel	125	–	7 645
xi)	4	100	2	19 × 200	38	138	66	138	Parallel	100	150	5 655
xii)	4	100	2	19 × 200	41	138	75	138	Parallel	100	150	5 925
xiii)	4	100	2	19 × 200	41	138	75	138	Parallel	125	200	7 135

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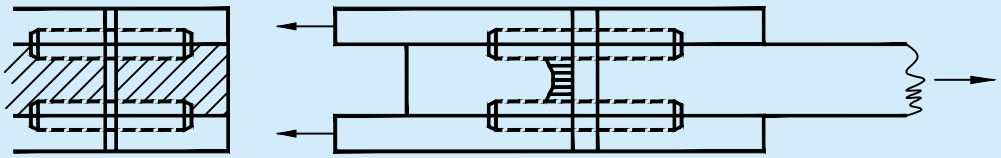
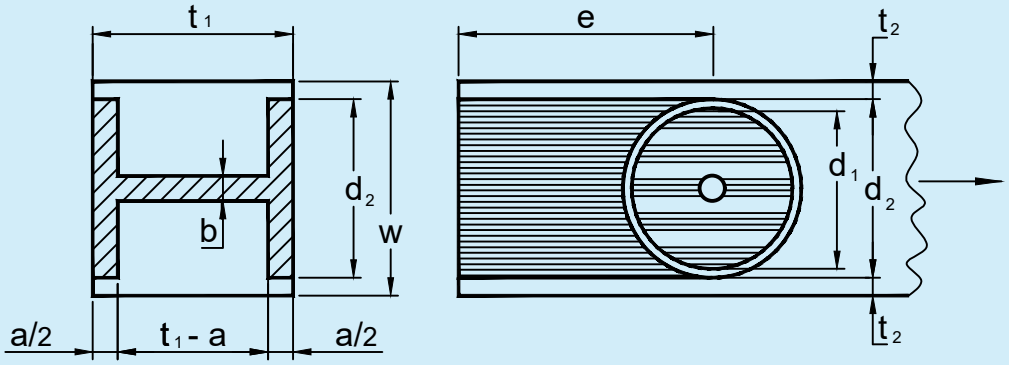


11A VERTICAL STRESS DITRBUTION



11B HORIZONTAL STRESS DITRBUTION

NOTE — Symbols are exclusive to this figure.

FIG. 11 STRESS DISTRIBUTION IN A SPLIT RING CONNECTOR

10.1.2.1 Strength of joint is controlled by one or another or combination of three properties:

a) Tension area (*see* Fig. 11A):

$$t_1 w - \left[2\left(\frac{ad_2}{2}\right) + b(t_1 - a)\right]$$

b) Compression area (*see* Fig. 11B):

$$2\left(\frac{ad_2}{2}\right) + b(t_1 - a)$$

c) Shear area (*see* Fig. 11B):

1) *Within core* : $2\left(\frac{\pi {d_1}^2}{4}\right)$

2) *Below core* : $2\left[d_2 e - \frac{1}{2}\left(\frac{\pi {d_2}^2}{4}\right) + 2\left(\frac{ae}{2}\right)\right]$

where

d_1 = inside diameter of connector;

t_1 = thickness of member;

w = width of member;

a = depth of connector;

d_2 = outside diameter of connector;

b = diameter of bolt;

e = end distance from centre of connector to end of member; and

t_2 = thickness of metal.

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10.2 Wooden Disc-Dowel

10.2.1 It is a circular hardwood disc generally tapered each way from the middle so as to form a double conical frustum. Such a disc is made to fit into preformed holes (recesses), half in one member and the other half in another, the assembly being held by one mild steel bolt through the centre of the disc to act as a coupling for keeping the jointed wooden members from spreading apart.

As timber always fails by shearing or crushing in front of bolts, the function of disc-dowel incorporated in the joint is to distribute the shearing stresses on a larger area of wood so as to transmit the full amount of stresses.

10.2.2 *Dimensions of Members*

The thickness of dowel may vary from 25 mm to 35 mm and diameter from 50 mm to 150 mm. The diameter of dowel shall be 3.25 to 3.50 times the thickness.

The edge clearance shall range from 12 mm to 20 mm as per the size of the dowel. The end clearance shall be at least equal to the diameter of dowel for joints subjected to tension and three-fourth the diameter for compression joints. Disc dowel shall be turned from radial (quarter sawn) planks of seasoned material.

10.2.3 *Choice of Species*

Wood used for making dowels shall be fairly straight grained, free from excessive liability to shrink and warp, and retain shape well after seasoning and having high shear strength. Species recommended include:

a) Babul;
b) Dhaman;
c) Irul;
d) Sissoo;
e) Rose wood;
f) Sandal;
g) Axle-wood;
h) Padauk; and
j) Yon.

NOTE – Data on the above species are as per Table 1.

10.2.4 *Design Considerations*

The forces on dowel in a lap joint and butt joint are illustrated in Fig. 12. Dowel is subjected to shearing at the mid-section, and compression along the grain at the bearing surfaces. For equal strength in both the forces, formula equations are given in the Fig. 12 to determine the size of dowel.

The making of wooden discs may present some problems in the field, but they may be made in small workshop to the specifications of the designer. This is also economically important. Once the wood fittings are shop tailored and made, the construction process in the field is greatly simplified.

Theoretical safe loads in design shall be confirmed through sample tests.

NOTE — Some experimental studies have indicated the following safe loads, in kN, for dowels bearing parallel to the grain, which are given here for guidance only.

Sl No.	*Species*		*Safe Loads for Dowels Bearing Parallel to the Grain, in kN* *Size of Dowel* (mm)					
	Members	Dowels	62 × 25	75 × 25	87 × 25	100 × 31	112 × 37	125 × 37
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	Sal	Babul	6.671	9.810	13.342	17.805	22.269	27.566
ii)	Sal	Sissoo	5.346	7.554	10.251	13.342	16.922	20.895

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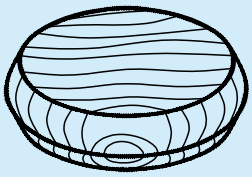
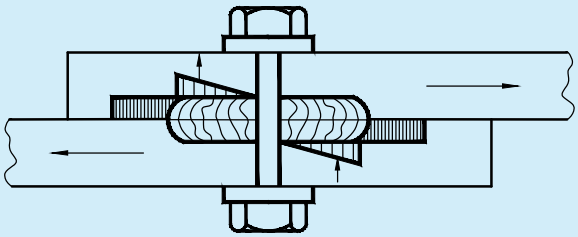
10.2.4.1 Rigid specifications regarding use of particular planks for fabrication of wooden disc-dowels, and the specific alignment of grains to be in line with grains of members forming the joint, can be relaxed. Application of appropriate factors in design of such joints whatever be the angle of load and members, etc, would eliminate risks involved by providing 60 percent more number of dowels at a joint. Thus, decrease the calculated load per dowel to 60 percent.

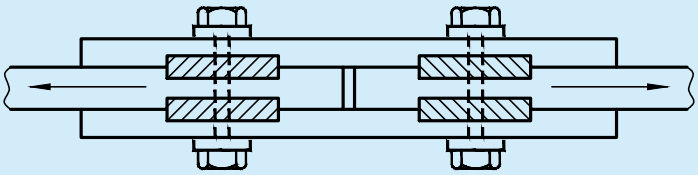
10.2.4.2 Analysis of variance for safe load in case of dowel joints made with MS bolt, timber bolt, bamboo pin as joiners showed no significant difference. Thus, the concept of 'all-timber construction' hold good for chemical industries where corrosion due to acid fumes is a recurring problem.



DOWEL



LAP JOINT



BUTT JOINT

Lap joint — Bolt in simple tension due to clockwise turning moment on dowel.

Butt joint — No tilting moment in dowel due to balancing effect [dowels are in shear (no bending, shearing and tensile stress on bolts)]

Size of dowel for equal strength in both shearing and bearing.

$$\frac{\pi d^2}{4} \times S = d \times \frac{t}{2} \times c$$

where

d = mid diameter of the dowel;

t = thickness of dowel;

s = safe working stress in shear along grain; and

c = safe compressive stress along grain.

NOTE — Symbols are exclusive to this figure.

FIG. 12 DISTRIBUTION OF FORCES IN DOWEL JOINT

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11 GLUED LAMINATED CONSTRUCTION

11.1 Developments in the field of synthetic adhesive have brought gluing techniques within the range of engineering practice. Timber members of larger cross-sections and long lengths can be fabricated from small sized planks by the process of glulam. The term glued laminated timber construction as applied to structural members refers to various laminations glued together, either in straight or curved form, having grain of all laminations essentially parallel to the length of the member.

11.1.1 *Choice of Glue*

The adhesive used for glued laminated assembly are 'gap filling' type. A 'filler' in powder form is introduced in the adhesive. Structural adhesives are supplied either in powder form to which water is added or in resin form to which a hardener or catalyst is added. For choice of glues, reference may be made to good practice [C-3A(11)]. However, it is important that only boiling water proof (BWP) grade adhesives shall be used for fabrication of glulam in tropical, high humid climates like India.

11.1.2 *Manufacturing Schedule*

In absence of a systematic flow-line in a factory, provisions of intermediate technology shall be created for manufacturing structural elements. The schedule involves the following steps:

a) Drying of planks;

b) Planning;

c) End-jointing by scarfs or fingers;

d) Machining of laminations;

e) Setting up dry assembly of structural unit;

f) Application of glue;

g) Assembly and pressing the laminations;

h) Curing the glue lines, as specified; and

j) Finishing, protection and storage.

11.1.3 The information with regard to block shear test results on glued timber joints is given in Table 18.

Table 18 Block Shear Test Results on Glued Timber Joints

(*Clause* 11.1.3)

Sl No.	Timber Species	Ultimate Shear Stress (Inside Location) N/mm^2		
		Solid Timber Along the Grain (Un-Jointed)	With Urea Formaldehyde	With Phenol Formaldehyde
(1)	(2)	(3)	(4)	(5)
i)	*Abies pindrow* (Fir)	5.984	7.652	9.614
ii)	*Acacia catechu* (Khair)	15.107	7.456	–
iii)	*Acacia milotica* (Babul)	14.126	5.199	3.237
iv)	*Albizia odoratissima* (Karasiris)	15.009	14.126	5.984
v)	*Azadirachta indica* (Neem)	12.557	7.259	8.240
vi)	*Casteanopsis hystrix* (Chestnut)	8.044	1.668	–
vii)	*Cedrus deodar* (Deodar)	6.867	4.218	3.237

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Table 18 (*Concluded*)

Sl No.	Timber Species	Ultimate Shear Stress (Inside Location) N/mm^2		
		Solid Timber Along the Grain (Un-Jointed)	With Urea Formaldehyde	With Phenol Formaldehyde
(1)	(2)	(3)	(4)	(5)
viii)	*Dalbergia sissoo* (Sissoo)	10.595	15.107	9.320
ix)	*Eucalyptus eugenioides* (Eucalyptus)	12.164	10.987	5.101
x)	*Grewia tiliefolia* (Dhawan)	8.927	10.889	6.475
xi)	*Hopea utilis* (Karunkangoo)	14.813	9.320	–
xii)	*Hopea glabra* & *H. parviflora* (Hopea)	15.009	13.832	0.491
xiii)	*Mangifera Indica* (Mango)	9.418	9.418	3.826
xiv)	*Madhuca longifolia* (Mahua)	9.810	0.883	7.456
xv)	*Mesua ferrea* (Mesua)	12.164	1.079	2.256
xvi)	*Morus alba* (Mulberry)	9.810	13.734	8.142
xvii)	*Pinus wallichiana* (Kail)	5.494	6.377	7.259
xviii)	*Pinus roxburghii* (Chir)	6.082	6.769	7.750
xix)	*Poecilone indicum* (Ballagi)	14.911	5.297	5.984
xx)	*Pterocarpus mersupium* (Bijasal)	9.221	5.003	6.671
xxi)	*Quercus spp.* (Oak)	8.142	10.791	5.396
xxii)	*Ougeinia oojeinensis* (Sanetan)	13.440	5.396	9.516
xxiii)	*Shorea robusta* (Sal) M.P	9.123	5.003	–
xxiv)	*Shorea robusta* (Sal) UP	12.361	5.396	1.472
xxv)	*Sterospermum chelonoides* (Padriwood)	11.674	9.221	8.829
xxvi)	*Syzygium cuzainii* (Jaman)	10.791	9.418	10.301
xxvii)	*Terminalia alata* (Sain)	9.320	13.538	8.142
xxviii)	*Tectona grandis* (Teak)	10.987	5.984	6.377
xxix)	*Terminalia myriocarpa* (Hollock)	8.339	9.221	9.810
xxx)	*Toona ciliata* (Toon)	6.867	4.905	6.180

NOTE — The information given in the above table is based on the block shear test results on glued timber joints conducted on the basis of BS 4169 : 1970 'Specification for manufacture of glued-laminated timber structural members' (since revised and subsequently superseded by BS EN 392 : 1995, BS EN 391 : 1995, BS EN 386 : 1995) and the failures observed with regard to glue line and the wood. Ultimate shear stresses for 29 species were studied keeping design glue shear strength at least one-third of ultimate. Glue mixing, spreading, pressing and curing was done as per the instructions of adhesive manufacturers under improvised techniques locally developed. Sustainable period before delamination of glue bond (urea formaldehyde) in weathering tests was recorded for over a year. Limited number of species survived with most of them failed within a period of six months.

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11.2 Design of Glued Laminated Beams

11.2.1 *General*

Glued laminated structural members shall be fabricated only where there are adequate facilities for accurate sizing and surfacing of planks, uniform application of glue, prompt assembly, and application of adequate pressure and prescribed temperature for setting and curing of the glue. Design and fabrication shall be in accordance with established engineering principles and good practice. A glued laminated beam is a straight member made from a number of laminations assembled both ways either horizontally or vertically (*see* Fig. 13). While vertical laminations have limitations in restricting the cross section of a beam by width of the plank, horizontally laminated section offers wider scope to the designer in creating even the curved members. Simple straight beams and joists are used for many structures from small domestic rafters or ridges to the light industrial structures.

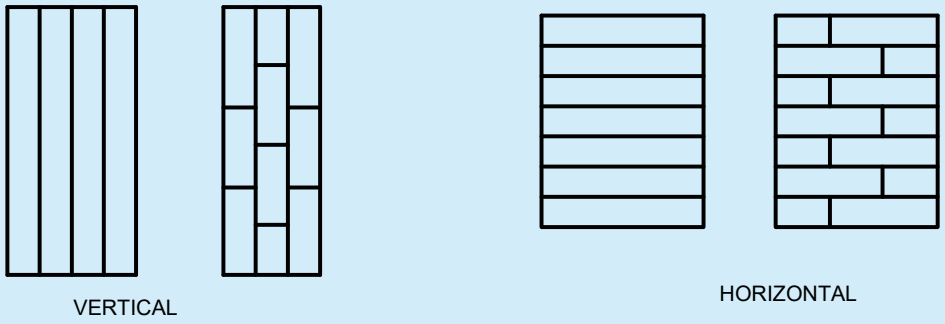

FIG. 13 POSSIBLE ORIENTATION OF PLANKS IN GLULAM

11.2.2 *Design*

The design of glue laminated wood elements shall be in accordance with good engineering practice and shall take into consideration the species and grade of timber used, presence of defects, location of end joints in laminations, depth of beams and moisture contents expected while in service. Beams of large spans shall be designed with a suitable camber to assist in achieving the most cost effective section where deflection governs the design. The strength and stiffness of laminated beams is often governed by the quality of outer laminations. Glued laminated beams can be tapered to follow specific roof slopes across a building and/or to commensurate with the varying bending moments. Glulam allows the engineers to mix grades within a beam. Obviously, the lower tensile portion should be of high grade material. However, it is possible to incorporate a considerable volume of lower grade material near the neutral axis taking care of the horizontal shear stress. Similarly, species can also be mixed depending upon the modulus of elasticity.

Straight beams that will be simply supported should be cambered during fabrication to allow for deflection due to dead loads over a period of time. The usual camber is $\frac{span}{400}$, unless otherwise specified.

11.2.2.1 *Design outline for horizontally laminated beams*

A design outline for horizontal laminated beams, specifying data required and a possible solution is given below.

a) *Data required:*

1) Span (L), loads including self weight of beam (W), species of timber, thickness of lamination proposed; and

2) Mechanical properties of timber species (permissible extreme fibre stress in bending-tension, permissible compressive stress perpendicular to grain, modulus of elasticity, permissible shear stress parallel to grain).

The permissible shear stress at the glue line is at least the same as the permissible shear stress parallel to grain of the species in design, unless otherwise tested.

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b) *Solution:*

1) Let, b = width of beam, and d = depth of beam, for lateral stability, $b : d = 1 : 4$ (*say*).
2) Determine bending moment on the basis of span, loading and support conditions (say, $WL/8$ for beams with both ends fixed).
3) Also, bending moment (M) = Bending stress (f_b) × Section modulus (Z):

$$\text{Therefore, } M = f_b \times \frac{bd^2}{6} = \frac{WL}{8}$$

$$\text{or } bd^2 = \frac{WL}{8} \times \frac{6}{f_b}$$

4) From bd^2 obtained at (c) and $b : d$ assumed at (a), arrive at appropriate values of b and d.
5) Check for deflection of beam. Since dead load is for long term, double this for deflection check, that is:

$$W = (2 \times \textit{Dead load}) + \textit{Imposed load}$$

Deflection is to be limited to the specified requirement, say 0.003 of span.

Substituting values of b and d from (d) and I ($= bd^3/12$) in the formula for deflection, perform check against the limit for deflection.

6) Decide the number and thickness of laminations for the required depth of section.

11.2.3 *Material*

Laminating boards shall not contain decay, knots or other strength reducing characteristics in excess of those sizes or amounts permitted by specifications. The moisture content shall approach that expected in service and shall in no case exceed 15 percent at the time of gluing. The moisture content of individual laminations in a structural member shall not differ by more than 3 percent at the time of gluing. Glue shall be of type suitable for the intended service of a structural member.

11.2.4 *Fabrication/Manufacture*

In order to assure a well-bonded and well-finished member of true shape and size, all equipment, end-jointing, glue spread, assembly, pressing, curing or any other operation in connection with the manufacture of glued structural members shall be in accordance with the available good practices and as per glue manufacturers' instructions as applicable.

11.2.4.1 End joints shall be scattered in adjacent laminations, which shall not be located in very highly stressed outer laminations.

11.2.5 *Testing*

Quality control, inspection and testing during production is essential to structural glulam. Special attention needs to be given to members exposed to weather.

For examining the quality of glue and its relative strength namely species of timber in glued laminated construction, it is necessary to conduct block shear and other related tests in accordance with accepted standard [C-3A(11)]. Structural loading tests on prototype sizes provide information on the strength properties, stiffness or rigidity against deflection of a beam. Information on different strength functions as obtained from such indigenous tests carried out on laminated timber beams are given in Table 19, for guidance in design.

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Table 19 Laminated Timber Beams — Structural Bending Test Under Three Point Loading

(*Clause* 11.2.5)

Species — *Pinus wallichiana* (Kail);
Effective span — 2.3 m, Cross section: 100 × 240 mm; and
Adhesive — Urea formaldehyde.

Sl No.	Strength Functions	Nail Laminated Vertical Laminates with Four Full Size Planks Using 8 SWG, 100 mm Long Nails	Glue Laminated		Solid (Control) Specimen
			Vertical Laminates Four Planks, Each 25 mm Thick	Horizontal Laminates Eight Planks, Each 30 mm Thick	
(1)	(2)	(3)	(4)	(5)	(6)
i)	Modulus of rupture, N/mm^2	27.439	31.363	21.568	30.509
ii)	Modulus of elasticity ($\times 10^3$ N/mm^2)	8.167	8.903	8.210	7.554
iii)	Fibre stress at limit of proportionality, N/mm^2	18.136	14.166	13.259	18.276
iv)	Maximum horizontal shear stress, N/mm^2	1.723	1.967	1.319	1.906

12 LAMINATED VENEER LUMBER

12.1 Certain reconstituted lignocellulosic products with fibre oriented along a specific direction have been developed and are being adopted for load bearing applications. Laminated veneer lumber is one such product developed as a result of researches in plantation grown species of wood. Density of laminated veneer lumber ranges from 600 kg/m^3 to 750 kg/m^3 which is manufactured in accordance with accepted standard [C-3A(12)].

12.1.1 *Dimensions*

Sizes of laminated veneer lumber composite shall be inclusive of margin for dressing and finishing unless manufactured to order. The margin for dressing and finishing shall not exceed 3 mm in the width and thickness and 12 mm in the length.

12.1.2 *Permissible Defects*

a) Jointing gaps : Not more than 3 mm wide, provided they are well staggered in their spacing and position between the successive plies;

b) Slope of grain : Not exceeding 1 in 10 in the face layers;

c) Tight knot : Three numbers up to 25 mm diameter in 1 m^2 provided they are spaced 300 mm or more apart; and

d) Warp : Not exceeding 1.5 mm per metre length.

12.1.3 *Strength Requirements*

The strength requirements for laminated veneer lumber shall be as per Table 20.

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Table 20 Requirements of Laminated Veneer Lumber

(*Clause* 12.1.3)

Sl No.	Properties	Requirement
(1)	(2)	(3)
i)	Modulus of rupture, N/mm^2, *Min*	50
ii)	Modulus of elasticity, N/mm^2, *Min*	7 500
iii)	Compressive strength parallel to grain, N/mm^2, *Min*	35
iv)	Horizontal shear:	
	a) Parallel to laminae, N/mm^2, *Min*	6
	b) Perpendicular to laminae, N/mm^2, *Min*	8
v)	Tensile strength parallel to grain, N/mm^2, *Min*	55
vi)	Screw holding power:	
	a) Edge (N), *Min*	2 300
	b) Face (N), *Min*	2 700
vii)	Thickness swelling in 2 h water soaking (percent), *Max*	3

13 GLUED FINGER JOINTS

13.1 Finger joints are glued joints connecting timber members end-to-end (*see* Fig. 14). Such joints shall be produced by cutting profiles (tapered projections) in the form of v-shaped grooves to the ends of timber planks or scantling to be joined, gluing the interfaces and then mating the two ends together under pressure. Finger joints provide long lengths of timber, ideal for upgrading timber by permitting removal of defects, minimizing warping and reducing wastage by avoiding short off-cuts. To improve the long-term durability and performance of glued finger joints, the timber used should be well-seasoned to maintain dimensional stability. Additional protective treatments may be required for finger-jointed timber used in outdoor or high-moisture environments to prevent decay and degradation of the adhesive bonds.

13.1.1 In finger joints the glued surfaces are on the side grain rather than on the end grain and the glue line is stressed in shear rather in tension.

13.1.1.1 The fingers can be cut from edge-to-edge or from face-to-face. The difference is mainly in appearance, although bending strength increases if several fingers share the load. A joist is slightly stronger with edge-to-edge finger joints and a plank is stronger with face-to-face finger joint. The choice of adhesive should consider the wood species being used, as different timber species may exhibit varying adhesive compatibility. For example, denser species may require stronger bonding adhesives, while lower-density woods may work well with more flexible adhesives. Testing of specific adhesives with the wood species used is recommended to ensure optimal joint strength and performance.

13.1.1.2 For structural finger jointed members for interior dry locations, adhesives based on melamine formaldehyde cross linked polyvinyl acetate (PVA) are suited. For high humid and exterior conditions, phenol formaldehyde and resorcinol formaldehyde type adhesives are recommended. Proper adhesives should be selected in consultation with the designer and adhesive manufacturers and assessed in accordance with accepted standard [C-3A(11)]. The environmental impact of adhesives used in finger-jointed construction should also be considered. Where possible, the use of low- volatile organic compounds (VOC) or environmentally friendly adhesives should be promoted, particularly in indoor applications, to reduce emissions and promote healthier indoor air quality.

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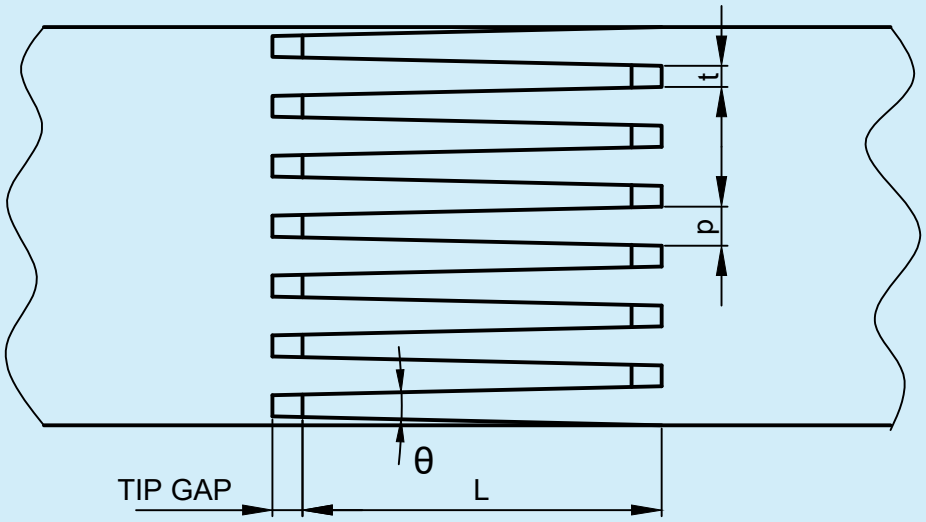
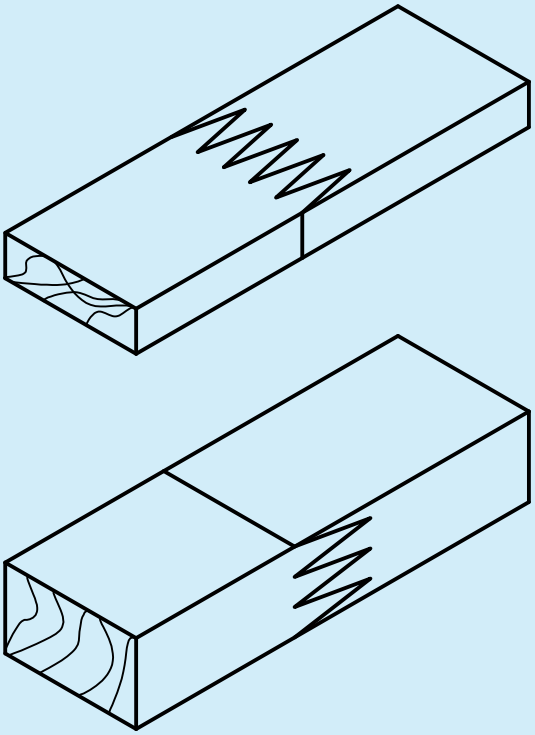


L = FINGER LENGTH
p = PITCH
t = TIP THICKNESS
θ = 6°



ORIENTATION OF FINGER JOINTS

FIG. 14 TYPICAL FINGER JOINT GEOMETRY

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13.1.2 *Manufacturing Process*

In the absence of sophisticated machinery, the finger joints shall be manufactured through intermediate technology with the following steps:

a) Drying of wood;

b) Removal of knots and other defects;

c) Squaring the ends of the laminating planks;

d) Cutting the profile of finger joint in the end grain;

e) Applying adhesives on the finger interfaces;

f) Pressing the joint together at specified pressure;

g) Curing of adhesive line at specified temperature; and

h) Planing of finger-jointed planks for smooth surface.

For higher precision and productivity, modern manufacturing facilities could implement computer numerical control (CNC) machines for cutting finger joints. Automation helps achieve consistent profile geometry, improves the quality of adhesive application, and reduces the margin for human error during production.

13.1.3 *Strength*

Strength of finger joints depends upon the geometry of the profile for structural purpose; this is generally 50 mm long, 12 mm pitch.

13.1.3.1 The strength properties of glued finger joints are given in Table 21 for information and guidance in design.

13.1.3.2 Tip thickness shall be as small as practically possible.

14 STRUCTURAL USE OF PLYWOOD

Unlike sawn timber, plywood is a layered panel product comprising veneers of wood bonded together with adjacent layers usually at right angles. As wood is strongest when stressed parallel to grain, and weak perpendicular to grain, the lay up or arrangement of veneers in the panel determines its properties. When the face grain of the plywood is parallel to the direction of stress, veneers parallel to the face grain carry almost all the load. Some information/guidelines for structural use of plywood which should be manufactured in accordance with accepted standards [C-3A(13)] are given in **14.1** to **14.3**.

14.1 The plywood has a high strength to weight ratio and is dimensionally stable material available in sheets of a number of thicknesses and construction. Plywood can be sawn, drilled and nailed with ordinary wood working tools. The glues used to bond these veneers together are derived from synthetic resins which are set and cured by heating. The properties of adhesives can determine the durability of plywood.

14.2 In glued plywood construction, structural plywood is glued to timber resulting in highly efficient and light structural components like web beams (I and box sections) (*see* Fig. 15 and Fig. 16) stressed skin panels (*see* Fig. 17) used for flooring and walling and pre-fabricated houses, cabins, etc. Gluing can be carried out by nail gluing techniques with special clamps. High shear strength of plywood in combination with high flexural strength and stiffness of wood result in structures characterized by high stiffness for even medium spans. Plywood can act as web transmitting shear stress in web bearing or stressed skin or sandwich construction. The effective moment of inertia of web beam and stressed skin construction depends on modular ratio that is, E of wood to E of plywood.

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Table 21 Strength Characteristics of Glued Finger Joints

(*Clause* 13.1.3.1)

Timber species – *Pinus wallichiana* (Kail)

(Averaged test results)

Sl No.	Character → / Treatment ↓	Compressive Parallel to Grain Tests			Tension Parallel to Grain Tests			Static Bending Tests											
								Central Point Loading						Two Point Loading					
								Edge-wise			Flat-wise			Edge-wise			Flat-wise		
		Maximum Compressive Stress (N/mm^2)	Fibre Stress at EL (N/mm^2)	MOE ($\times 10^3$ N/mm^2)	Maximum Tensile Stress (N/mm^2)	Tensile Stress at EL (N/mm^2)	MOE ($\times 10^3$ N/mm^2)	Fibre Stress at EL (N/mm^2)	MOR (N/mm^2)	MOE ($\times 10^3$ N/mm^2)	Fibre Stress at EL (N/mm^2)	MOR (N/mm^2)	MOE ($\times 10^3$ N/mm^2)	Fibre Stress at EL (N/mm^2)	MOR (N/mm^2)	MOE ($\times 10^3$ N/mm^2)	Fibre Stress at EL (N/mm^2)	MOR (N/mm^2)	MOE ($x10^3$ N/mm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
i)	UL_1	31.196	16.971	10.202	30.019	23.446	10.202	20.993	29.136	6.769	39.142	46.401	8.142	25.114	28.743	8.044	32.471	38.553	9.516
ii)	UL_2	37.474	17.462	11.282	30.607	23.642	10.301	20.209	31.686	6.671	35.120	46.205	7.161	23.446	42.085	7.652	41.202	51.306	9.712
iii)	UL_3	35.905	19.914	10.595	32.667	23.838	10.006	22.171	35.022	6.867	38.750	47.480	8.437	25.898	29.234	7.848	29.626	38.357	8.633
iv)	PL_1	28.743	21.484	9.908	20.895	17.560	10.497	20.699	23.446	8.142	22.955	27.076	9.123	18.443	21.876	7.848	25.506	27.174	8.927
v)	PL_2	28.743	19.424	10.006	18.149	17.069	9.614	16.285	18.639	5.984	24.133	26.095	8.731	16.579	17.854	7.259	26.389	28.547	8.535
vi)	PL_3	27.860	20.111	9.908	24.721	19.326	10.497	15.009	17.069	7.652	21.680	26.193	8.829	10.301	11.183	6.867	26.487	30.313	11.183
vii)	C	35.512	27.959	10.301	31.490	23.152	10.104	25.506	45.518	7.063	36.788	57.192	7.063	31.883	44.537	7.946	38.161	52.582	9.221

MOE – Modulus of elasticity (values rounded off for decimals)
MOR – Modulus of rupture
EL – Elastic limit
U – Urea formaldehyde with liquid hardener
P – Cold setting phenolic glue with hardener
L_1 – Finger length 38 mm
L_2 – Finger length 50 mm
L_3 – Finger length 62 mm
C – Control (unjointed specimen)

NOTE — The information given in the above table is based on the tests conducted on glued finger joints of varying lengths/pitch with two types of adhesives.

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14.3 Structural plywood is also very efficient as cladding material in wood frame construction such as houses. This type of sheathing is capable of resisting racking due to wind and earthquake forces. Structural plywood has been widely used as diaphragm (horizontal) as in roofing and flooring in timber frame construction. It has been established that 6 mm thick plywood can be used for sheathing and even for web and stressed skin construction, 9 mm to 12 mm thick plywood is suitable for beams, flooring diaphragms, etc. Phenol formaldehyde (PF) and PRF adhesive are suitable for fabrication of glued plywood components. 6 mm to 12 mm thick structural plywood can be very well used as nailed or bolted gussets in fixing members of trusses or lattice girders or trussed rafters.

Normally, scarf joints are used for fixing plywood to required length and timber can be joined by using either finger or scarf joints. Arch panels, folded plates, shelves are other possibilities with this technique.

15 TRUSSED RAFTER

15.1 General

A roof truss is essentially a plane structure which is very stiff in the plane of the members, that is the plane in which it is expected to carry loads, but very flexible in every other direction. Thus, it can virtually be seen as a deep, narrow girder liable to buckling and twisting under loads. In order, therefore, to reduce this effect, eccentricity of loading and promote prefabrication for economy, low-pitched trussed rafters are designed with bolt ply/nail ply joints. Plywood as gussets, besides being simple have inherent constructional advantage of grain over solid wood for joints and a better balance is achievable between the joint strength and the member strength.

Trussed rafters are light weight truss units spaced at close centres for limited spans to carry different types of roof loads. They are made from timber members of uniform thickness fastened together in one plane. The plywood gussets may be nailed or glued to the timber to form the joints. Conceptually a trussed rafter is a triangular pin jointed system, traditionally meant to carry the combined roof weight, cladding services and wind loads. There is considerable scope for saving timber by minimizing the sections through proper design without affecting structural and functional requirements.

Trussed rafters are required to be supported only at their ends so that there is no need to provide load bearing internal walls, purlins, etc. In comparison with traditional methods of construction they use less timber and considerably reduce on-site labour and mass production of reliable units can be carried out under workshop controls.

Additional consideration may include:

a) *Climatic Suitability* — Trussed rafters should be designed with consideration for India's diverse climatic zones. Timber species suitable for humid, dry and coastal regions should be preferred to prevent issues related to swelling, shrinkage, or decay due to moisture variations;

b) *Use of Local Timber Species* — Indian species such as Teak (*Tectona grandis*), Sal (*Shorea robusta*), Neem (*Azadirachta indica*), and other durable hardwoods should be prioritized. The use of locally available plantation-grown timber helps reduce transportation costs and supports sustainability; and

c) *Fire and Termite Resistance* — Due to the prevalence of termites in many regions of India, timber rafters should be treated with appropriate termite-resistant chemicals. Similarly, the risk of fire in construction should be mitigated through the application of fire-retardant coatings.

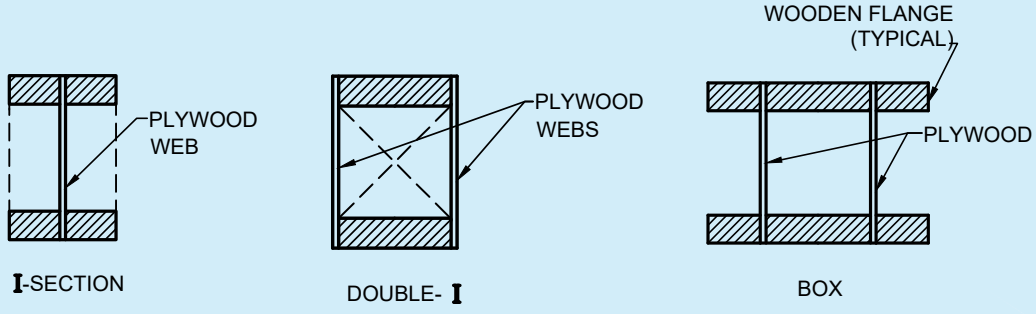

FIG. 15 TYPICAL CROSS SECTION OF WEB BEAMS

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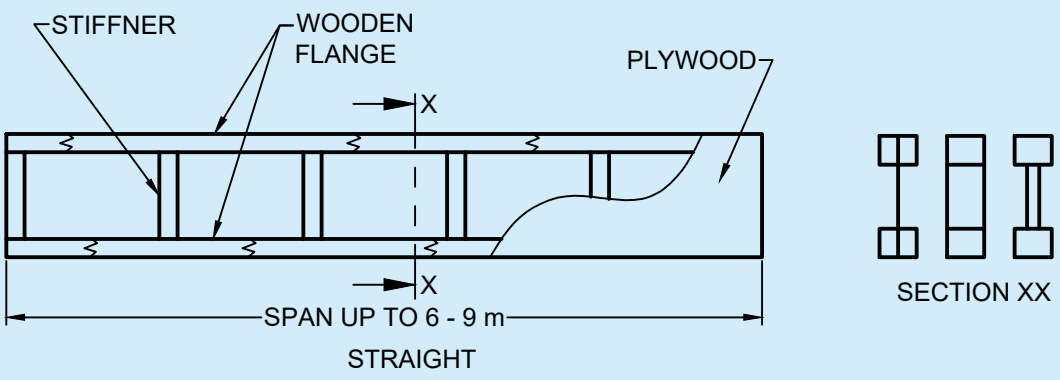

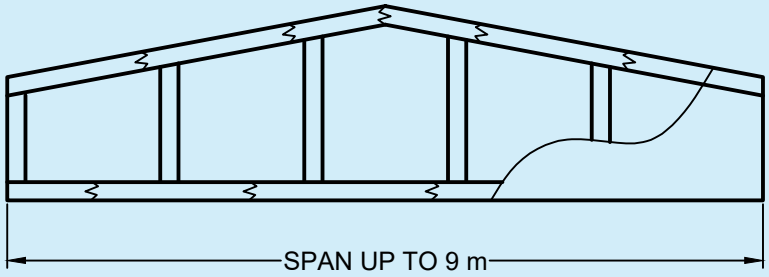

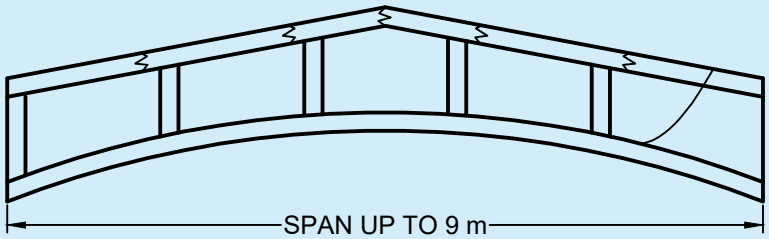

FIG. 16 WEB BEAM CONFIGRATIONS

FIG. 16 WEB BEAM CONFIGURATIONS

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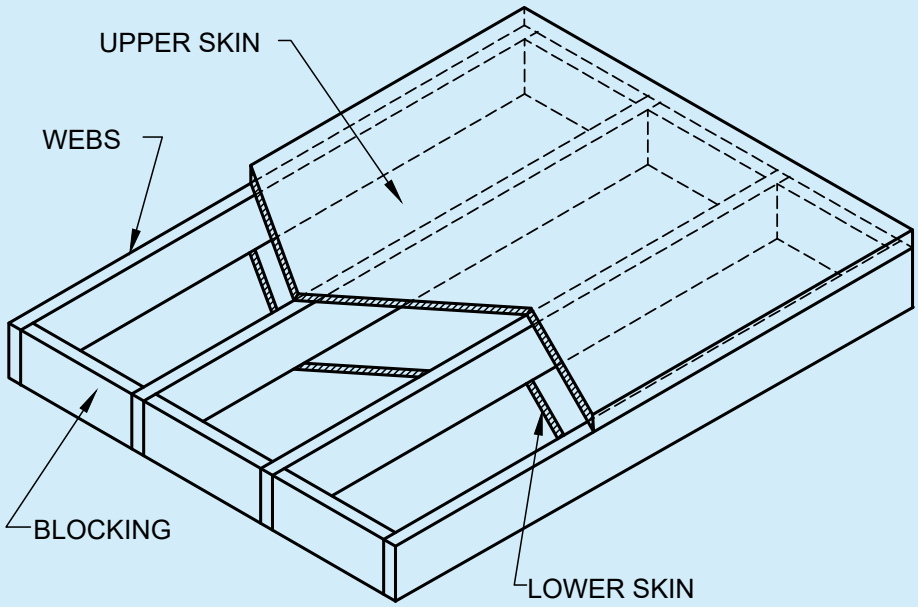

FIG. 17 STRESSED SKIN PANEL CONSTRUCTION (SINGLE SKIN OR DOUBLE SKIN)

15.2 Design

15.2.1 The design may be based on the use of Group 'C' species of standard grade timber in accordance with good practice [C-3A(7)] for nail jointed timber construction. As the strength and stiffness of any framing depends upon efficiency of joints, design calls for proper size and number of fasteners, their spacings and thickness of gusset plates for transfer of stress from one member to the other.

Trussed rafter shall be designed to sustain the dead and imposed loads specified in Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS and the combinations expected to occur. Extra stresses/ deflections during handling, transportation and erection shall be taken care of. Structural analysis, use of load-slip and moment, rotation characteristics of the individual joints may be used if feasible. Alternatively, the maximum direct force in a member may be assessed to be given by an idealized pin-jointed framework, fully loaded with maximum dead and imposed load in the combination in which they may reasonably be expected to occur.

NOTE — Some trussed rafters with plywood gusseted joint, span 6 m, designed in *Pinus roxburghii* (Chir) were tested in prototypes at Forest Research Institute, Dehradun for structural adequacy as under, in accordance with accepted standards [C-3A(14)]:

a) Destructive test (short-term); and

b) Proof test (long-term).

Results demonstrated an apparent factor at safety of 4 and actual factor of safety of 3. Deflection was well within the allowable deflection at design load. A pre-camber of 1/240 of span at the bottom chord was recommended.

15.3 Timber

The species of timber including plantation grown species which can be used for trussed rafter construction and permissible stresses thereof shall be in accordance with Table 1. Moisture contents shall be as per zonal requirements in accordance with **4.4**.

15.4 Plywood

Boiling water proof (BWP) grade preservative treated plywood shall be used in accordance with accepted standards [C-3A(13)]. Introduction of a plywood gusset simplifies the jointing and in addition provides rigidity to the joint. Preservation of plywood and other panel products shall be done in accordance with good practice [C-3A(15)].

15.5 Some design data on the lateral load bearing strength of common wire nails in plywood-to-wood joints are presented for timber species, Sal and Chir in Table 22, for guidance in design.

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16 STRUCTURAL SANDWICHES

16.1 General

Sandwich constructions are composites of different materials including wood based materials formed by bonding two thin facings of high strength material to a light weight core which provides a combination of desirable properties that are not attainable with the individual constituent materials (*see* Fig. 18). The thin facings are usually of strong dense material since that are the principal load carrying members of the construction. The core should be stiff enough to ensure the faces remain at the correct distance apart. The sandwiches used as structural elements in building construction shall be adequately designed for their intended services and shall be fabricated only where there are adequate facilities for gluing or otherwise bonding cores to facings to ensure a strong and durable product. The entire assembly provides a structural element of high strength and stiffness in proportion to its mass.

Non-structural advantages can also be derived by proper selection of facing and core material, for example, an impermeable facings can be used to serve as a moisture barrier for walls and roof panels and core may also be selected to provide thermal and/or acoustic insulation, fire resistance, etc, besides the dimensional stability.

16.2 Cores

Sandwich cores shall be of such characteristics as to give to the required lateral support to the stressed facings to sustain or transmit the assumed loads or stresses. Core generally carries shearing loads and to support the thin facings due to compressive loads. Core shall maintain the strength and durability under the conditions of service for which their use is recommended. A material with low *E* and small shear modulus may be suitable.

16.3 Facings

Facings shall have sufficient strength and rigidity to resist stresses that may come upon them when fabricated into a sandwich construction. They shall be thick enough to carry compressive and tensile stresses and to resist puncture or denting that may be expected in normal usages.

16.4 Designing

Structural designing may be comparable to the design of I-beams, the facings of the sandwich represent the flanges of the I-beam and the sandwich core I-beam web.

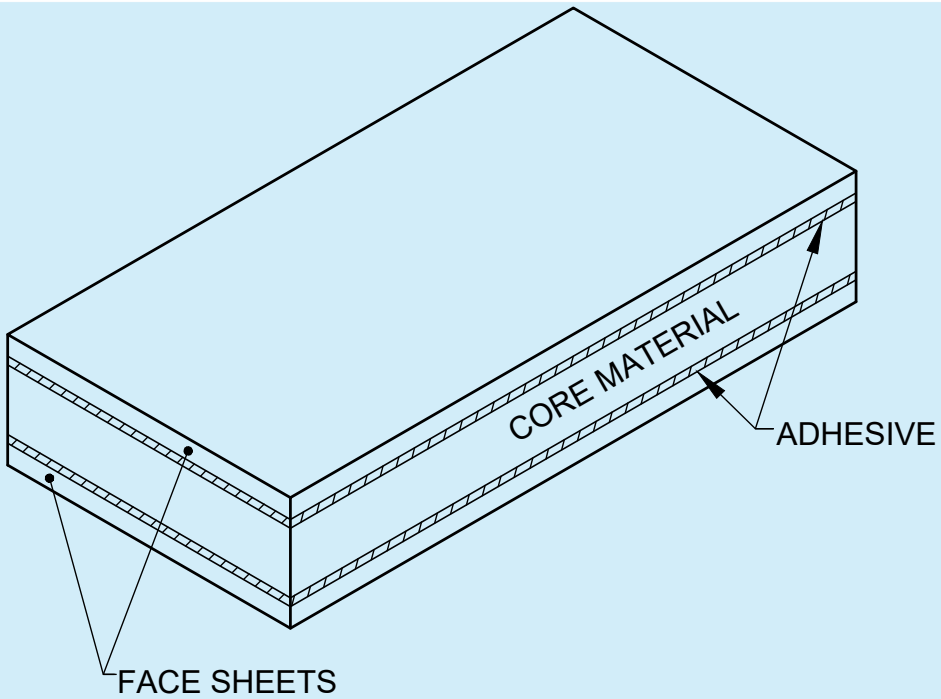

FIG. 18 SANDWICH CONSTRUCTION IN STRUCTURAL APPLICATIONS

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Table 22 Lateral Load Bearing Strength of Wire Nails in Plywood-to-Wood Composite Joints Permissible Lateral Strength of 9 SWG (3.55 mm diameter) Nail in Double Shear (kg) Safe Load per Nail

(*Clause* 15.5)

Sl No.	Thickness of Members mm		Facing Veneer of Plywood Along the Loading Direction (Loading Direction in the Main Member of the Joint)								Across Loading Direction (Loading Direction in the Main Member of the Joint)	
	Main Member Timber	Side Members Plywood	Parallel to Grain		Perpendicular to Grain		At 60° Inclination to Grain		At 30° Inclination to Grain		Loaded Parallel to Grain	
			Permanent Structures	Temporary Structures	Permanent Structures	Temporary Structures	Permanent Structures	Temporary Structures	Permanent Structures	Temporary Structures	Permanent Structures	Temporary Structures
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
i)	Species — *Shorea robusta* (Sal):											
	a) 30	9	109	121	135	156	151	168	140	164	142	151
	b) 40		123	124	121	166	142	181	145	158		
	c) 50		89	123	157	167	162	170	99	139		
	d) 30	12	157	177	150	154	164	179	111	171	135	180
	e) 40		179	191	145	193	154	194	132	192		
	f) 50		151	183	149	191	155	202	128	182		
	g) 30	19	245	283	Larger thickness of plywood is generally not recommended to be used in structures unless heavily stressed						150	209
	h) 40		259	264								
	j) 50		231	316								
ii)	Species — *Pinus roxburghii* (Chir):											
	a) 30	9	102	127	112	141	60	125	92	124	107	148
	b) 40		103	148	100	153	98	142	101	141		
	c) 50		87	133	119	150	50	168	50	114		
	d) 30	12	133	163	116	111	80	132	127	150	118	174
	e) 40		114	173	112	136	64	157	137	174		
	f) 50		99	194	117	138	90	170	119	161		

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Table 22 (*Concluded*)

Sl No.	**Thickness of Members** mm		**Facing Veneer of Plywood Along the Loading Direction** (Loading Direction in the Main Member of the Joint)								**Across Loading Direction** (Loading Direction in the Main Member of the Joint)	
	Main Member Timber	Side Members Plywood	Parallel to Grain		Perpendicular to Grain		At 60° Inclination to Grain		At 30° Inclination to Grain		Loaded Parallel to Grain	
			Permanent Structures	Temporary Structures	Permanent Structures	Temporary Structures	Permanent Structures	Temporary Structures	Permanent Structures	Temporary Structures	Permanent Structures	Temporary Structures
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
	g) 30	19	219	309	Larger thickness of plywood generally not used in structures unless heavily stressed						158	249
	h) 40		238	306								
	j) 50		236	333								

NOTES

1 Plywood shall be of shuttering grade as per the accepted standard [C-3A(16)].

2 Subject to satisfying all the design requirements in structures:

a) For denser species, timber thickness of 30 mm may be used by means of a plywood of 9 mm thickness for all practical purposes. For species like Chir, plywood thickness of 12 mm may be used with 30 mm or 40 mm timber thickness.

b) Regarding aligning the veneers of plywood plate; nail strength data may be used as design function irrespective of alignment of face veneer to overcome practical difficulties.

c) While prebore is essential to drive nail in structural timber as per the good pratice [C-3A(7)], plywood plates need not be prebored for nailing.

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16.5 Tests

Panels of sandwich construction shall be subject to testing in accordance with accepted standards [C-3A(17)]. Tests shall include, as applicable, one or more of the following:

a) Flexural strength/stiffness;
b) Edge-wise compressions;
c) Flat-wise compression;
d) Shear in flat-wise plane;
e) Flat-wise tensions;
f) Flexural creep (creep behaviour of adhesive);
g) Cantilever vibrations (dynamic property); and
h) Weathering for dimensional stability.

17 LAMELLA ROOFING

17.1 General

The lamella roofing offers an excellent architectural edifice in timber, amenable to prefabrication, light weight structure with high central clearance. It is essentially an arched structure formed by a system of intersecting skewed arches built-up of relatively short timber planks of uniform length and cross section. Roof is designed as a two hinged arch with a depth equal to the depth of an individual lamella and width equal to the span of the building. The curved lamellas (planks) are bevelled and bored at the ends and bolted together at an angle, forming a network (grid) pattern of mutually braced and stiffened members (*see* Fig. 19 and Fig. 20).

The design shall be based on the balanced or unbalanced assumed load distribution used for roof arches. Effect of deformation or slip of joints under load on the induced stresses shall be considered in design. Thrust components in both transverse and longitudinal directions of the building due to skewness of the lamella arch shall be adequately resisted. Thrust at lamella joints shall be resisted by the moment of inertia in the continuous lamella and roof sheathing (decking) of lamella roofing. The interaction of arches in two directions adds to the strength and stability against horizontal forces. For design calculations several assumption tested and observed derivations, long-duration loading factors, seasoning advantages and effects of defects should be taken into account.

17.2 Lamellas

Planking shall be of a grade of timber that is adequate in strength and stiffness to sustain the assumed loads, forces, thrust and bending moments generated in lamella roofing. Lamella planks shall be seasoned to a moisture content approximating that they will attain in service. Lamella joints shall be proportioned so that allowable stresses at bearings of the non-continuous lamellas on the continuous lamellas or bearings under the head or washer of bolts are not exceeded.

17.2.1 *Design Sequence and Methods*

A lamella roof is essentially an arch (segmental, parabolic or elliptical) formed by a system of intersecting skewed arches, built up of relatively short members of small dimensional planks called ‘lamellas’. The arch is designed as a two hinged arch with a depth equal to the depth of the individual lamella and a width equal to the span of the building of certain length. For design purposes, a unit length of the given span forming a single arch is considered loaded with the given loads. For the particular arch, curvature offers resultant outward suction on the roof to be less than its self weight to facilitate design and construction for economy (this is possible only when the semi-angle of arch is less than 42°).

Some design assumptions that should be considered include:

a) Self weight of timber components (in particular species), namely lamella planks, roof boarding, wall plates, etc is assumed for the particular timber section;

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b) Imposed loads as per Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS for curved roofs where slope at springing point is greater than 10°. Take the reduced worked out values of imposed loads for actual design for timber components as per this Subsection;

c) Wind load is taken as per Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS for structures with large openings, that is, openings larger than 20 percent of wall area (or as the case may be);

d) Stresses developed in curved lamella planks should not exceed the allowable safe stresses of timber species used;

e) Range of various sizes of timber planks for lamellas shall be as under:

1) *Length* — 1.0 m to 2.5 m;

2) *Section*:

i) *Thickness* — 25 mm to 50 mm; and

ii) *Width of uncurved plank* — 100 mm to 250 mm.

f) Limitations to be observed for spacing of lamellas over wall plate (or bressummer):

1) i) For skew angle of 21° (diamond angle 42°);
ii) For 2.5 m length of plank, spacing shall be 870 mm maximum; and
iii) For 1.0 m length, spacing shall be 360 mm maximum.

2) i) For skew angle of 19° (diamond angle 38°);
ii) For 2.5m length of plank, spacing shall be 800 mm maximum; and
iii) For 1.0 m length of plank, spacing shall be 320 mm maximum.

g) Value of horizontal thrust, resultant thrust and maximum bending moment, are taken on per metre length of structure in case of transverse direction of arch;

h) Bending moment may be suitably reduced, if so desired by the designer, by applying stability factor if the ratio of the length of structure to the curved length of arch so warrants (as this ratio increases, the stability of the structure decreases);

j) The thrust in the skewed direction of arch, within one spacing of lamellas is shared by two lamella-lines in skewed direction; and

k) More than one bolt (MS) shall be used for nodal joint. For design check of bolt against tension, half the value of thrust in skewed direction will be considered for calculations purposes in light of (j) {in case of stiff roof covering material [that is, corrugated galvanized iron (CGI) sheet, close boarding, etc] all lamella planks will be held jointly behaving as a monolithic unit to a large extent and the actual stresses developed in bolt will be reduced resulting in lesser actual tension in bolts than the calculated design value}.

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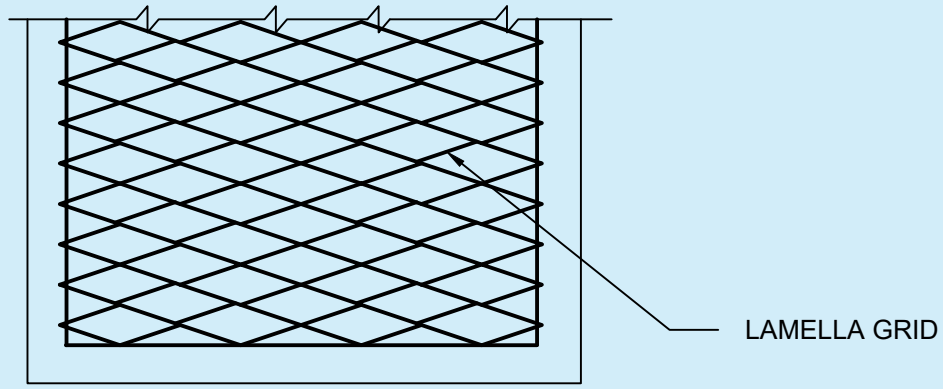
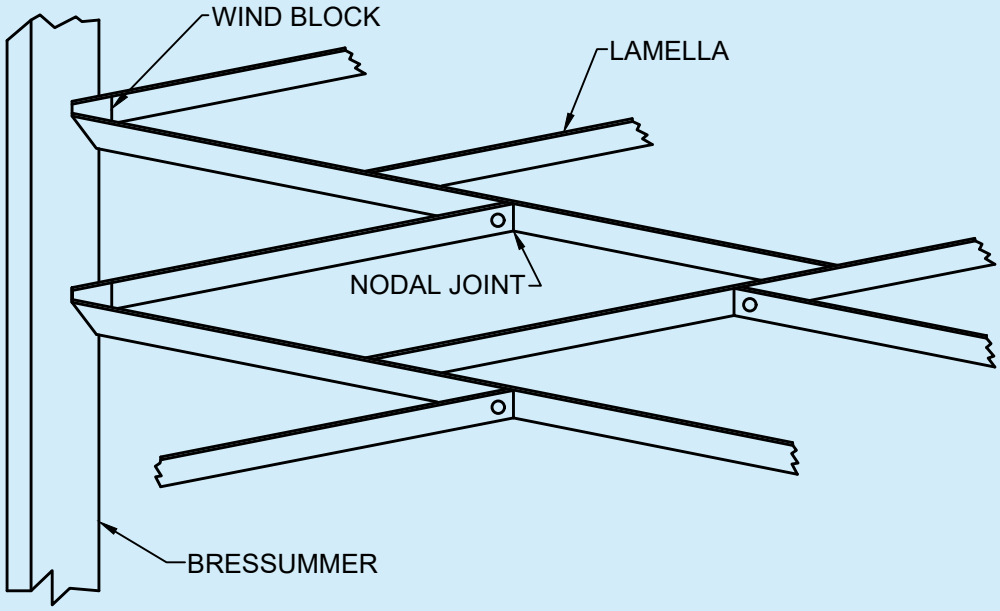
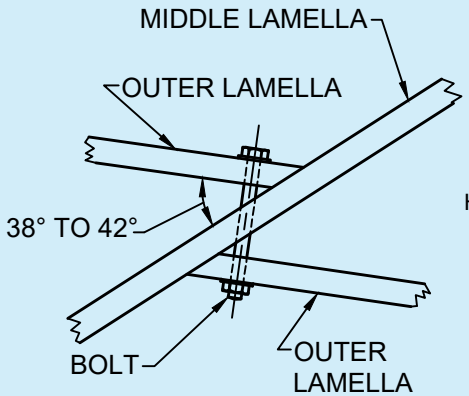
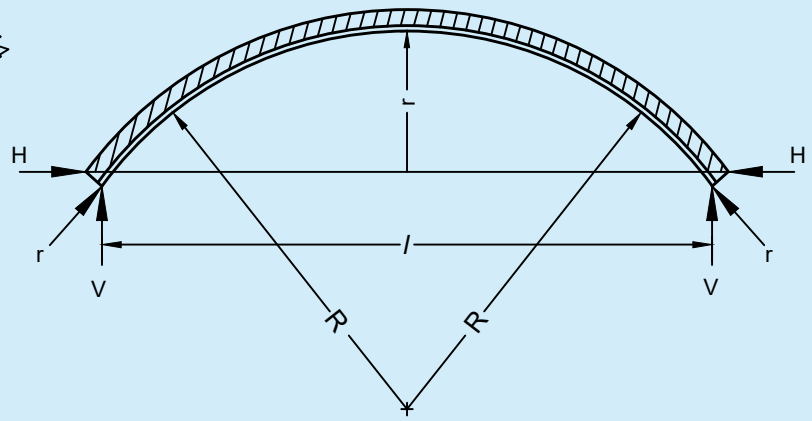




LAMELLA GRID



LAMELLA NODAL JOINT



LOAD REACTION DIAGRAM

FIG. 19 TYPICAL ARRANGEMENT OF LAMELLA ROOFING

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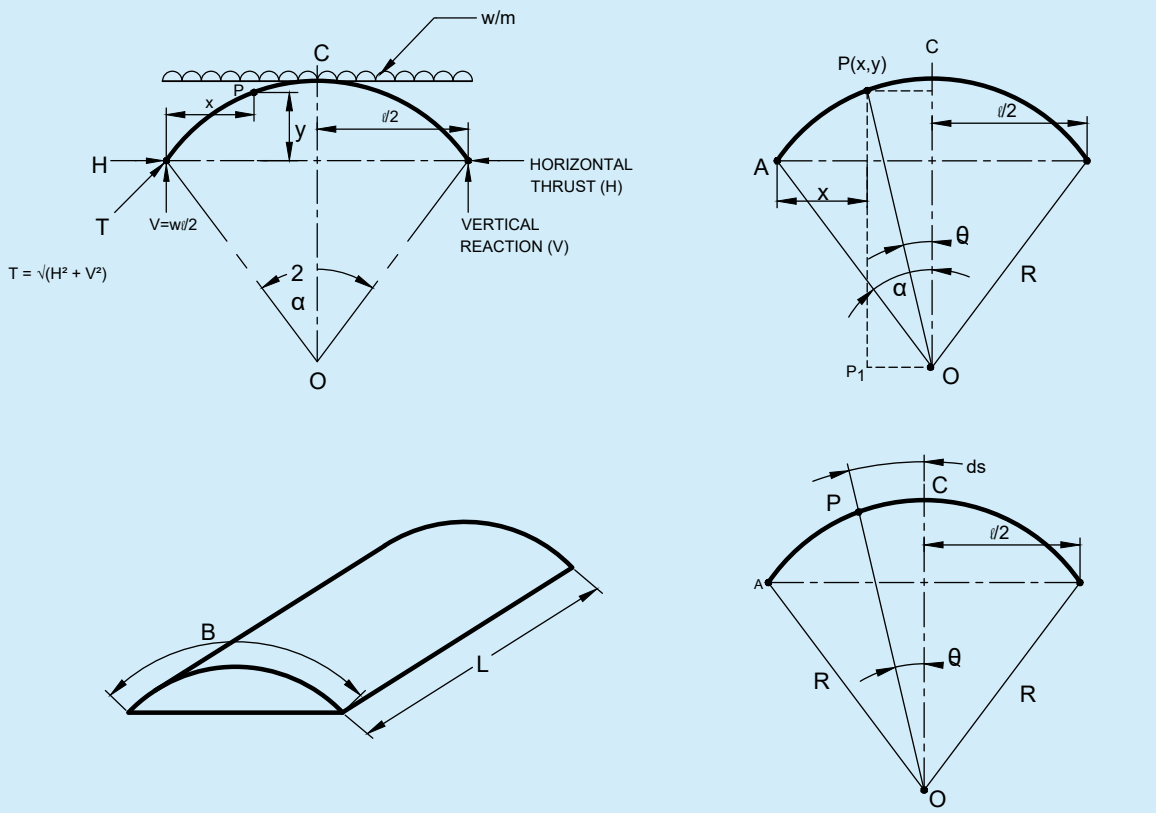

NOTES

1 When the ratio of length of the structure (*L*) to the circumference of the arch (*B*) increases, stability of the structure decreases. The roof should be designed as:

a) short roof (say, *L*/*B* = 1), or

b) long roof (say, *L*/*B* = 1.5).

Shorter roofs have an advantage over long roofs for economical design. However, there is no such advantage beyond *L*/*B* of 4.

2 Due to stiff roof covering material, stiffness is induced in an arch, which results in safer design.

FIG. 20 STABILITY OF LAMELLA ARCH ROOF

17.2.1.1 *Horizontal thrust*

Any of the known methods for solving a statically indeterminate structure is used to evaluate horizontal thrust, bending moment, etc, to which a segmental arch will be subjected under the given condition of loadings. For 60° segmental arch, the radius of an arch will be equal to the span of the arch. Horizontal thrust can be calculated by the following two methods:

a) *Method* 1

Horizontal thrust (H_B) per length of one bay (that is, distance between 'tie-rod spacing' or 'column centres'):

$$H_B = (DL + IL) \times \left[\frac{l^2 + {s_t}^2}{8r}\right]$$

where

H_B = horizontal thrust, in N;

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DL = dead load, in N;

IL = imposed load, in N;

l = span of arch, in m;

s_t = tie-rod spacing, in m; and

r = rise of roof arch, in m.

b) *Method* 2

Horizontal thrust (H_1) per metre length of structure (in N):

$$H_1 = C \times w \times R$$

where

C = coefficient varying with the angle of arch and radius of arch;

w = equivalent distributed load per metre of span (in N/m); and

R = radius of circular arch (in m).

Horizontal thrust (H_1) is evaluated as:

$$H_1 = \frac{\int \mu y ds}{\int y^2 ds}$$

where

H_1 = horizontal thrust (per metre length of structure), in N;

μ = bending moment at any point '*P*' distance 'x' metre from one end of springing point of arch and for vertical loads only (consider arch as a beam simply supported carrying the same load);

y = height of point '*P*' above springing line of arch;

θ = angle which the point '*P*' makes at the centre of arch with the vertical; and

ds = $Rd\theta$.

Now solution for numerator:

$$\mu = \frac{wl}{2}x - wx\frac{x}{2}$$

$$= \frac{wx}{2}(l - x)$$

where

$$x = \frac{l}{2} - R\sin\theta$$

$$x = \frac{R}{2} - R\sin\theta$$

$$x = R\left(\frac{1}{2} - \sin\theta\right)$$

and $y = R\cos\theta - R\cos\alpha$

$$y = R(\cos\theta - \cos\alpha)$$

and $ds = Rd\theta$

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Substitute the values in expression:

$$\int \mu y ds = \int \frac{wx(l-x)}{2} R(cos\,\theta - cos\,\alpha) R d\theta$$

Solve for limits of θ from $-\alpha$ to $+\alpha$ = $\int_0^{\alpha} wR^4(\frac{1}{4} - sin^2\,\theta)(cos\,\theta - cos\,\alpha)d\theta$

Again for denominator;

$$\int y^2\, ds = \int_{-\alpha}^{+\alpha} R^2(\cos\theta - \cos\alpha)^2 R d\theta$$

$$= 2R^3 \int_0^{\alpha} (\cos\theta - \cos\alpha)^2\, d\theta$$

Therefore, $H_1 = \frac{\int \mu y ds}{\int y^2 ds}$

$$= \frac{wR^4 \int_0^{\alpha} \left[\frac{1}{4} - \sin^2\theta\right](\cos\theta - \cos\alpha)\, d\theta}{2R^3 \int_0^{\alpha} (\cos\theta - \cos\alpha)^2\, d\theta}$$

On simplification of the above equation for a particular arch, α = 30°, R = 1 (α in radian = 0.5 approx),

sin α = 1/2
cos α = √3/2

H_1 = 0.923 wR

Vertical reaction, $V_1 = \frac{Total\ load \times span}{2}$ at each end of arch due to dead and live loads per metre length (curved strip) of arch roofing.

17.2.1.2 *Resultant thrust*, T_1

Due to horizontal thrust and the vertical reaction, resultant transverse thrust, in kg per metre length on the arch can be worked out:

$$T_1 = \sqrt{{H_1}^2 - {V_1}^2}$$

17.2.1.3 *Maximum bending moment*

Bending moment at any point 'P' on arch making an angle of θ with the vertical line OC is, $BM = \mu - (H \times y)$

$$= \frac{wR^2}{2}\left(\frac{1}{4} - \sin^2\theta\right) - 0.923wR^2\left(\cos\theta - \frac{\sqrt{3}}{2}\right)$$

Differentiating the above expression for BM, we get

$$\frac{dM}{d\theta} = (-\sin\theta\cos\theta + 0.923\sin\theta)wR^2$$

It is well known that $\frac{dM}{d\theta} = \text{shear force}$

This shear is zero when BM is maximum, that is:

either θ = zero

or cos θ = 0.923

Therefore, $\theta = \cos^{-1}0.923 = 22.6°$

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Substituting these values in the above equation of bending moment, we get maximum BM, that is, the maximum BM occurs at:

a) crown of arch; and

b) at 22.6° from the central line OC.

Thus when $\theta = 0$

$$BM = \frac{wR^2}{2} \times \frac{1}{4} - 0.923wR^2\left(1 - \frac{\sqrt{3}}{2}\right)$$

$$= wR^2(0.125 - 0.12368)$$

$$= 0.001\ 32wR^2$$

When $\theta = 22.6°$, $\sin\theta = 0.384$

$$BM = \frac{wR^2}{2}\left(\frac{1}{4} - 0.384\right)^2 - 0.923wR^2(0.923 - 0.866)$$

$$= wR^2(0.125 - 0.073\ 728) - 0.923 \times 0.057wR^2$$

$$= -0.001\ 339wR^2 \text{ or say } -0.001\ 34wR^2$$

The arch has to be designed for the greater of the two values (subject to the longitudinal stiffness coefficient on the basis of ratio *L*/*B* of length of the structure to the circumference of the arch) for economical design.

NOTE — Value of horizontal thrust, H_1, resultant thrust, T_1 and bending moment, M_1 shall be for 1 m length of building.

17.2.1.4 *Spacing of lamellas*

Lengthwise spacing of lamellas depends upon the angle of skew lamellas and the length of the planks used for lamellas (As a general guidance it may be stated that the angle enclosed between the intersecting skewed arches should not exceed 45°. An angle between 38° and 40° is always preferred).

17.2.1.5 *Design of lamella sizes*

Net section of each of lamellas is assumed in curved profile conforming to the curvature of the roof surface.

Maximum BM = M_s over 600 mm spacing of lamella in the direction of skew angle, for example:

$$= \frac{0.6M_1 \times 10}{\cos 19°} \text{ Nm}$$

where

M_1 = maximum bending moment per metre length of structure in transverse direction.

And maximum thrust *N* (in Newton) on this strip of 600 mm in skewed direction is:

$$N = \frac{0.6T_1}{\cos 19°} \times \frac{1}{2}$$

(as the force T_1 is shared by two lines of lamellas)

where

N = maximum thrust on 600 mm strip in skewed direction; and

T_1 = resultant thrust in transverse direction (in N) per metre length of structure.

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Now total compressive and flexural stresses should not exceed the allowable safe stress of timber species used for lamella plank. For fulfilling this condition the following equation should be checked:

$$\frac{N/A}{f_{c\,\mathrm{II}}} + \frac{M_s/Z}{f_t} \leq 1$$

where

N = maximum thrust in skewed direction on strip of 600 mm (in N) corresponding to the maximum bending moment;

A = cross-sectional area of lamella plank, in mm^2;

Z = section modulus of lamella plank, in mm^3;

$f_{c\,\mathrm{II}}$ = safe compression strength of timber fibre parallel to grain, N/mm^2;

M_s = bending moment over 600 mm strip of arch in skewed direction, Nmm;

f_t = safe tensile strength of timber parallel to grain, N/mm^2; and

NOTE — In case the lamella plank is under stressed, the minimum section for practical reasons shall be 25 mm × 100 mm subject to satisfying minimum edge and intermediate distances for accommodating bolts at nodal joints.

17.2.1.6 *Design of bolts for nodal joint*

To avoid any possibility of hinge action with the use of bolts, it is imperative to adopt two or more bolts of smaller diameter and adequately spaced in accordance with the good practice [6-3A(9)].

Assume diameter of bolt '*d*' in mm, and length of bolt in one plank '*t*' mm, that is, thickness of middle lamella plank.

Area of contact of bolt with timber plank = $d \times t$ mm^2

a) *Check against crushing of timber at contact point of bolt* — For the bolt to transmit load without crushing the timber fibres, the following condition shall be satisfied:

$t \times d \times$ compressive stress perpendicular to grain for the timber species $> P$

where

P = total load to be transmitted through bolt, in N.

b) *Check against tension in bolt* — Arrangement of bolts at the junction of lamellas result in pure tension in bolt as bolt line is at right angle to the outer two lamellas:

Therefore, $Z_t = N \times \cot 2\phi$

where

Z_t = tension in bolt, in N;

N = normal thrust corresponding to maximum moment M_s (in N); and

$= \frac{T_1}{\cos} \times \frac{1}{2}$

T_1 = resultant thrust in transverse direction (in N) per metre length of structure.

Therefore;

Area of bolt (A_t) × tensile stress of steel (F_t) ≥ Z_t

Area of bolt required for safe design = Z_t/F_t

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Select the diameter to engage at least two bolts.

(Required area of bolt is the effective area after considering reduction of area at the threaded end, which is generally 17 percent of the actual bolt diameter).

17.2.1.7 *Design of steel washer at joints*

Area of steel plate = A_t mm^2

Stress perpendicular to grain of wood at the point of contact with steel plate washer = Z_t/A_t N/mm^2

If $Z_t/A_t \leq$ compressive stress perpendicular to grain of the species of timber used, the assumed section is safe.

17.3 Construction

Design and construction of lamella roofs in India assumes the roof surfaces to be cylindrical with every individual lamella an elliptic segment of an elliptical arch of constant curved length but of different curvature. Lamella construction is thus more of an art than science as there is no analytical method available for true generation of schedule of cutting lengths and curvature of curved members forming the lamella grid. Dependence of an engineer on the practical ingenuity of master carpenter is almost final. All the lamella joints shall be accurately cut and fitted to give full bearing without excessive deformation or slip. Bolts at lamella splices shall be adequate to hold the members in their proper position and shall not be over tightened to cause bending of the lamellas or mashing of wood under the bolt heads. Connection of lamellas to the end arches shall be adequate to transmit the thrust or any other force. Sufficient false work or sliding jig shall be provided for the support of lamella roof during actual construction/erection. To avoid excessive deformation in a lamella system, all joints should be carefully fitted and well tightened.

18 NAIL AND SCREW HOLDING POWER OF TIMBER

18.1 General

One of the most common ways of joining timber pieces to one another is by means of common wire nails and wood screws. Timber is used for structural and non-structural purposes in form of scantlings, rafters, joists, boarding, crating and packing cases, etc, needing suitable methods of joining them. Nevertheless, it is the timber which holds the nails or screws and as such pulling of the nails/screws is the chief factor which come into play predominantly. In structural nailed joints, nails are essentially loaded laterally, the design data for which is already available in good practice [C-3A(7)]. Data on holding power of nails/screws in different species is also useful for common commercial purposes. The resistance of mechanical fastenings is a function of the specific gravity of wood, direction of penetration with respect to the grain direction, depth of penetration and the diameter of fastener assuming that the spacing of fasteners should be adequate to preclude splitting of wood.

18.2 Nails

Nails are probably the most common and familiar fastener. They are of many types and sizes in accordance with the accepted standard [C-3A(18)]. In general nails give stronger joints when driven into the side grain of wood than into the end grain. Nails perform best when loaded laterally as compared to axial withdrawal so the nailed joints should be designed for lateral nail bearing in structural design. Information on withdrawal resistance of nails is available and joints may be designed for that kind of loading as and when necessary.

18.3 Screw

Next to the hammer driven nails, the wood screw may be the most commonly used fastener. Wood screws are seldom used in structural work because of their primary advantage is in withdrawal resistance for example for fixing of ceiling boards to joists, purlin cleats, besides the door hinges, etc. They are of considerable structural importance in fixture design and manufacture. Wood screws are generally finished in a variety of head shapes and manufactured in various lengths for different screw diameters or gauges in accordance with the accepted standards [C-3A(19)].

The withdrawal resistance of wood screws is a function of screw diameter, length of engagement of the threaded portion into the member, and the specific gravity of the species of wood. Withdrawal load capacity of wood

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screws are available for some species and joints may be designed accordingly. End grain load on wood screws are unreliable and wood screws shall not be used for that purpose.

18.4 Nail and screw holding power is one of the very important properties of timber which governs its utilization in diverse industrial and engineering fields. This has been evaluated by experimental method followed for evaluation of such properties as per normal procedure given in the accepted standards [C-3A(20)], using:

a) *Specimen* — of size 150 mm × 50 mm × 50 mm;

b) *Nail* — diamond pointed wire nail of 50 mm length and 2.50 mm shank diameter;

c) *Screw* — galvanized gimlet pointed, No. 8; and

d) *Conditions* — penetration 25 mm.

While nails were driven by a hand hammer, screws were driven by a screw driver in a 2.5 mm diameter prebore on radial, tangential and end faces.

Composite nail and screw holding power of some Indian timbers, evaluated as above, is presented in Table 23 for guidance in design.

Table 23 Composite Nail and Screw Holding Power of Some Indian Timbers

(*Clause* 18.4)

Sl No.	Timber Species	Locality	Standard Specific Gravity (Weight Oven Dry)/Volume Green	Composite Holding Power N	
				Nail	Screw
(1)	(2)	(3)	(4)	(5)	(6)
i)	*Acacia catechu* (Khair)	Uttar Pradesh	0.888	2 119	5 454
ii)	*Acacia ferruginea*	Maharashtra	0.876	1 893	5 700
iii)	*Acacia leucophloea*	Madhya Pradesh	0.660	1 776	4 503
iv)	*A. nilotica* (Babul)	Uttar Pradesh	0.670	1 815	5 219
v)	*Acrocarpus fraxinifolius* (Mundani)	Uttar Pradesh	0.662	1 020	2 865
vi)	*Adina oligocephala*	Arunachal Pradesh	0.612	1 795	3 453
vii)	*Aesculus indica* (Horse chestnut)	Himachal Pradesh	0.428	755	2 119
viii)	*Albizia lucida*	Arunachal Pradesh	0.501	1 462	3 188
ix)	*Anogeissus sericea*	Madhya Pradesh	0.738	1 570	4 944
x)	*Aphanamixis polgstachya* (Pitraj)	West Bengal	0.557	1 648	3 110
xi)	*Araucaria cunning hanvil*	Uttar Pradesh	0.486	1 128	2 943
xii)	*Aritocarpus lakoocha* (Lakooch)	Uttar Pradesh	0.530	1 050	2 982
xiii)	*Azadirachta indica* (Neem)	Uttar Pradesh	0.693	2 001	5 219
xiv)	*Betula alnoides* (Birch)	West Bengal	0.569	1 648	3 237
xv)	*Broussonetia papyrifera*	Uttar Pradesh	0.394	746	1 383
xvi)	*Canarium strictum* (White dhup)	Assam	0.486	1 207	2 619
xvii)	*Cassia fistula* (Amaltos)	Uttar Pradesh	0.746	2 109	4 748
xviii)	*Cassia siamea*	Madhya Pradesh	0.697	1 315	3 649

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Table 23 (*Continued*)

Sl No.	Timber Species	Locality	Standard Specific Gravity (Weight Oven Dry)/Volume Green	Composite Holding Power N	
				Nail	Screw
(1)	(2)	(3)	(4)	(5)	(6)
xix)	*Castanopsis indica* (Indian chestnut)	Meghalaya	0.591	1 697	3 090
xx)	*Cupressus cashmeriana*	Uttar Pradesh	0.409	903	2 325
xxi)	*Dipterocarpus bourdilloni* (Gurjan)	Kerala	0.577	912	2 766
xxii)	*Dipterocarpus indicus* (Gurjan)	Karnataka	0.618	1 109	2 904
xxiii)	*Eucalyptus hybrid*	Uttar Pradesh	0.632	1 736	2 453
xxiv)	*Exbucklandia –populnea* (Pipli)	West Bengal	0.558	1 403	3 443
xxv)	*Fraxinus udheri*	Uttar Pradesh	0.539	1 354	3 483
xxvi)	*Gluta travancorica* (Gluta)	Tamil Nadu	0.621	1 324	2 923
xxvii)	*Grewia tiliifolia* (Dhaman)	Karnataka	0.679	1 746	2 884
xxviii)	*Holarhena antidysenterica* (Kurchi)	Uttar Pradesh	0.445	1 020	2 698
xxix)	*Lagerstroemia specicosa* (Jarul)	Kerala	0.531	1 697	2 717
xxx)	*Lannea coromendelica* (Jhingan)	Rajasthan	0.497	1 216	2 894
xxxi)	*Magnolia hookeri*	Arunachal Pradesh	0.403	1 128	1 746
xxxii)	*Mangifera indica* (Mango)	Uttar Pradesh	0.411	1 383	2 433
xxxiii)	*Manglietia spp.*	Assam	0.409	912	2 031
xxxiv)	*Mesna assamica* (Sianhor)	Assam	0.687	1 795	3 502
xxxv)	*Michelia champaca* (Champ)	Assam	0.488	1 128	2 737
xxxvi)	*Mimusops elengi*	Karnataka	0.762	2 158	5 101
xxxvii)	*Morus alba* (Mulberry)	Uttar Pradesh	0.557	1 030	4 091
xxxviii)	*Morus laevigata* (Bola)	Assam	0.567	932	2 600
xxxix)	*Morus spp.*	Uttar Pradesh	0.594	1 589	3 129
xl)	*Nephelium litchi*	Uttar Pradesh	0.762	1 825	4 238
xli)	*Oroxylum indicum*	Andaman	0.370	961	1 736
xlii)	*Ougenia oogeinensis* (Sandan)	Uttar Pradesh	0.701	1 570	5 317
xliii)	*Pinus kesiya*	Nagaland	0.489	1 089	2 354
xliv)	*Pinus kesiya*	Orissa	0.453	1 118	2 325
xlv)	*Pinus roxburghii* (Chir)	Uttar Pradesh	0.453	1 128	2 482
xlvi)	*Pinus wallichiana* (Kail)	Himachal Pradesh	0.468	873	1 521
xlvii)	*Pistacia integerrima*	Jammu and Kashmir	0.733	1 324	4 120
xlviii)	*Polyalthia spp.* (Debdara)	Madhya Pradesh	0.586	1 452	2 953
xlix)	*Pometa pinnata*	Andaman	0.636	1 579	3 532

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Table 23 (*Concluded*)

Sl No.	Timber Species	Locality	Standard Specific Gravity (Weight Oven Dry)/Volume Green	Composite Holding Power N	
				Nail	Screw
(1)	(2)	(3)	(4)	(5)	(6)
l)	*Prunus nepalensis* (Arupati)	West Bengal	0.482	1 099	2 531
li)	*Pterocarpus marsupium*	Bihar	0.587	1 305	2 982
lii)	*Pterygota alata* (Narikel)	Assam	0.475	1 403	3 463
liii)	*Pterygota alata* (Narikel)	Kerala		1 089	2 639
liv)	*Robinia spp.*	Jammu and Kashmir	0.629	2 227	3 777
lv)	*Sageraea elliptica* (Chooi)	Andaman	0.682	1 805	4 522
lvi)	*Schleichera oleosa* (Kusum)	Bihar	0.841	1 884	4 277
lvii)	*Shorea robusta* (Sal)	Uttar Pradesh	0.774	1 216	3 620
lviii)	*Soymida ferifuga*	Tamil Nadu	0.965	1 687	6 033
lix)	*Tectona grandis* (Teak)	Uttar Pradesh	0.574	1 020	3 404
lx)	*Tectona grandis* (Teak)	Tamil Nadu	0.633	1 364	3 483
lxi)	*Tectona grandis* (Teak)	Maharashtra	0.526	971	3 728
lxii)	*Terminalia alata* (Laurel)	Uttar Pradesh	0.806	2 521	3 806
lxiii)	*Terminalia citrina*	Assam	0.629	1 540	3 855
lxiv)	*Terminalia paniculata* (Kindal)	Kerala	0.630	1 982	3 257
lxv)	*Tetrameles nudiflora* (Maina)	Assam	0.289	667	1 413
lxvi)	*Thespesia populnea* (Bhendi)	Maharashtra	0.647	1 923	5 425
lxvii)	*Tsuga brunonsiana*	West Bengal	0.380	540	2 050
lxviii)	*Ulmus coallichiana* (Elm)	Himachal Pradesh	0.455	834	1 766
lxix)	*Zanthoxylum spp.*	West Bengal	0.496	1 138	3 071

19 PROTECTION AGAINST TERMITE ATTACK IN BUILDINGS

19.1 Two groups of organisms which affect the mechanical and aesthetic properties of wood in houses are fungi and insects. The most important wood destroying insects belong to termites and beetles. Of about 250 species of wood destroying termites recorded in India, not more than a dozen species attack building causing about 90 percent of the damage to timber and other cellulosic materials. Subterranean termites are the most destructive of the insects that infest wood in houses justifying prevention measures to be incorporated in the design and construction of buildings.

19.1.1 Control measures consist in isolating or sealing off the building from termites by chemical and non-chemical construction techniques. It is recognized that 95 percent damage is due to internal travel of the termites from ground upwards rather than external entry through entrance thus calling upon for appropriate control measures in accordance with good practices [C-3A(21)].

19.2 Chemical Methods

Termites live in soil in large colonies and damage the wooden structure in the buildings by eating up the wood or building nests in the wood. Poisoning the soil under and around the building is a normal recommended practice. Spraying of chemical solution in the trenches of foundations in and around walls, areas under floors before and

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after filling of earth, etc. In already constructed building the treatment can be given by digging trenches all around the building and then giving a liberal dose of chemicals and then closing the trenches.

19.3 Wood Preservatives

Natural resistance against organisms of quite a few wood species provides durability of timber without special protection measure. It is a property of heartwood while sapwood is normally always susceptible to attack by organisms. Preservatives should be well applied with sufficient penetration into timber. For engineers, architects and builders, the following are prime considerations for choice of preservatives:

a) Inflammability of treated timber is not increased and mechanical properties are not decreased;

b) Compatibility with the glue in laminated wood, plywood and board material;

c) Water repellent effect is preferred;

d) Possible suitability for priming coat;

e) Possibility of painting and other finishes;

f) Non-corrosive nature in case of metal fasteners; and

g) Influence on plastics, rubber, tiles and concrete.

19.4 Constructional Method

Protection against potential problem of termite attack can simply be carried out by ordinary good construction which prevents a colony from gaining access by:

a) periodic visual observations on termite galleries to be broken off;

b) specially formed and properly installed metal shield at plinth level; and

c) continuous floor slabs, apron floors and termite grooves on periphery of buildings.

20 CROSS-LAMINATED TIMBER (CLT)

Cross-laminated timber (CLT) is an engineered wood product used as a structural component in mass timber construction. CLT panels shall consist of not less than three layers of solid-sawn lumber or structural composite lumber, with adjacent layers cross-oriented and bonded with structural adhesives to form a solid wood element.

20.1 General Requirements

20.1.1 *Material Specifications*

CLT panels shall be manufactured to meet stringent performance criteria for mechanical properties, including load-bearing capacity, dimensional tolerances, and adhesive bond durability to ensure structural reliability. Panels should also demonstrate resistance to delamination and other essential physical properties through rigorous testing to validate long-term performance under various environmental conditions.

The timber used for CLT production shall be kiln-dried to a moisture content of 12 percent ± 2 percent, ensuring stability and resistance to deformation.

20.1.2 *Dimensions*

CLT panels shall have a minimum thickness of 50 mm, with thicknesses ranging up to 500 mm for structural applications. The typical panel dimensions for commercial use may be up to 18 000 mm in length and 3 000 mm in width.

The number of layers shall be odd (for example, 3, 5, 7), with layer thicknesses adjusted according to structural requirements.

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20.2 Physical Properties

20.2.1 *Delamination Resistance*

Delamination testing shall be conducted to ensure that total delamination ($Delam_{tot}$) shall not exceed 10 percent, and maximum delamination ($Delam_{max}$) shall not exceed 40 percent, ensuring durability and structural integrity.

Results from studies indicate that CLT made from plantation-grown hardwoods such as Rubberwood, *Melia dubia* (*Malabar Neem*), and Eucalyptus meets these requirements, with delamination values comparable to Radiata pine, a commonly used species in CLT production.

20.2.2 *Water Absorption and Swelling Properties*

CLT panels shall be evaluated for water absorption (WA), thickness swelling (TS), and volumetric swelling (VS) using the water soaking method. These parameters shall be assessed to ensure that the panels meet acceptable limits for moisture-induced dimensional changes, maintaining stability under varying environmental conditions.

Reference to results from studies (Table 24 and Table 25) shows that hardwood species demonstrate comparable performance to Radiata pine, indicating their suitability for structural applications.

For detailed information, a reference may be made to *A Hybrid Approach to Green Building Construction Using Mass Timber February* 2024, DOI: 10.58532/V3BICE3P8CH2 by Ram Kumar V R and Amitava Sil.

Table 24 Physical Properties of Polyurethane (PUR) Bonded Homogenous CLT in Comparison to Radiata Pine CLT

(*Clause* 20.2.2)

Sl No.	Type of CLT	$Delam_{tot}$ Percent	$Delam_{max}$ Percent	WA Percent		TS Percent		VS Percent	
		CLT	CLT	CLT	Solid wood	CLT	Solid wood	CLT	Solid Wood
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
i)	Rubberwood	6.49 (0.98)	29.01 (2.90)	31.23 (4.19)	52.91 (5.47)	3.26 (0.93)	9.93 (1.82)	3.26 (0.93)	9.93 (1.82)
ii)	*Melia dubia*	6.77 (0.78)	26.65 (3.86)	23.51 (3.48)	39.98 (9.94)	4.38 (2.28)	9.15 (3.29)	7.86 (1.89)	13.58 (6.85)
iii)	Silver oak	8.08 (1.21)	21.3 (4.13)	14.10 (3.66)	35.88 (12.28)	5.05 (1.02)	13.19 (1.89)	8.37 (1.43)	20.83 (2.62)
iv)	Eucalyptus	6.61 (2.29)	34.10 (3.28)	25.68 (7.57)	38.97 (7.57)	4.06 (1.24)	15.08 (7.76)	7.39 (1.70)	17.92 (7.74)
v)	Radiata pine	6.63 (0.86)	26.70 (3.63)	29.35 (5.46)	40.03 (7.20)	4.29 (0.29)	12.47 (3.01)	14.09 (0.66)	20.62 (2.82)

NOTE — Values in the parentheses indicate standard deviation.

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Table 25 Physical Properties of Hybrid CLT in Comparison to Radiata Pine CLT

(*Clause* 20.2.2)

Sl No.	Type of CLT	$Delam_{tot}$ Percent	$Delam_{max}$ Percent	WA Percent	TS Percent	VS Percent
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	Hybrid with *Eucalyptus* (E) and *Melia dubia*(M) (EME)	7.68 (0.57)	31.62 (4.37)	26.43 (3.66)	6.33 (1.77)	15.86 (1.47)
ii)	Hybrid with *Melia dubia* (M) and *Eucalyptus* (E) (MEM)	6.68 (0.53)	28.65 (3.82)	24.35 (6.86)	5.59 (1.38)	8.85 (1.93)
iii)	Hybrid with *Melia dubia* (M) and Silver oak (S) (MSM)	7.06 (1.22)	28.96 (1.12)	21.86 (5.09)	5.43 (1.84)	7.04 (3.94)
iv)	Hybrid with silver oak (S) and *Melia dubia* (M) (SMS)	7.65 (0.23)	26.62 (1.45)	15.51 (5.6)	8.43 (2.30)	8.24 (2.96)
v)	Radiata pine	6.63 (0.86)	26.70 (3.63)	29.35 (5.46)	4.29 (0.29)	14.09 (0.66)

NOTE — Values in the parentheses indicate standard deviation.

20.2.3 *Density*

The density of CLT panels shall be determined according to the timber species and adhesive used. The density of CLT panels shall range from 5 500 kg/m³ to 8 500 kg/m³, depending on the species (for example, Rubberwood, Eucalyptus). Table 26 provides the density values for different species used in CLT production.

Table 26 Density of CLT Panels

(*Clause* 20.2.3)

Sl No.	Species	Density of Wood (at 12 Percent Moisture Content) kg/m³	Density of CLT kg/m³
(1)	(2)	(3)	(4)
i)	Rubberwood	6 200 (0.05)	6 500 (0.08)
ii)	*Melia dubia*	5 100 (0.04)	5 500 (0.01)
iii)	Eucalyptus	8 000 (0.05)	8 500 (0.03)
iv)	Silver oak	6 200 (0.02)	6 800 (0.02)
v)	Radiata pine	4 200 (0.03)	4 500 (0.03)
vi)	EME	–	7 700 (0.02)
vii)	MEM	–	6 400 (0.03)
viii)	MSM	–	5 500 (0.02)
ix)	SMS	–	6 700 (0.01)

NOTE — Values in the parentheses indicate standard deviation.

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20.3 Mechanical Properties

20.3.1 *Bending, Compression, and Shear Strength*

Mechanical testing shall include four-point bending tests, compression parallel to grain tests and block shear tests. The results shall comply with the minimum strength requirements for structural applications as specified in Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS.

Table 27 and Table 28 present the mechanical properties of homogeneous and hybrid CLT panels, showing adequate performance for structural uses. Rubberwood CLT exhibited higher mechanical properties than hybrid configurations in some tests.

20.3.2 *Impact of Adhesive Type on Mechanical Performance*

The choice of adhesive (for example, phenol resorcinol formaldehyde, PUR) shall be considered, as it affects the bonding strength and durability. Mechanical properties of CLT bonded with different adhesives, indicating that PRF-bonded panels have superior performance.

Table 27 Mechanical Properties of Homogeneous CLT Panels

(*Clause* 20.3.1)

Sl No.	CLT Made from	MoE	MoR	Compression Parallel to Grain	Block Shear Strength
		MPa	MPa	MPa	MPa
(1)	(2)	(3)	(4)	(5)	(6)
i)	Rubberwood	8 600 (328.3)	48.7 (13.8)	25.0 (2.3)	6.7 (1.4)
ii)	*Melia dubia*	6 758 (598.6)	44.8 (5.7)	23.0 (1.8)	6.3 (0.7)
iii)	Silver oak	8 835 (739.5)	64.2 (6.9)	26.34 (2.0)	6.7 (0.5)
iv)	*Eucalyptus*	13 811 (1468.7)	81.6 (8.3)	35.40 (6.7)	6.65 (0.6)
v)	Radiata pine	4 542 (1 024.3)	49.6 (10.3)	16.4 (2.3)	3.1 (0.5)

NOTE — Values in the parentheses indicate standard deviation.

Table 28 Mechanical Properties of Hybrid CLT Panels

(*Clause* 20.3.1)

Sl No.	CLT Made from	MoE	MoR	Compression Parallel to Grain	Block Shear Strength
		MPa	MPa	MPa	MPa
(1)	(2)	(3)	(4)	(5)	(6)
i)	EME	10 943 (1 074.8)	70.0 (7.2)	30.5 (3.7)	5.8 (0.6)
ii)	MEM	7 770 (938.5)	47.0 (4.5)	24.6 (2.2)	5.8 (0.6)
iii)	SMS	7 701 (600.5)	61.6 (7.5)	33.5 (4.0)	5.6 (0.5)
iv)	MSM	6 328 (652.8)	45.5 (4.3)	24.2 (3.1)	5.3 (0.5)

NOTE — Values in the parentheses indicate standard deviation.

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20.4 Fire Safety Requirements

20.4.1 *Fire Resistance Ratings*

CLT panels should have a fire-resistance rating in accordance with Part F 'Fire and Life Safety' of this NBCS. This shall be achieved through inherent fire resistance, encapsulation with materials like gypsum board (minimum thickness 13 mm), or a combination of methods.

The charring rate of CLT shall be considered in fire design calculations, with a recommended charring rate of 0.65 mm/min for calculating the load-bearing capacity during fire exposure.

20.4.2 *Protection of Connections*

Steel connectors and other metal components shall be protected with fire-resistant coatings or encased in non-combustible materials to ensure structural integrity during a fire.

20.5 Durability and Biological Resistance

20.5.1 *Preservative Treatments*

CLT panels shall be treated with preservatives like bifenthrin or copper chrome borate to enhance resistance to biological degradation. Testing has shown that preservative-treated CLT has improved resistance to termite and fungal attacks compared to untreated samples (*see* Table 29 for termite degradation results).

Table 29 Degradation of Samples due to Termite Attack

(*Clause* 20.5.1)

Sl No.	Sample	Average Degradation	Duration of Exposure
		percent	months
(1)	(2)	(3)	(4)
i)	Control solid wood	100	6
ii)	Control CLT	100	9
iii)	Calcium channel blockers (CCB) treated CLT	60	36
iv)	Bifenthrin treated CLT	0	36

20.5.2 *Performance Under Biological Exposure*

Durability testing shall include assessments for resistance to termites, white rot and brown rot fungi. Untreated rubberwood is highly susceptible to biological degradation, while treated samples showed minimal degradation after prolonged exposure.

20.5.3 *Effect of Preservatives on Mechanical Properties*

The use of preservatives should not adversely affect the mechanical properties of CLT. Block shear strength tests indicate that treated panels retain comparable mechanical performance to untreated panels.

20.6 Handling, Storage and Installation

20.6.1 *Moisture Protection*

During construction, CLT panels shall be protected from moisture exposure. Panels should be stored in a dry environment and water infiltration during transportation and installation shall be minimized.

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21 MASS TIMBER

Structural elements primarily composed of solid, built-up, panelized, or engineered wood products are categorized as products which include, but are not limited to, cross-laminated timber (CLT), glued-laminated timber (glulam), and laminated veneer lumber (LVL). It consists of multiple layers of wood, mechanically or adhesively bonded together to create large, strong and stable components suitable for load-bearing and non-load-bearing uses in construction. Mass timber elements can be used for structural framing, walls, floors, roofs and columns. It applies to the construction of residential, commercial, and public buildings where structural elements, including floors, walls, roofs and columns, are made from engineered wood products like cross-laminated timber (CLT), glued-laminated timber (glulam) and other solid or panellised wood products.

These load-bearing and non-load-bearing products include:

a) Cross-laminated timber (CLT);

b) Glued-laminated timber (Glulam);

c) Nail-laminated timber (NLT); and

d) Laminated veneer lumber (LVL).

21.1 Design and Structural Integrity

21.1.1 *Structural Design Requirements*

a) *Load-bearing capacity* — Mass timber elements shall be designed to carry the vertical and lateral loads imposed on the building in compliance with Part C ‘Structural Design, Section 1 Loads and Load Effects’ of this NBCS. The design shall account for dead loads, live loads, wind loads, seismic loads and other imposed loads;

b) *Cross-sectional dimensions* — The minimum cross-sectional dimensions for mass timber elements (for example, columns, beams, floor slabs) shall be specified based on building height, function and occupancy. For example, CLT panels used for floor slabs should meet a minimum thickness of 100 mm, depending on the structural requirements; and

c) *Deflection limits* — The design of mass timber structures shall ensure that the deflection under service load conditions remains within acceptable limits as defined in this Part of this NBCS.

21.1.2 *Hybrid Systems*

Where mass timber is used in conjunction with steel or concrete, the design should ensure that load paths are continuous and that materials are compatible. Load transfer mechanisms shall be adequately designed to prevent failure at the connection points between materials.

21.1.3 *Seismic Performance*

Mass timber buildings located in seismic zones should be designed to accommodate lateral loads and displacements as per Part C ‘Structural Design, Section 1 Loads and Load Effects’ of this NBCS. Timber's flexibility can offer good seismic performance, but connections and structural integrity should be robustly designed to prevent failure.

21.2 Fire and Life Safety

It should be ensured in accordance with Part F ‘Fire and Life Safety’ of this NBCS.

21.2.1 *Protection of Connections*

All steel connectors, bolts and fasteners used in mass timber construction shall be protected with fire-resistant coatings or enclosed within non-combustible materials to prevent failure during fire exposure.

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21.3 Minimum Dimensions and Thickness of Mass Timber Elements

21.3.1 *Columns and Beams*

Mass timber columns and beams shall be designed to meet the structural load requirements with a minimum cross-sectional dimension of 200 mm for columns and 150 mm for beams, depending on the span and load requirements.

21.3.2 *Floor and Roof Panels*

CLT panels used for floor and roof assemblies should meet a minimum thickness of 100 mm for floor systems and 85 mm for roof systems to achieve the required fire and structural performance.

21.3.3 *Wall Panels*

CLT or NLT wall panels shall meet a minimum thickness of 125 mm to ensure fire resistance and structural stability under lateral and vertical loads.

21.4 Construction Practices

21.4.1 *Handling and Installation of Mass Timber Elements*

During construction, care should be taken to protect mass timber elements from moisture. Elements shall be stored in dry conditions and protected from water exposure during construction to prevent damage and degradation.

21.4.2 *Assembly and Connections*

Connections between mass timber elements (for example, floor-to-wall, beam-to-column) should be engineered to handle both static and dynamic loads. These connections shall be made using metal connectors, dowels, or bolts designed to prevent movement and ensure continuous load transfer. Special attention should be paid to fire protection at these connection points.

21.4.3 *Fire Sealing of Penetrations*

Any penetrations for services (for example, plumbing, electrical) through mass timber elements shall be sealed with approved fire-rated materials to maintain the integrity of the fire-rated assembly.

21.5 Sustainability and Environmental Benefits

21.5.1 *Environmental Performance*

Mass timber is recognized for its carbon sequestration properties, where wood captures and stores carbon dioxide. Buildings made from mass timber reduce the overall carbon footprint of the construction process compared to traditional materials like steel and concrete.

21.5.2 *Energy Efficiency*

Mass timber inherent thermal insulation properties help to regulate indoor temperatures, thereby reducing energy consumption for heating and cooling. The thermal resistance of wood should be considered in the building envelope design, allowing for thinner wall assemblies with better insulation compared to conventional materials.

22 LIFE CYCLE ASSESSMENT (LCA) OF TIMBER FOR USE IN BUILDINGS

22.1 Timber used in building structures and elements shall undergo a comprehensive life cycle assessment (LCA) to evaluate its environmental impacts across its entire lifecycle, from raw material extraction to disposal or recycling. LCA provides valuable insights into the sustainability of timber and its environmental footprint in building applications. The assessment shall cover the following stages:

a) *Production and Construction* — This stage evaluates the energy and resources required for the extraction, transportation and processing of raw timber into building materials. It also includes the energy

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consumption and waste generated during the construction process, as well as the transportation of materials to the construction site.

b) *Use Stage* — The use stage assesses the environmental impacts of timber during the building's operational lifespan. This includes maintenance, repairs and potential replacement of timber components, considering the durability, performance and long-term sustainability of the material.

c) *End-of-Life* — At the end of the building's useful life, the timber structure may undergo demolition, with the materials being processed for waste management, recycling, or reuse. This stage assesses the environmental effects of these processes and the potential for timber materials to be repurposed.

d) *Externalized Impacts beyond the System Boundary* — The incidental effects of recycling, reusing and recovering energy, water, or materials beyond the system boundaries of the LCA study are included in this stage. These externalized impacts consider how recycled timber or materials might contribute to environmental benefits outside the building's lifecycle.

22.2 Environmental Impact Categories

The LCA may be used to assess the environmental impact of timber buildings in several categories, which encompass emissions and pollutants generated throughout the lifecycle. These categories include:

a) *Global Warming Potential* — The contribution of greenhouse gas emissions to climate change;

b) *Acidification Potential* — The potential for acidic compounds to form in the environment, affecting ecosystems and infrastructure;

c) *Eutrophication Potential* — The impact of nutrient loading on water bodies, potentially causing excessive algal growth;

d) *Ozone Depletion Potential* — The impact of substances that contribute to the depletion of the ozone layer; and

e) *Smog Formation Potential* — The potential for air pollution to contribute to the formation of ground-level ozone and smog.

All relevant material and process quantities shall be included in the LCA inventory and their impacts shall be calculated using recognized tools and methodologies.

22.3 Requirements for Life Cycle Assessment

In the context of building with timber, LCA may be required for the following purposes:

a) *Decision-Making Support* — Providing building owners and stakeholders with data on the environmental and sustainability aspects of timber, aiding informed decision-making;

b) *Design Evaluation* — Offering insights into material selection and environmental performance across different design possibilities;

c) *Product Comparison* — Demonstrating the environmental advantages of timber in comparison to alternative building materials;

d) *Environmental Benefit Evaluation* — Helping assess the ecological benefits of innovative timber products or policies; and

e) *Performance Benchmarking* — Comparing the environmental performance of timber buildings against established benchmarks and sustainability goals.

22.4 Methodology for Conducting LCA

The LCA of timber buildings may be conducted iteratively, with increasing detail as the design or construction phases progress. The process should follow these steps:

a) Establish the purpose and scope of the LCA, focusing on the objectives of evaluating timber's environmental impact;

b) Gather data on the timber materials, including their types, quantities, lifespans and lifecycle stages. Special attention should be given to materials used in the production and use stages;

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c) Using LCA tools (for example, SimaPro, MATLAB), assess the environmental impacts of timber buildings. The evaluation shall include biogenic carbon considerations, as timber stores carbon from the atmosphere, contributing to carbon sequestration during its lifecycle;

d) Interpret the LCA results by categorizing environmental impacts by building component, material type, and lifecycle stage. Identify key contributors to the building's overall environmental footprint and visualize the outcomes; and

e) Document all findings, including assumptions, methodology and conclusions and present the results in reports for stakeholders. The results shall guide sustainable construction practices and decision-making in timber building projects.

22.5 Reporting

The results of the LCA should be documented, including all assumptions, methodologies and conclusions. Reports should be prepared in accordance with relevant standards and may be used to guide sustainable building practices and decision-making in construction.

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ANNEX A

(*Clause* 4.1)

MECHANICAL PROPERTIES OF FOREIGN TIMBER SPECIES

A-1 The Table 30 below specifies the mechanical properties of selected foreign timber species, indicating their density, modulus of elasticity, permissible stresses, and preservative characteristics. This information is intended to serve as a reference for designers and engineers, particularly in cases where such species are imported for structural or allied applications, while ensuring that due consideration is given to relevant standards and treatment requirements.

Table 30 Mechanical Properties of Foreign Timber Species

(*Clause* A-1)

Sl No.	Species		Locality from where Tested	Average Density at 12 Percent Moisture Content	Modulus of Elasticity (All Grades and All Locations) × 10^3	Permissible Stress			
	Botanical Name	Trade Name				Bending-Tension Along Grain, Extreme Fibre Stress	Shear all Locations Horizontal	Compression Parallel to Grain	Compression Perpendicular to Grain
				kg/m³	N/mm²	N/mm²	N/mm²	N/mm²	N/mm²
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(8)	(8)
i)	*Pseudotsuga menziesii*	Douglas fir (DF)	Pacific NW, USA	513	13.4	Inside: 11.0 Outside: 9.4 Wet: 9.4	1.24 (Inside) 1.05 (Outside)	Inside: 11.7 Outside: 9.9 Wet: 9.9	2.9
ii)	*Tsuga heterophylla*	Western hemlock (WH)	Pacific NW, USA	500	11.2	Inside: 9.3 Outside: 7.9 Wet: 7.9	1.03 (Inside) 0.88 (Outside)	Inside: 9.3 Outside: 7.9 Wet: 7.9	2.4

NOTE — The above data on foreign timber species is reproduced from the *USDA Wood Handbook* (*USA*). This information is for reference only. At present, no indigenous test data under Indian climatic and service conditions is available for these species. Therefore, before adopting them in structural applications in India, the properties should be verified through appropriate laboratory testing as per accepted standards.

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LIST OF STANDARDS

The following list records those standards which are acceptable as 'good practice' and 'accepted standards' in the fulfilment of the requirements of this NBCS. The latest version of a standard shall be adopted at the time of the enforcement of this NBCS. The standards listed may be used for conformance with the requirements of the referred clauses in this NBCS.

In the following list, the number appearing in the first column within parenthesis indicates the number of the reference in this Subsection.

	IS No.	*Title*
(1)	IS 707 : 2011	Timber technology and utilization of wood, bamboo and cane — Glossary of terms (*third revision*)
(2)	IS 1708 (Parts 1 to 18) : 1986	Methods of testing small clear specimens of timber (*second revision*)
	IS 2408 : 1963	Methods of static tests of timbers in structural sizes
	IS 2455 : 1990	Methods of sampling of model trees and logs and their conversion for timer testing (*second revision*)
(3)	IS 4970 : 1973	Key for identification of commercial timbers (*first revision*)
(4)	IS 287 : 1993	Permissible moisture content of timber used for different purposes — Recommendations (*third revision*)
(5)	IS 3629 : 1986	Specification for structural timber in building (*first revision*)
(6)	IS 401 : 2001	Preservation of timber — Code of practice (*fourth revision*)
(7)	IS 2366 : 1983	Nail-jointed timber construction — Code of practice (*first revision*)
(8)	IS 4983 : 1968	Code of practice for design and construction of nailed laminated timber beams
(9)	IS 11096 : 1984	Code of practice for design and construction of bolt jointed timber construction
(10)	IS 4907 : 2004	Methods of testing timber connector joints (*first revision*)
(11)	IS 9188 : 1979	Performance requirements for adhesive for structural laminated wood products for use under exterior exposure conditions
(12)	IS 14616 : 1999	Laminated veneer lumber — Specification
(13)	IS 10701 : 2012	Structural plywood — Specification (*first revision*)
	IS 4990 : 2024	Plywood for concrete shuttering works — Specification (*fourth revision*)
(14)	IS 4924	Method of test for nail-jointed timber trusses:
	(Part 1) : 1968	Destructive test
	(Part 2) : 1968	Proof test
(15)	IS 12120 : 1987	Code of practice for preservation of plywood and other panel products
(16)	IS 4990 : 2024	Plywood for concrete shuttering works — Specification (*fourth revision*)
(17)	IS 9307 (Parts 1 to 8) : 1979	Methods of test for wood-based structural sandwich construction
(18)	IS 723 : 2023	Steel countersunk head wire nails — Specification (*second revision*)
(19)	IS 451 : 1999	Technical supply conditions for wood screws (*third revision*)
	IS 6736 : 1972	Specification for slotted raised countersunk head wood screw

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	IS No.	*Title*
	IS 6739 : 1972	Specification for slotted round head wood screws
	IS 6760 : 1972	Specification for slotted countersunk head wood screws
(20)	IS 1708 (Parts 1 to 18) : 1986	Methods of testing small clear specimens of timber (*second revision*)
(21)	IS 6313 (Part 1) : 1981	Code of practice for anti-termite measures in buildings: Part 1 Constructional measures (*first revision*)
	IS 6313	Anti-termite measures in buildings — Code of practice:
	(Part 2) : 2022	Pre-constructional chemical treatment measures (*fourth revision*)
	(Part 3) : 2022	Treatment for existing buildings (*fourth revision*)

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 3 TIMBER AND BAMBOO: 3B BAMBOO

BUREAU OF INDIAN STANDARDS

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National Building Code Sectional Committee, CED 46

FOREWORD

This NBCS (Part C/Subsection 3B) covers the general principles involved in the design of structural bamboo in buildings with regard to mechanical resistance and durability of structures. Design of both bamboo (round bamboo, split bamboo, glued laminated bamboo) and bamboo-based panels joined together with adhesives or mechanical fasteners are covered in this Subsection. It also covers minimum strength data, dimensional stability, grading requirements and traditional bamboo joints for quality assurance. Constructional aspects using bamboo, such as, work on site, fabrication of components off-site and their erection on site is also covered to the extent necessary to indicate and ensure the quality of material and standard of workmanship to comply with the assumptions of the design rules and the limitations.

Bamboo is a versatile resource possessing high strength-to-weight ratio and cost ratio and offers considerable ease in working with simpler tools. It has a long and well established tradition as a building material. The application of bamboo as a constructional material is largely based on established traditions and intuitions of forefathers throughout the tropical and sub-tropical regions.

Resilience coupled with light weight makes bamboo an ideal material for housing in disaster prone/earthquake prone areas. It has the capacity to withstand repetitive loadings and absorb more energy and show larger deflections before collapse and as such is safer under earth tremors. As the design guidelines were inadequate, the application of bamboo as an engineering material was largely based on practical and engineering experience. A need was therefore felt to develop a Code for design and construction with bamboo so as to cater to a number of social and trade advantages, engineering recognition and the improved status of bamboo as an engineering material. This was achieved by adding a new chapter in the 2005 revision of this code, covering structural design with bamboo. Forest Research Institute, Dehradun and some other organizations were engaged in bamboo research to establish its silviculture, botanical, entomological and pathological aspects besides creating a utilization base. All these works were utilized in the preparation of the 2005 version of this Subsection of this

Some of the suitable species grown in India and neighbouring countries are enlisted in Annex A along with their local names and source, for general information.

The bamboo culm has a tubular structure consisting essentially of nodes and inter-nodes. In the inter-nodes the cells are axially oriented while the nodes provide the transverse inter-connection. The disposition of the nodes and the wall thickness are significant in imparting strength to bamboo against bending and crushing. In a circular cross-section, bamboo is generally hollow and for structural purposes this form is quite effective and advantageous. Each of the species of bamboo has widely different characteristics affecting its usefulness as constructional material. The strength of bamboo culms, their straightness, lightness combined with hardness, range and size of hollowness make them potentially suitable for a variety of applications both structural and non-structural. With good physical and mechanical properties, low shrinkage and good average density, bamboo is well suited to replace wood in several applications, especially in slats and panel form. Mass production of bamboo components, if required, may be integrated with treatment facilities near to the site of plantation or site of construction, for speed and economy in construction.

The Section on structural design with bamboo was first published in 2005 as a part of National Building Code of India under Subsection 3B of Part 6. In the 1983 version of this code, timber was covered under Section 3 of Part 6 ‘Structural Design’ under the title ‘Wood’, which did not cover bamboo. In the 2005 version of this code, the scope of Section 3 was enlarged as Section 3 ‘Timber and bamboo’, which was subdivided into Subsection 3A ‘Timber’ and Subsection 3B ‘Bamboo’. This Subsection pertains to 3B ‘Bamboo’.

Key changes to the 2016 version included: inclusion of provisions on construction practices using bamboo that was addressed in erstwhile Part 7 ‘Construction Management, Practices and Safety’; updated provisions for bamboo trusses, bamboo foundations; bamboo as reinforcement in concrete; introduction of the use of bamboo-based materials for walling, flooring, roofing, and in-fill panels, along with provisions for reconstituted wood from bamboo; and updates to the terms and definitions related to bamboo-based panel products.

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With the standardization on design and construction aspects of bamboo in this Subsection of this code, it was expected to ensure durable structures, however, some specific guidelines were required in the event of tropical cyclones in the sub-continent and the traditional non-engineered structures generally constructed. Keeping this in view, some recommendations are given in Annex B which may be followed in addition to provisions given in the main text of this Subsection.

As a result of experience gained in implementation of 2016 version of this code and further research work done, feedback received as well as revision/updation of some of the existing standards, a need to revise this Subsection was felt. This revision has, therefore, been brought out to take care of these aspects.

In this revision, this publication has been renamed as National Building Construction Standards (NBCS).

The significant changes incorporated in this revision include the following:

a) Bamboo grading, in **4.4**, has been expanded to include special grades based on mean outer diameter;

b) Bamboo mat corrugated sheets (BMCS) are included, in **6.9.9**, as a sustainable roofing material with high strength, resilience, and thermal resistance for diverse climates;

c) Comprehensive life cycle assessment to evaluate environmental impacts from production to end-of-life of bamboo structures;

d) Annex C has been introduced illustrating conventional bamboo construction details such as trusses, roofing, walls, and flooring; and

e) References to all the concerned Indian Standards have been updated.

Reference may be made to Part F ‘Fire and Life Safety’ of this NBCS for fire protection of the bamboo buildings.

The information contained in this Subsection is largely based on the works carried out at Forest Research Institute, Dehradun; Institute of Wood Science and Technology, Bangalore; International Network for Bamboo and Rattan (INBAR) documents and Indian Standards given below. Assistance has also been derived from ISO 22156 : 2021 ‘Bamboo structures- Bamboo culms- Structural design’, apart from the following:

IS 9096 : 2006	Preservation of bamboo for structural purpose — Code of practice
IS 15912 : 2018	Structural design using bamboo — Code of practice

All standards, whether given herein above or cross-referred to in the main text of this Subsection, are subject to revision. The parties to agreement based on this Subsection are encouraged to investigate the possibility of applying the most recent editions of the standards.

As the regulation of land development and buildings is a State subject and is dealt by the States and local bodies such as urban local bodies within their jurisdiction through building regulatory documents like building regulations, building byelaws, and fire regulations, this document is voluntary in nature and is non-binding. The implementation depends on adoption by concerned parties or stipulations in a contract or by appropriate adoption by the concerned authorities.

For the purpose of deciding whether a particular requirement of this Subsection is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 2022 ‘Rules for rounding off numerical values (*second revision*)’. The number of significant places retained in the rounded off value should be the same as that of the specified value in this Subsection.

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Important Explanatory Note for Users of this NBCS

In any Part/Section of this NBCS, where reference is made to **'good practice'** in relation to design, constructional procedures or other related information, and where reference is made to **"accepted standard"** in relation to material specification, testing, or other related information, the Indian Standards listed at the end of the Part/Section shall be used as a guide to the interpretation.

At the time of publication, the editions indicated in the standards were valid. All standards are subject to revision and parties to agreements based on any Part/Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

In the list of standards given at the end of a Part/Section, the number appearing within parentheses in the first column indicates the number of the reference of the standard in the Part/Section. For example:

a) Good practices [C-3B(1)] refers to the Indian Standard(s) give at serial number (1) of the list of standards given at the end of this Subsection, that is, IS 9096 : 2006, 'Code of practice for preservation of bamboo for structural purposes (*first revision*)'

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 3 TIMBER AND BAMBOO: 3B BAMBOO

1 SCOPE

1.1 This NBCS (Part C/Subsection 3B) covers the general principles involved in the design of structural bamboo in buildings with regard to mechanical resistance and durability of structures. Design of both bamboo (round bamboo, split bamboo, glued laminated bamboo) and bamboo-based panels joined together with adhesives or mechanical fasteners are covered in this Subsection. It also covers minimum strength data, dimensional stability, grading requirements and traditional bamboo joints for quality assurance. Constructional aspects using bamboo, such as, work on site, fabrication of components off-site and their erection on site are also covered to the extent necessary to indicate and ensure the quality of material and standard of workmanship to comply with the assumptions of the design rules and the limitations.

1.2 The following aspects are not covered in this Subsection:

a) Limit state design and performance of structures; and

b) Scientific designing of bamboo joints and their fastenings.

2 TERMINOLOGY

For the purpose of this Subsection, the following definitions shall apply.

2.1 Anatomical Purpose Definitions

2.1.1 *Bamboo* — Tall perennial grasses found in tropical and sub-tropical regions. They belong to the family *Poaceae* and sub-family *Bambusoideae*.

2.1.2 *Bamboo Culm* — A single shoot of bamboo usually hollow except at nodes which are often swollen.

2.1.3 *Bamboo Clump* — A cluster of bamboo culms emanating from two or more rhizomes in the same place.

2.1.4 *Cellulose* — A carbohydrate, forming the fundamental material of all plants and a main source of the mechanical properties of biological materials.

2.1.5 *Cell* — A fundamental structural unit of plant and animal life, consisting of cytoplasm and usually enclosing a central nucleus and being surrounded by a membrane (animal) or a rigid cell wall (plant).

2.1.6 *Cross Wall* — A wall at the node closing the whole inside circumference and completely separating the hollow cavity below from that above (*Syn*. Diaphragm, that is, horizontal partition in a standing culm) (*see also* **2.2.19** and **2.2.32**).

2.1.7 *Hemi Cellulose* — The polysaccharides consisting of only 150 to 200 sugar molecules, also much less than the 10 000 of cellulose.

2.1.8 *Lignin* — A polymer of phenyl propane units, in its simple form ($C_6H_5CH_3CH_2CH_3$).

2.1.9 *Sliver* — Thin strips of bamboo processed from bamboo culm.

2.1.10 *Tissue* — Group of cells, which in higher plants consist of (a) Parenchyma — a soft cell of higher plants as found in stem pith or fruit pulp, (b) Epidermis — the outermost layer of cells covering the surface of a plant, when there are several layers of tissue.

2.2 Structural Purpose Definitions

2.2.1 *Bamboo Mat Board* — A board made of two or more bamboo mats bonded with an adhesive.

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2.2.2 *Bamboo Mat Corrugated Sheet* — A sheet made up of adhesive soaked and coated mats assembled and pressed under specified temperature and pressure to obtain sinusoidal or other suitable corrugations.

2.2.3 *Bamboo Mat Corrugated Sheets* (*BMCS*) — Roofing sheets made from woven bamboo mats, treated and pressed with resin to form a durable, weather-resistant corrugated material.

2.2.4 *Bamboo Mat Ridge Cap* (*BMRC*) — It is a specially shaped bamboo composite designed to cover and seal the ridge line of BMCS roofs, providing a weatherproof finish to the roof structure.

2.2.5 *Bamboo Mat Veneer Composite* — Panel manufactured with a combination of bamboo mat and veneer. Bamboo mat can be either as outer skins or as core/cross-bands. However, the composite panel shall be balanced construction on either side of central ply.

2.2.6 *Beam* — A structural member which supports load primarily by its internal resistance to bending.

2.2.7 *Breaking Strength* — A term loosely applied to a given structural member with respect to the ultimate load it can sustain under a given set of conditions.

2.2.8 *Bundle-Column* — A column consisting of three or more number of culms bound as integrated unit with wire or strap type of fastenings.

NOTE — During bunching of beams/columns, binders (5 mm tie bar/wire and 15 mm bamboo dowel pins) may be carefully inserted after proper pre-drilling the holes throughout the assembly as per the requirements of structural design.

2.2.9 *Centre Internode* — A test specimen having its centre between two nodes.

2.2.10 *Characteristic Load* — The value of load which has a 95 percent probability of not exceeding during the life of the structure.

2.2.11 *Characteristic Strength* — The strength of the material below which not more than 5 percent of the test results are expected to fall.

2.2.12 *Cleavability* — The ease with which bamboo can be split along the longitudinal axis. The action of splitting is known as cleavage.

2.2.13 *Column* — A structural member which supports axial load primarily by inducing compressive stress along the fibres.

2.2.14 *Common Rafter* — A roof member which supports roof battens and roof coverings, such as boarding and sheeting.

2.2.15 *Curvature* — The deviation from the straightness of the culm.

2.2.16 *Delamination* — Separation of mats through failure of glue.

2.2.17 *De-structured Bamboo* — Raw material (green bamboo) reduced partially to a condition, as far as practicable, in which the element fibres remain interlocked and in their highly aligned state and which with the help of binder can be reconstituted into a valuable product of a desirable size.

2.2.18 *End Distance* — The distance measured parallel to the fibres of the bamboo from the centre of the fastener to the closest end of the member.

2.2.19 *Flattened Bamboo* — Bamboo consisting of culms that have been cut and unfolded till it is flat. The culm thus is finally spread open, the diaphragms (cross walls) at nodes removed and pressed flat.

2.2.20 *Full Culm* — The naturally available circular section/shape.

2.2.21 *Fundamental or Ultimate Stress* — The stress which is determined on a specified type/size of culms of bamboo, in accordance with standard practice and does not take into account the effects of naturally occurring characteristics and other factors.

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2.2.22 *Inner Diameter* — Diameter of internal cavity of a hollow piece of bamboo.

2.2.23 *Inside Location* — Position in buildings in which bamboo remains continuously dry or protected from weather.

2.2.24 *Joint* — A connection between two or more bamboo structural elements.

2.2.25 *Joist* — A small beam directly supporting floor, ceiling or roof of a structure.

2.2.26 *Length of Internode* — Distance between adjacent nodes.

2.2.27 *Loaded End or Compression End Distance* — The distance measured from the centre of the fastener to the end towards which the load induced by the fastener acts.

2.2.28 *Machette* — A light cutting and slashing tool in the form of a large knife.

2.2.29 *Mat* — A woven sheet made using thin slivers.

2.2.30 *Mortise and Tenon* — A joint in which the reduced end (tenon) of one member fits into the corresponding slot (mortise) of the other.

2.2.31 *Net Section* — Section obtained by deducting from the gross cross-section (A), the projected areas of all materials removed by boring, grooving or other means.

2.2.32 *Node* — The place in a bamboo culm where branches sprout and a diaphragm is inside the culm and the walls on both sides of node are thicker.

2.2.33 *Outer Diameter* — Diameter of a cross-section of a piece of bamboo measured from two opposite points on the outer surface.

2.2.33.1 *Mean outer diameter* — The diameter taken twice on the outer surface, in direction perpendicular to each other at each cross-section on both the ends. Thus, resulting in mean of four readings for the middle of culm.

2.2.34 *Outside Location* — Position in building in which bamboos are occasionally subjected to wetting and drying as in case of open sheds; and outdoor exposed structures.

2.2.35 *Permissible Stress* — Stress obtained after applying factor of safety to the ultimate stress. This is also the working stress unless adjusted to the particular structural form in design.

2.2.36 *Principal Rafter* — A roof member which supports purlins.

2.2.37 *Purlins* — A roof member directly supporting roof covering or common rafter and roof battens.

2.2.38 *Roof Battens* — A roof member directly supporting tiles, corrugated sheets, slates or other roofing materials.

2.2.39 *Roof Skeleton* — The skeleton consisting of bamboo truss or rafter over which solid bamboo purlins are laid and lashed to the rafter or top chord of a truss by means of galvanized iron wire, cane, grass, bamboo leaves, etc.

2.2.40 *Slenderness Ratio* — The ratio of the length of member to the least radius of gyration is known as slenderness ratio of member (the length of the member is the equivalent length due to end conditions).

2.2.41 *Splits* — The pieces made from quarters by dividing the quarters radially and cutting longitudinally.

2.2.42 *Taper* — The ratio of difference between minimum and maximum outer diameter to length.

2.2.43 *Unloaded End Distance* — The end distance opposite to the loaded end.

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2.2.44 *Wall Thickness* — Half the difference between outer diameter and inner diameter of the piece at any cross section.

2.2.44.1 *Mean wall thickness* — The wall thickness taken four times in the same points as the diameter has been measured. Thus, resulting in mean of eight values for the middle of culm.

2.2.45 *Wet Location* — Position in buildings in which the bamboos are almost continuously damp, wet or in contact with earth or water, such as piles and bamboo foundations.

2.3 Definitions Relating to Defects

2.3.1 *Bamboo Bore/GHOON Hole* — The defect caused by bamboo ghoon beetle (*Dinoderus spp*. *Bostrichidae*), which attacks felled culms.

2.3.2 *Crookedness* — A localized deviation from the straightness in a piece of bamboo.

2.3.3 *Discolouration* — A change from the normal colour of the bamboo which does not impair the strength of bamboo or bamboo composite products.

2.4 Definition Relating to Drying Degrades

2.4.1 *Collapse* — The defect occurring on account of excessive shrinkage, particularly in thick walled immature bamboo. When the bamboo wall shrinks, the outer layers containing a larger concentration of strong fibro-vascular bundles set the weaker interior portion embedded in parenchyma in tension, causing the latter to develop cracks. The interior crack develops into a wide split resulting in a depression on the outer surface. This defect also reduces the structural strength of round bamboo.

2.4.2 *End Splitting* — A split at the end of a bamboo. This is not so common a defect as drying occurs both from outer and interior wall surfaces of bamboo as well as the end at the open ends.

2.4.3 *Surface Cracking* — Fine surface cracks not detrimental to strength. However, the cracking which occurs at the nodes reduces the structural strength.

2.4.4 *Wrinkled and Deformed Surface* — Deformation in cross section, during drying, which occurs in immature round bamboos of most species; in thick walled pieces, besides this deformation the outer surface becomes uneven and wrinkled. Very often the interior wall develops a crack below these wrinkles, running parallel to the axis.

3 SYMBOLS

For the purpose of this Subsection, the following letter symbols shall have the meaning indicated against each, unless otherwise stated:

A = cross-sectional area of bamboo (perpendicular to the direction of the principal fibres and vessels), mm^2

= $\frac{\pi}{4}[D^2 - (D - 2w)^2]$;

A_B = area of reinforcement in bamboo reinforced cement concrete, mm^2;

D = mean outer diameter of bamboo culm, mm;

d = inner diameter, mm

= $(D - 2w)$;

E = modulus of elasticity (MOE) in bending, N/mm^2;

f_c = calculated stress in axial compression, N/mm^2;

f_{cp} = permissible stress in compression along the fibres, N/mm^2;

I = moment of inertia (the second moment of area) of culm section, mm^4

= $\frac{\pi}{64}[D^4 - (D - 2w)^4]$;

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ℓ = unsupported length of column, mm;

m = moisture content, percent;

r = radius of gyration, mm

= $\sqrt{\frac{I}{A}}$;

R = modulus of rupture (MOR), N/mm^2;

w = mean of wall thickness of bamboo culm, mm;

Z = section modulus, mm^3

= $\frac{\pi}{32}\left[\frac{D^4-(D-2w)^4}{D}\right]$; and

δ = deflection or deformation, mm.

4 MATERIALS

The requirements shall be met by the choice of suitable materials, by appropriate design and detailing and by specifying control procedures for production, construction and use.

4.1 Species of Bamboo

More than 100 species of bamboo are native to India and a few of them are solid but most of them are hollow in structure. In all 20 species have been systematically tested so far. Sixteen species of bamboo are recommended for structural usages in round form. Some physical and mechanical properties of the same are given in Table 1 both for air-dry conditions and green conditions.

4.1.1 Matured bamboo of at least 4 years of age shall be used. The bamboo shall be used after at least six weeks of felling period.

4.1.2 Bamboo shall be properly treated in accordance with the good practice [C-3B(1)].

4.1.3 Solid bamboos or bamboos whose wall thickness is comparatively more and which are generally having nodes very closer are often considered good for structural purposes.

4.1.4 Broken, damaged or collapsed bamboo shall be rejected. Dead and immature bamboos, bore/*Ghoon* holes, decay, collapse, checks more than 3 mm in depth, shall be avoided.

4.2 Grouping

4.2.1 Sixteen species of bamboo recommended for structural applications are classified in three groups on the basis of their strength properties, namely, modulus of elasticity (E) in bending in green condition and modulus of rupture (R) (extreme fibre stress in bending). For structural use in piles and mines' components, an additional criterion of compressive strength is considered in classification. The limits in ultimate strength values of these groups shall be as given in Table 2 (these are corresponding to the spread of properties in structural timber).

4.2.2 Bamboo species may be identified using suitable methods.

NOTE — Methods of identification of bamboo through anatomical characters have not been perfected so far. Identification through morphological characters could be done only on full standing culm by experienced sorters.

4.2.3 Species of bamboo other than those listed in the Table 3 may be used, provided their strength characteristics are determined and found more than the limits specified in Table 2.

4.3 Moisture Content in Bamboo

Normally only dry or seasoned bamboo shall be used otherwise special attention shall be given to dimensional changes occurring during the drying process in the joints, assemblies.

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With decrease of moisture content, the strength of bamboo increases exponentially and bamboo has an intersection point (fibre saturation point) at around 25 percent moisture content depending upon the species. A typical moisture strength relationship is given at Fig. 1. The moisture content of bamboo shall be determined in accordance with method given in the accepted standard [C-3B(2)]. Matured culms shall be seasoned to about 15 percent moisture content before use.

Freshly felled bamboo with shoots and leaves intact, dry fairly well in the open sun with lesser surface cracking, etc, and reduced starch contents. Rapid drying in the open can control degrades.

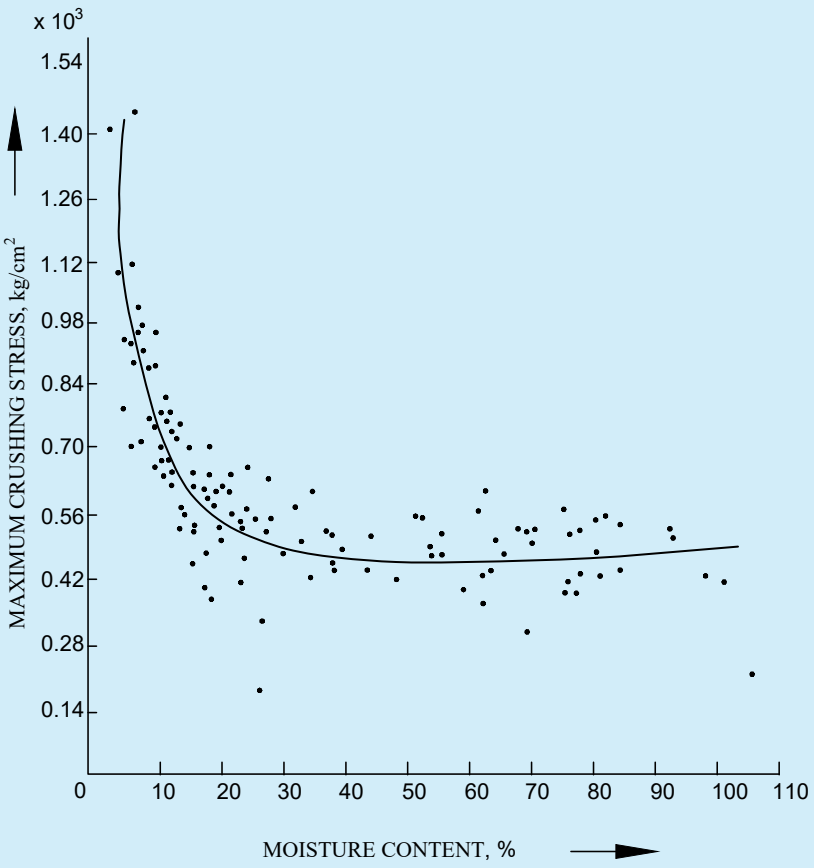

FIG. 1 MOISTURE STRENGTH RELATIONSHIP OF BAMBUSA NUTANS (BAMBOO)

4.4 Grading of Structural Bamboo

Bamboo shall be graded to ensure that the properties of bamboo are satisfactory for use, particularly the strength and stiffness properties. Grading bamboo is a process that ensures its structural integrity, standardization, efficient use, and marketability. It plays a crucial role in enhancing trade and promoting sustainable harvesting by ensuring that bamboo meets the required specifications for various applications. This process is essential for ultimately producing safe and economical structures, encouraging its use in mainstream construction and other industries.

4.4.1 Grading is sorting out bamboo on the basis of characteristics important for structural utilization as under:

a) Diameter and length of culm;

b) Taper of culm;

c) Straightness of culm;

d) Inter nodal length and distribution of nodes;

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e) Wall thickness;

f) Density and strength; and

g) Durability and seasoning.

4.4.2 *Diameter and Length of Culm*

4.4.2.1 *Gradation according to the mean outer diameter*

For structural Group A and Group B species, culms shall be segregated in steps of 10 mm of mean outer diameter as follows:

a) Special grade A : 100 mm < Diameter ≤ 130 mm

b) Special grade B : 70 mm < Diameter ≤ 100 mm

c) Grade I : 50 mm < Diameter ≤ 70 mm

d) Grade II : 30 mm < Diameter ≤ 50 mm

e) Grade III : Diameter ≤ 30 mm

For structural Group C species culms shall be segregated in steps of 20 mm of mean outer diameter as follows:

a) Grade I : 80 mm < Diameter ≤ 100 mm

b) Grade II : 60 mm < Diameter ≤ 80 mm

c) Grade III : Diameter ≤ 60 mm

4.4.2.2 The minimum length of culms shall be preferably 4.5 m for facilitating close fittings at joints, etc.

4.4.3 *Taper*

The taper shall not be more than 5.8 mm per metre length (or 0.58 percent) (1 in 170) of bamboo in any grade of bamboo.

4.4.4 *Curvature*

The maximum curvature should not be more than 75 mm in a length of 6 m of any grade of bamboo.

NOTE — Due to the constraint of preferential selection of straighter material out of a lot, more bent material may be accommodated to avoid high cost. Following remedial options may be helpful:

a) Adopt component assembly method to take care of some degree of curvature (bunching done after holding components straight).

b) Cut longer bamboo components with less curvature out of taller bamboo and use cambered ones for shorter length components.

4.4.5 *Wall Thickness*

Preferably minimum wall thickness of 8 mm shall be used for load bearing members unless calculations and the availability dictate otherwise.

4.5 Durability and Treatability

4.5.1 *Durability*

The natural durability of bamboo is low and varies between 12 months and 36 months depending on the species and climatic conditions. In tropical countries the bio-deterioration is very severe. Bamboos are generally destroyed in about one to two years' time when used in the open and in contact with ground; while a service life of two to five years can be expected from bamboo when used under cover and out of contact with ground. The mechanical strength of bamboo deteriorates rapidly with the onset of fungal decay in the sclerenchymatous fibres. Split bamboo is more rapidly destroyed than round bamboo

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4.5.2 Fire safety of bamboo structures shall be ensured, for which the treatment prescribed in **7.3.4** may be followed.

5 PERMISSIBLE STRESSES

5.1 Ultimate stress values of different species and groups of bamboo shall be determined according to the accepted standard [C-3B(2)]. These values shall then be divided by appropriate factors of safety to obtain permissible stresses to cover the effects of variability, long term loading, grade, location of use and expected dynamic loading.

NOTE — As an information, it may be noted that the method of test for split bamboos is covered in the accepted standard [C-3B(4)].

5.1.1 The strength factors for deriving safe working stresses of bamboo shall be as under:

a)	Extreme fibre stress in beams	:	4
b)	Modulus of elasticity	:	4.5
c)	Maximum compressive stress parallel to grain/fibres	:	3.5

5.2 The available data for the safe working stresses for 16 species of bamboos are given in Table 3.

5.3 For change in duration of load other than continuous (long-term), the permissible stresses given in Table 3 shall be multiplied by the modification factors given below:

a)	For permanent loads	:	1.0
b)	For imposed or medium term loading (permanent + temporary load)	:	1.25
c)	For short term loading (permanent + temporary load + wind load)	:	1.50

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Table 1 Physical and Mechanical Properties of Indian Bamboos (in Round Form)

(*Clause* 4.1)

Sl No.	Species	Properties							
		In Green Condition				In Air Dry Condition			
		Density	Modulus of Rupture	Modulus of Elasticity	Maximum Compressive Strength	Density	Modulus of Rupture	Modulus of Elasticity	Maximum Compressive Strength
		kg/m^3	N/mm^2	× 10^3 N/mm^2	N/mm^2	kg/m^3	N/mm^2	× 10^3 N/mm^2	N/mm^2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
i)	*Bambusa auriculata*	594	65.1	15.01	36.7	670	89.1	21.41	54.3
ii)	*B. balcooa*	783	65.4	7.31	46.7	–	–	–	60.6
iii)	*B. bambos* (*Syn. B. arundinacea*)	559	58.3	5.95	35.3	663	80.1	8.96	53.4
iv)	*B. burmanica*	570	59.7	11.01	39.9	672	105.0	17.81	65.2
v)	*B. glancescens* (*Syn. B. nana*)	691	82.8	14.77	53.9	–	–	–	–
vi)	*B. nutans*	603	52.9	6.62	45.6	673	52.4	10.72	47.9
vii)	*B. pallida*	731	55.2	12.90	54.0	–	–	–	–
viii)	*B. tulda*	658	51.1	7.98	40.7	722	66.7	10.07	68.0
ix)	*B. ventricosa*	626	34.1	3.38	36.1	–	–	–	–
x)	*B. vulgaris*	626	41.5	2.87	38.6	–	–	–	–
xi)	*Cephalostachyum pergracile*	601	52.6	11.16	36.7	640	71.3	19.22	49.4
xii)	*Dendrocalamus longispathus*	711	33.1	5.51	42.1	684	47.8	6.06	61.1

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Table 1 (*Concluded*)

Sl No.	Species	Properties							
		In Green Condition				In Air Dry Conditions			
		Density	Modulus of Rupture	Modulus of Elasticity	Maximum Compressive Strength	Density	Modulus of Rupture	Modulus of Elasticity	Maximum Compressive Strength
		kg/m^3	N/mm^2	× 10^3 N/mm^2	N/mm^2	kg/m^3	N/mm^2	× 10^3 N/mm^2	N/mm^2
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
xiii)	*D. strictus*	631	73.4	11.98	35.9	728	119.1	15.00	69.1
xiv)	*Melocanna baccifera*	817	53.2	11.39	53.8	751	57.6	12.93	69.9
xv)	*Oxytenanthera abyssinicia*	688	83.6	14.96	46.6	–	–	–	–
xvi)	*Thyrsostachys oliveri*	733	61.9	9.72	46.9	758	90.0	12.15	58.0

NOTE — The values of stress in N/mm^2 have been obtained by converting the values in kgf/cm^2 by dividing the same by 10.

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Table 2 Properties of Structural Bamboo

(*Clauses* 4.2.1 *and* 4.2.3)

Sl No.	Group	Modulus of Rupture (*R*)	Modulus of Elasticity in Bending (*E*)	Maximum Compressive Strength ($f_{c,\,max}$)
		N/mm^2	× 10^3 N/mm^2	N/mm^2
(1)	(2)	(3)	(4)	(5)
i)	A	$R > 70$	$E > 9$	$MCS > 35$
ii)	B	$70 \geq R > 50$	$9 \geq E > 6$	$35 > MCS > 30$
iii)	C	$50 \geq R > 30$	$6 \geq E > 3$	$30 > MCS > 25$

Table 3 Safe Permissible Stresses of Bamboos for Structural Designing[1)]

(*Clauses* 4.2.3, 5.2 *and* 5.3)

Sl No.	Species	Extreme Fibre Stress in Bending	Modulus of Elasticity	Allowable Compressive Stress
		N/mm^2	× 10^3 N/mm^2	N/mm^2
(1)	(2)	(3)	(4)	(5)
i)	Group A:			
	a) *Bambusa glancescens* (Syn. B.nana)	20.7	3.28	15.4
	a) *Dendrocalamus strictus*	18.4	2.66	10.3
	b) *Oxytenanthera abyssinicia*	20.9	3.31	13.3
ii)	Group B:			
	a) *Bambusa balcooa*	16.4	1.62	13.3
	b) *B.pallida*	13.8	2.87	15.4
	c) *B.nutans*	13.2	1.47	13.0
	d) *B.tulda*	12.8	1.77	11.6
	e) *B.auriculata*	16.3	3.34	10.5
	f) *B.burmanica*	14.9	2.45	11.4
	g) *Cephalostachyum pergracile*	13.2	2.48	10.5
	h) *Melocanna baccifera* (*Syn. M.bambusoides*)	13.3	2.53	15.4
	j) *Thyrsotachys oliveri*	15.5	2.16	13.4
iii)	Group C:			
	a) *Bambusa arundinacea* (*Syn. B.bambos*)	14.6	1.32	10.1
	b) *B.ventricosa*	8.5	0.75	10.3

[1)] The values given pertain to testing of bamboo in green condition.

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Table 3 (*Concluded*)

Sl No.	Species	Extreme Fibre Stress in Bending	Modulus of Elasticity	Allowable Compressive Stress
		N/mm^2	$\times 10^3 N/mm^2$	N/mm^2
(1)	(2)	(3)	(4)	(5)
c)	*B.vulgaris*	10.4	0 .64	11.0
d)	*Dendrocalamus longispathus*	8.3	1.22	12.0

NOTE — The values of stress in N/mm^2 have been obtained by converting the values in kgf/cm^2 by dividing the same by 10.

NOTE — The above properties are based on the accepted standard [C-3B(2)].

6 DESIGN CONSIDERATIONS

6.1 Basic Requirements

A structure shall be designed and constructed by personnel having the appropriate skill and experience in such a way that:

a) With acceptable probability, it will remain fit for the use for which it is required, having due regard to its intended life and costs;

b) With appropriate degree of reliability, it will sustain all forces and effects likely to occur during execution and use and have adequate durability in relation to maintenance costs;

c) The structure will be adequately maintained;

d) The structure will be used in accordance with design limit briefs;

e) All structural members, assemblies or framework in a building shall be capable of sustaining, without exceeding the limits of relevant stress specified, the worst combination of all loadings;

f) A fundamental aspect of design will be to determine the forces to which the structure/structural element might be subjected to, starting from the roof and working down to the soil by transferring the forces through various components and connections;

g) Accepted principles of mechanics for analysis and specified design procedures shall be applied (or) the design requirements may be satisfied by prototype testing;

h) Loads shall be in accordance with Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS, keeping into consideration that permeability of walls, floors and roof made from bamboo cause internal pressure and can therefore affect the wind load factor; and

j) The worst combination and location of loads shall be considered for design. Also, wind and seismic forces shall not be considered to act simultaneously.

6.2 Unlike timber, bamboo properties do not relate well to species, being dependent among other factors, on position of the culm, geographic location and age. The practice in timber engineering is to base designs on safe working stresses (*see* Note) and the same may be adopted to bamboo with the limitations that traditional experience rather than precise calculations generally govern the detailing.

NOTE — Limit states are states beyond which the structure no longer satisfies the design performance requirements. Limit states are classified into ultimate limit state relating to collapse mode and serviceability limit states associated with deformations, vibrations, etc. Design is usually based on ultimate stages with partial factors of safety.

6.3 Age old traditional experience of construction with bamboo is to be well preserved as non-standard practices for similar situations considered as an informal based on general social pattern and wisdom. Confirmation of such structures be based on reports after they have sustained the severity of earthquakes, cyclones, etc, as criteria for recommendations by the evaluation by competent engineer/builder with adequate experience in the field.

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6.4 Net Section

It is determined by passing a plane or a series of connected planes transversely through the members. Least net sectional area is used for calculating load carrying capacity of a member. In the design of an intermediate or a long column, gross section shall be used in calculating load carrying capacity of column.

6.5 Structural Components

6.5.1 Main structural applications in bamboo may include roofing and flooring, shear walls, wall panelling, beams, piles, columns, arches, etc. Both from the point of view of capacity and deformation, trusses and framed skeletons are much better options in bamboo.

6.5.2 *Peculiarity of Bamboo as a Design Material*

This shall be based on the principles of applied mechanics involving the following assumptions:

a) Elastic behaviour of bamboo, until failure (plastic behaviour being considered to be not significant);

b) Bamboo culms are analyzed as hollow-tube structures (not perfectly straight) member on mean diameter and mean wall thickness basis. Designing as a variable thickness hollow-tube structure is a bit complex;

c) Nodes do not occur at constant intervals;

d) Structural elements of bamboo shall be appropriately supported near the nodes of culm as and where the structural system demands. The joints in the design shall be located near nodes;

e) Bamboo structures are designed like any other conventional structural analysis taking care of details with regards to supports and joints; and the diameter, wall thickness and initial curvature; and

f) Joints shall be considered to generally act as a hinge or as pinned, unless substantiating data justify a fixed joint.

6.6 Flexural Members

6.6.1 All flexural members may be designed using the principles of beam theory.

NOTE — The tendency of bamboo beams to acquire larger deflections under long continual loading (including creep) has to be taken care of either by giving a factor of safety to E or by considering double the design dead loading in calculation of deflections.

6.6.2 The moment of inertia, I shall be determined as follows:

a) Outside diameter and the wall thickness shall be measured at both ends, correct up to 1 mm for diameter of culm and 0.1 mm for the wall thickness; and

b) With these values the mean diameter and the mean thickness for the middle of the beam shall be calculated and moment of inertia determined.

6.6.3 The maximum bending stress shall be calculated and should be within the allowable stress.

6.6.4 The deflection shall be within the prescribed limits. The initial curvature shall also be considered in the calculation of the deflection.

6.6.5 The shear stress in the neutral layer at the small end shall be checked, if the length of the beam is less than 25 times the diameter at that end.

6.6.6 Forces acting on a beam, being loads or reaction forces at supports, shall act in nodes or as near to nodes as by any means possible.

6.7 Columns (Predominantly Loaded in Axial Direction)

6.7.1 Columns and struts are essential components sustaining compressive forces in a structure. They transfer load to the supporting media.

6.7.2 Design of columns shall be based on one of the following two criteria:

a) Full scale buckling tests on the same species, size and other relevant variables; and

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b) Calculations, based on the following:

1) Moment of inertia shall be determined as per **6.6.2**;

2) For bamboo columns the best available straight bamboo culms shall be selected; and

3) Bending stresses due to initial curvature, eccentricities and induced deflection shall be taken into account, in addition to those due to any lateral load.

NOTE — Unlike timber, MOE has already been provided a factor of safety in case of bamboo. This is on the basis of studies on variability of MOR and MOE. In the case of timber, no factor of safety for MOE was used because there has been an understanding that there is no variability in MOE. Subsequently it was confirmed that MOE varies as the MOR and there is very strong, highly significant correlation between the two. In fact, this correlation has formed the basis of development of machine grading of timber world-over. However, somehow the practice of not giving factor of safety to MOE continued in Indian Standard. Later on when the standard on structural use of bamboo was formulated, factor of safety was given both in MOR and MOE on the basis of latest variability studies on strength of bamboo.

6.7.3 Buckling calculation shall be according to Euler, with a reduction to 90 percent of moment of inertia, to take into account the effect of the taper which shall be less than 1 : 170.

6.7.4 For strength and stability, larger diameter thick walled sections of bamboo with closely spaced nodes shall be used. Alternatively, smaller sections may be tied together as a bundle-column.

6.8 A single bamboo culm is not an optimum component to structurally design different load bearing members. Bunching of bamboo is inevitable to fabricate beams, columns, etc. There is no end to the number of culmns in bunching. However, different components pose manufacturing problems during assembly, which should be appropriately taken care of (*see also* Fig. 9D and Fig. 9E).

6.8.1 Following precautions shall be taken to avoid possible split/crack in walling of bamboo:

a) Maintaining a minimum 100 mm distance of the drills from the end of the main bamboo culms drilled;

b) Choosing drilling points preferably within 30 mm from node;

c) Maintaining the interference fit between bamboo body and binders as under:

1) For wall thickness ≥ 8 mm : 0.25 to 0.30 mm; and

2) For wall thickness < 8 mm : 0.20 to 0.25 mm.

d) The inserting ends of the dowel pins may be dressed as tapered (conical).

6.9 Assemblies and Roof Trusses

6.9.1 Elements in structure are generally built-up in the form of assembled members for which a triangle is a simple figure of stability. Besides sloped chords, parallel chord construction is also appropriate as external profile.

6.9.2 A truss is essentially a plane structure which is very stiff in the plane of the members, that is the plane in which it is expected to carry load, but very flexible in every other direction. Roof truss generally consists of a number of triangulated frames, the members of which are fastened at ends and the nature of stresses at joints is either tensile or compressive and designed as pin-ended joints (*see* Fig. 2A). Bamboo trusses may also be formed using bamboo mat board or bamboo mat-veneer composite or plywood gusset (*see* Fig. 2B).

6.9.3 Trusses shall be analysed for the determination of axial forces in members and the deformation of joints. For the influence of eccentricities, due allowance shall be made in design.

6.9.4 The truss height shall exceed 0.15 times the span in case of a triangular truss (pitched roofing) and 0.10 times the span in case of a rectangular (parallel) truss.

6.9.5 For members in compression, the effective length for in-plane strength verification shall be taken as the distance between two adjacent points of contraflexure. For fully triangulated trusses, effective length for simple span members without especially rigid end-connection shall be taken as the span length.

6.9.6 The spacing of trusses shall be consistent with use of bamboo purlins.

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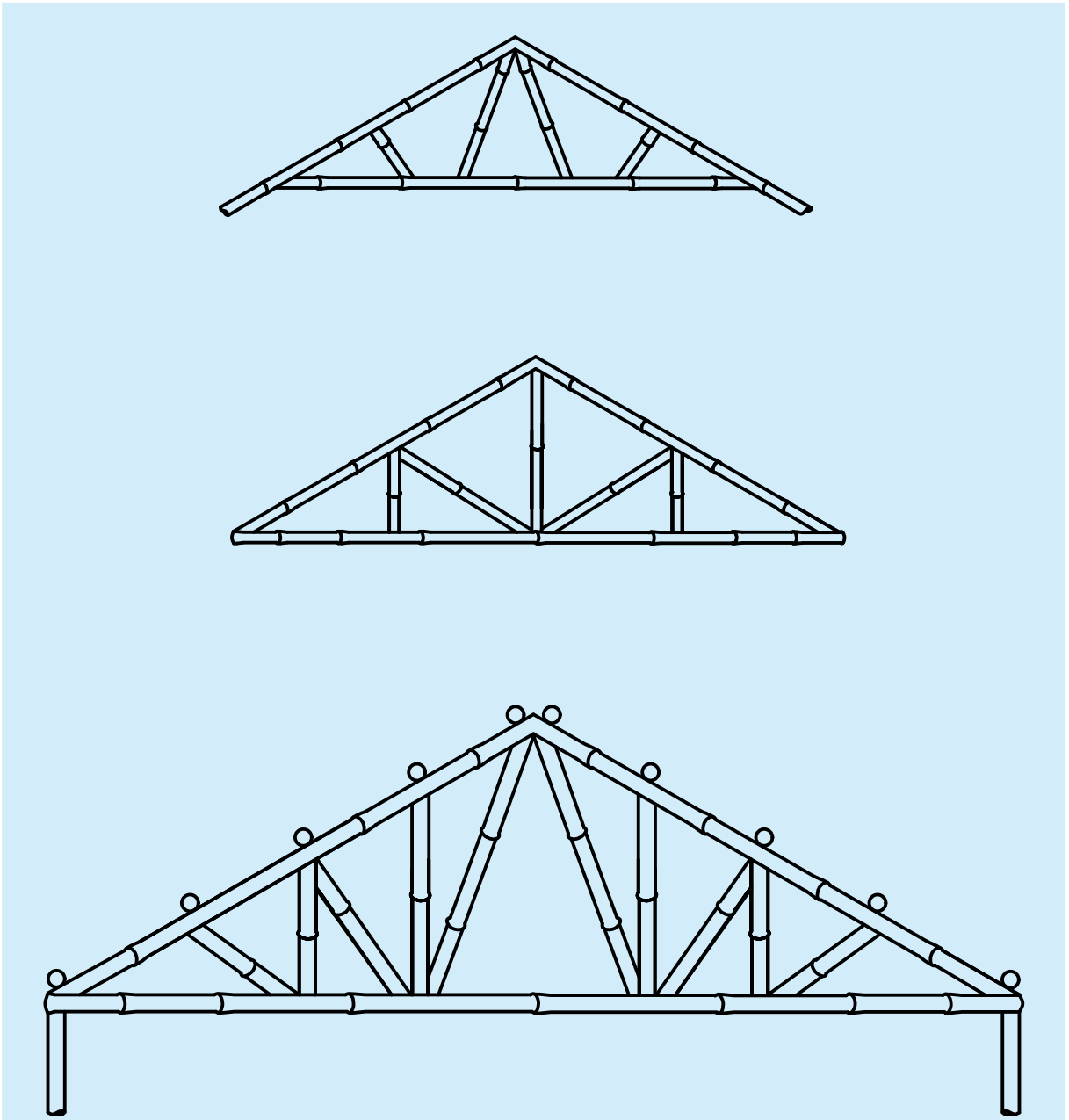
6.9.7 The ends in open beams, joists, rafters, purlins shall be suitably plugged. Bamboo roof coverings shall be considered as non-structural in function.

6.9.8 Bamboo trusses with wire bound joints using bamboo pins and/or wooden splice plates fastened with bamboo pins when tested at load for factor of safety of at least 5, result in deflection within permissible range at design load; good recovery but abnormal deflection beyond ultimate load without collapse was observed.

NOTE — This is based on the work done on prototypes of a few bamboo trusses (span 4 m) tested at Forest Research Institute, Dehradun.



2A PIN ENDED JOINT TRUSSES

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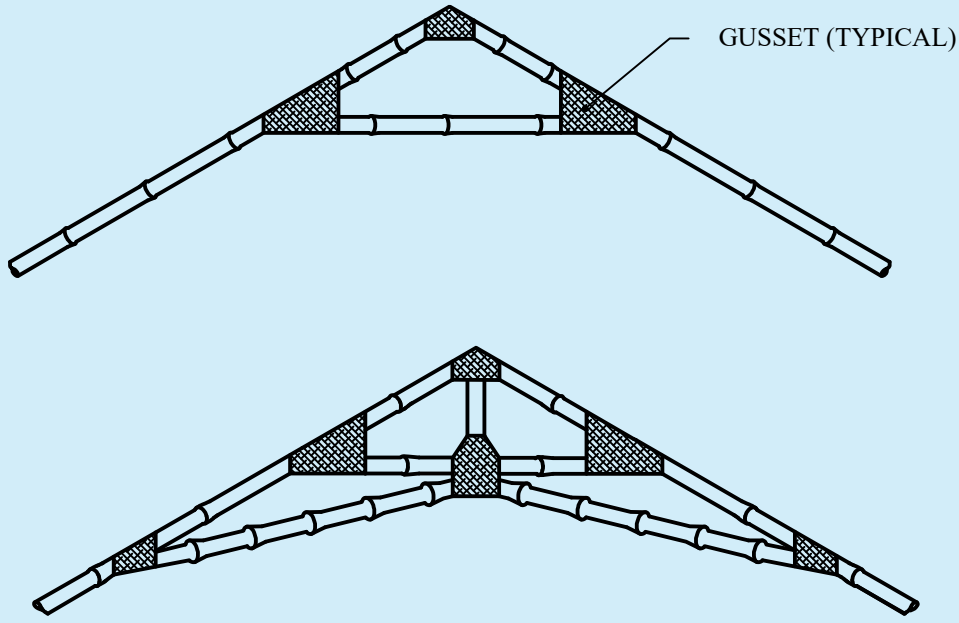

2B GUSSET JOINT TRUSSES

FIG. 2 SOME TYPICAL CONFIGRATIONS FOR SMALL AND LARGE TRUSSES IN BAMBOO

6.9.9 *Bamboo Mat Corrugated Sheets for Building Roofs*

Bamboo mat corrugated sheets (BMCS) shall be used as a sustainable roofing material suitable for residential, commercial, and institutional buildings. BMCS are made from woven bamboo mats treated with phenolic resin and pressed under heat and pressure, these sheets offer a durable and eco-friendly alternative to traditional roofing materials. BMCS sheets shall provide high strength, resilience, and resistance to water, decay, and UV exposure, making them suitable for varied climates. These sheets should contribute to energy efficiency by offering thermal resistance and natural temperature regulation. BMCS shall comply with the accepted standard [C-3B(5)], meeting the necessary performance requirements for load-bearing capacity, impermeability, water absorption, and fire resistance.

6.9.10 *Bamboo Mat Ridge Cap (BMRC)*

Bamboo mat ridge cap (BMRC) is a sustainable, lightweight, and eco-friendly roofing solution developed to complement bamboo mat corrugated sheets (BMCS) in building construction. Produced from bamboo mats, BMRCs are designed to cover the ridge (top edges) of BMCS roofs, providing strong physical and mechanical properties suitable for roofing applications. BMRCs are energy-efficient and cost-effective, offering a high strength-to-weight ratio, low thermal conductivity, and an aesthetically pleasing appearance.

7 DESIGN, TECHNIQUES OF JOINTS AND CONSTRUCTION PRACTICES

7.1 Connecting the load-bearing elements together for effective transfer of stress and to achieve continuity between elements with controlled displacements is one of the serious problem. As joints are a source of weakness in any bamboo structure, they have to be made as strong and rigid as possible. Joints are quite critical in assemblies, and these should be stable in relation to time. For general information, the diagrams of typical joints given in Figs. 3 to 7 may be used based on tests for adequacy of strength.

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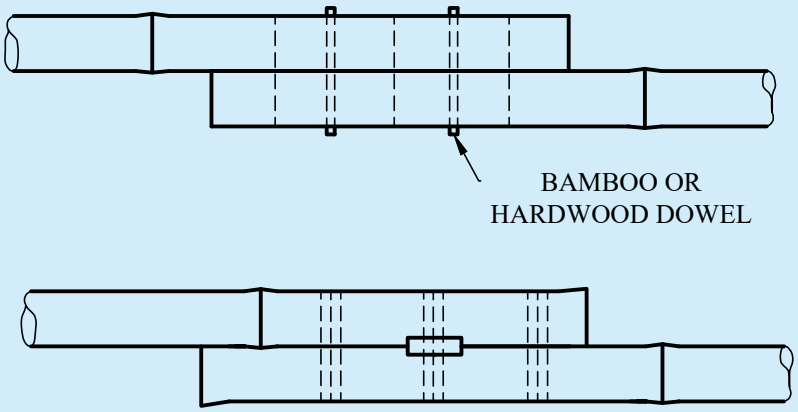
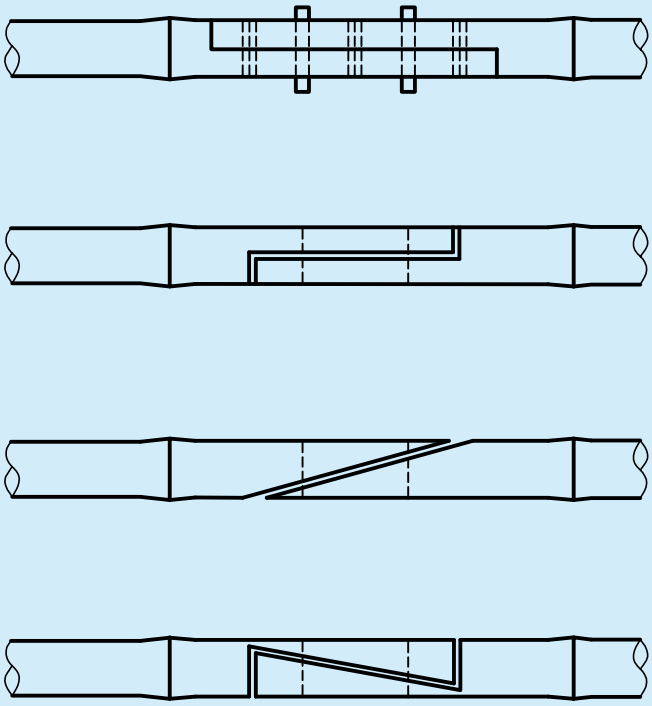


3A FULL LAPPED JOINT



3B HALF LAPPED JOINT
(REBATED/SPLAYED)

FIG. 3 SPLICED JOINT

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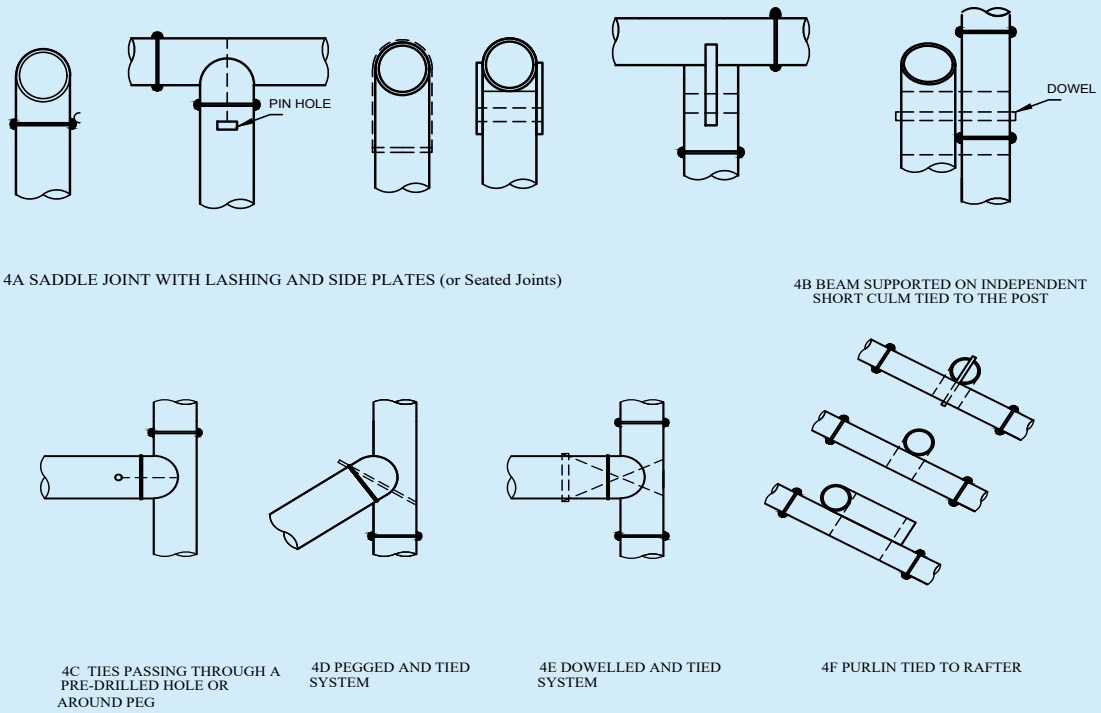

FIG. 4 BEARING JOINTS

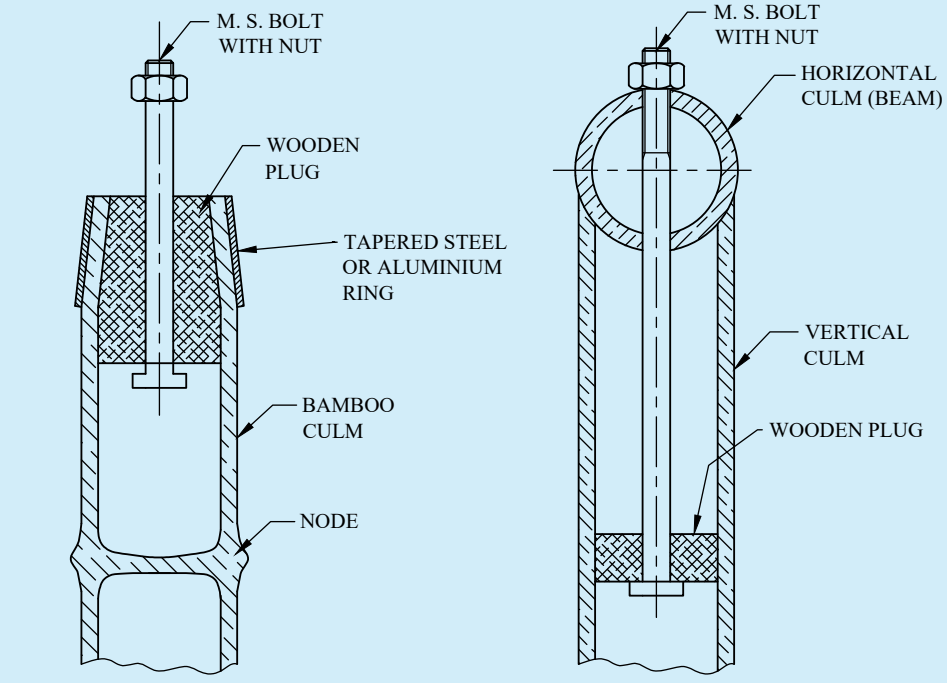

FIG. 5 SOME NUT BOLT JOINERY DETAILS

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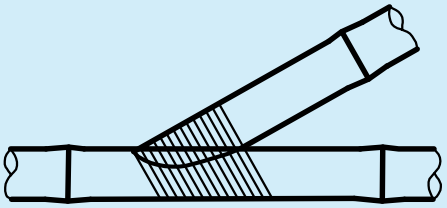
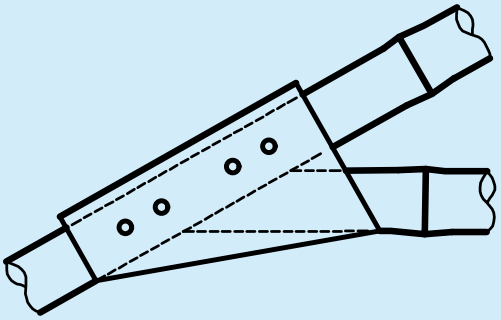
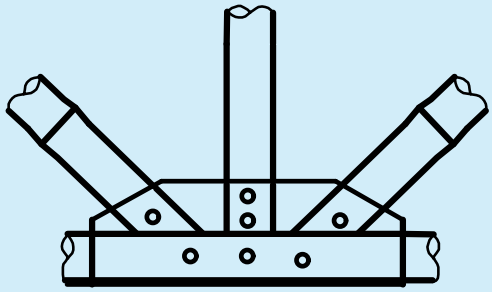
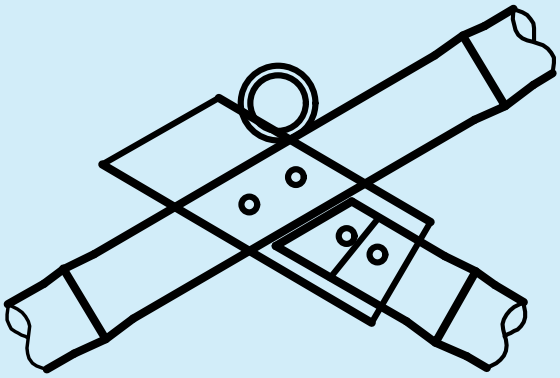


6A LASHED JOINT





6B TRUST CHORD AND WEB CONNECTIONS



6C SOLID TIMBER OR SHUTTERING GRADE PLYWOOD PLATES FIXED WITH EITHER MS BOLTS OR BAMBOO PEGS

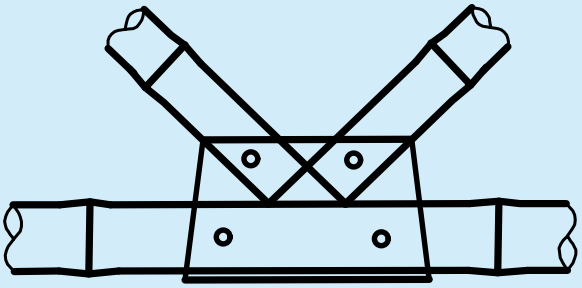


FIG. 6 GUSSET PLATED JOINTS

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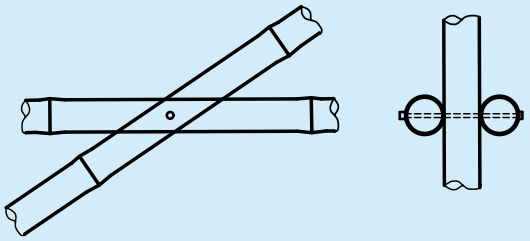
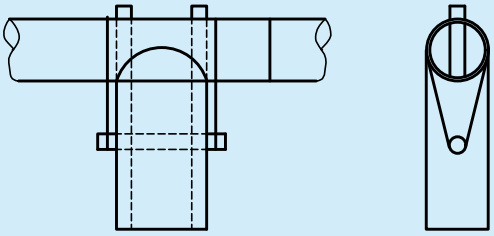
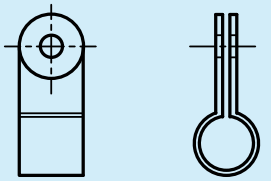


7A SIMPLY BOLTED CONNECTION WITH MS ARC WASHERS

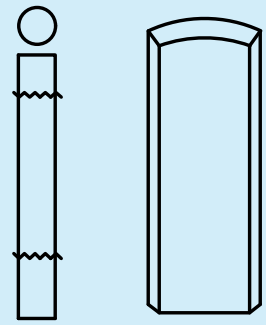


7B HORNED JOINT WITH PIN AND ROPE LASHING



7C STEEL BAND TO BE FITTED AROUND BAMBOO SECTION

7D HARDWOOD DOWEL (SQUARE OR CIRCULAR)



7E SOLID BAMBOO OR SPLIT BAMBOO PINS (WITH SKIN ON)

FIG. 7 OTHER JOINTS AND SECTIONS

7.2 Bamboo Joints

Susceptibility to crushing at the open ends, splitting tendency, variation in diameter, wall thickness and straightness are some of the associated issues which have to be taken care of while designing and detailing the connections with bamboo.

Joints shall be designed to include force transmissions in a required manner, predictable deformations to be within prescribed limits and specific geometry/load direction.

Tests on full scale joints or on components shall be carried out in accordance with the good practice [C-3B(6)] and the accepted standards [C-3B(7)] as for nailed jointed timber structures.

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7.2.1 *Traditional Practices*

Such joining methods revolve around lashing or tying by rope or string with or without pegs or dowels. Such joints lack stiffness and have low efficiency but have served the mankind for long in traditional bamboo construction.

7.2.1.1 *Lengthening joints* (*end jointing*) (*see Fig. 3*)

7.2.1.1.1 *Lap joint*

End of one piece of bamboo is made to lap over that of the other in line and the whole is suitably fastened. It may be full lapping or half lapping. Full section culms are overlapped by at least one internode and tied together in two or three places. Efficiency could be improved by using bamboo or hardwood dowels.

7.2.1.1.2 *Butt joints*

Culms of similar diameter are butted end to end, inter-connected by means of side plates made of quarter-round culm of slightly large diameter bamboo, for two or more internode lengths. Assembly shall be fixed and tied preferably with dowel pins.

7.2.1.1.3 *Sleeves and inserts*

Short length of bamboo of appropriate diameter may be used either externally or internally to join two culms together.

7.2.1.1.4 *Scarf joints*

A scarf joint is formed by cutting a sloping plane 1 in 4 to 1 in 6 on opposite sides from the ends of two similar diameter bamboo culms to be joined. They shall be lapped to form a continuous piece and the assembly suitably fastened by means of lashings or glued.

7.2.1.2 *Bearing joints* (*see Fig. 4*)

For members which either bear against the other or cross each other and transfer the loads at an angle other than parallel to the axis, bearing joints are formed.

7.2.1.2.1 *Saddle joints*

The simplest form consists of a horizontal member supported directly on top of a vertical member. The top of the post may be cut to form a saddle to ensure proper seating of beam for good load transfer. The saddle should be close to a node to reduce risk of splitting.

7.2.1.2.2 *Angled joints*

When two or more members meet or cross other than at right angles, angled joints are formed.

7.2.2 *Improved Practices in Bamboo Jointing*

7.2.2.1 Plywood or solid timber gusset plates may be used at joint assemblies of web and chord connection in a truss and fixed with bamboo pins or mild steel (MS) bolts. Hollow cavities of bamboo need to be stuffed with wooden plugs or otherwise by suitable means.

7.2.2.2 Use of wooden inserts to reinforce the ends of the bamboo before forming the joints. Alternatively, steel bands clamps with integral bolt/eye may be fitted around bamboo sections for jointing.

7.2.3 *Fixing Methods and Fastening Devices*

In case of butt joints the tie-wire may be passed through a pre-drilled hole or around hardwood or bamboo pegs or dowels inserted into preformed holes to act as horns. Pegs are driven from one side, usually at an angle to

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increase strength and dowels pass right through the member, usually at right angles. Normally 1.60 mm (16 SWG) diameter galvanized iron wire may be used for tight lashing.

NOTE — Drill size needs to be specified in order to avoid possible split/crack in walling of bamboo.

7.2.3.1 *Wire bound joints with or without pins*

Usually 2.00 mm (14 SWG) diameter galvanized iron wire is tightened around the joints by binding the respective pieces together. At least two holes are drilled in each piece and wire is passed through them for good results. Generally, 10 mm to 12 mm diameter bamboo pins are driven and fastened to culms.

7.2.3.2 *Fish plates/Gusset plated joints* (*see Fig.* 6)

At least 25 mm thick hardwood splice plate or 12 mm thick structural grade plywood are used. Solid bamboo pins help in fastening the assembly.

7.2.4 For any complete joint alternative for a given load and geometry, description of all fastening-elements, their sizes and location shall be indicated. Data shall be based on full scale tests.

7.2.5 Tests on full scale joints or on components shall be carried out in a recognized laboratory.

7.2.6 In case of high wind and seismic areas, good construction practices shall be followed taking care of joints, their damping and possible ductility. Bracings in walls shall be taken care of in bamboo structures.

7.2.7 Metal fasteners and other structural joints shall be made inherently corrosion resistant or be protected against corrosion, thereby making the joints more durable.

7.3 Construction Practices

7.3.1 Bamboo being a versatile resource characterized by high strength, low mass and ease of working with simple tools, it is desirable to increasingly make appropriate use of this material. Design of structures using bamboo should be done in accordance with **6**. It should be ensured that the elements of structure satisfy the appropriate fire resistance requirements as specified in Part F 'Fire and Life Safety' of this NBCS, and quality of building materials/components used shall be in accordance with Part B 'Building Materials' of this NBCS. For construction using bamboo, some of the important constructional provisions given in **7.3.2** to **7.3.7** shall be followed.

7.3.2 The following sound construction practices shall be taken into account:

a) Use of air dry bamboo; and ensure that bamboo in building remains air dry; and in case the bamboo has become wet, shall provide it the opportunity to dry again before the it deteriorates due to moisture;

b) Permeability of walls, floors and roofs made from bamboo, cause internal pressure and can therefore change the net wind-load acting on the roof, wall and floor; and

c) Check whether the workmanship of the labour force is according to the assumptions.

7.3.3 *Working Finishing*

7.3.3.1 Bamboo can be cut and split easily with very simple hand tools. Immature bamboos are soft, pliable and can be moulded to desired shape. It takes polish and paint well.

7.3.3.2 While it is possible to work with bamboo simply using a machete, a few basic tools such as, machete, hack saw, axe, hatchet, sharpening tools, adze, chisel (20 mm), drill, wood rasps, steel rod, and pliers, will greatly increase the effectiveness of the construction process.

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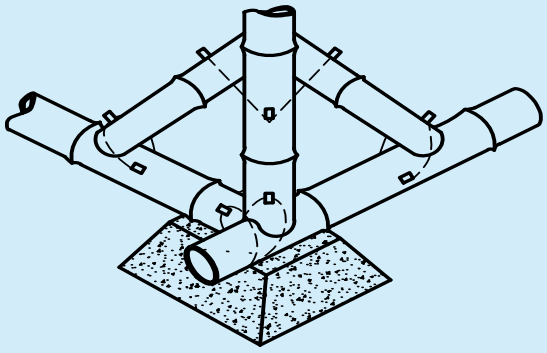
7.3.4 For providing safety to the structure against fire, bamboo may be given fire retardant treatment using following chemicals; a few drops of concentrated Hcl shall be added to the solution to dissolve the precipitated salts:

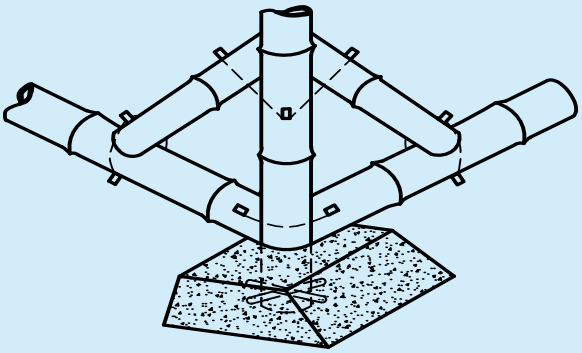
Ammonium phosphate	:	3 parts
Boric acid	:	3 parts
Copper sulphate	:	1 part
Zinc chloride	:	5 parts
Sodium dichromate	:	3 parts
Water	:	100 parts

7.3.5 *Foundations*

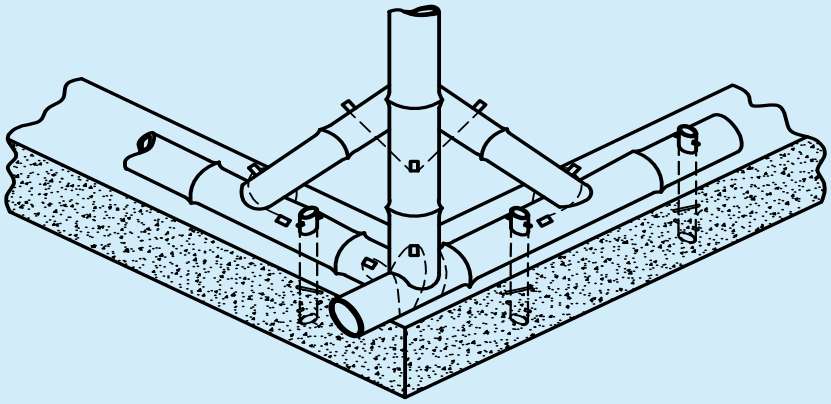
Bamboo in direct contact with ground, bamboo on rock or preformed concrete footing, bamboo incorporated into concrete, may form the foundation structure (*see* Fig. 8). Also, arrangements with use of metallic strap, C-clamp and bunched bamboo columns embedded in concrete may be used for foundation of bamboo structures (*see* Fig. 9).



8A BAMBOO ON PREFORMED CONCRETE FOOTINGS



8B BAMBOO INCORPORATED INTO CONCRETE FOOTINGS (SINGLE POST FOOTING)



8C BAMBOO OUT OF GROUND CONTACT ON STRIP FOOTINGS OF CONCRETE (LARGE DIAMETER THICK WALLED BAMBOO WITH CLOSELY SPACED NODES TO BE USED)

FIG. 8 BAMBOO FOUNDATIONS

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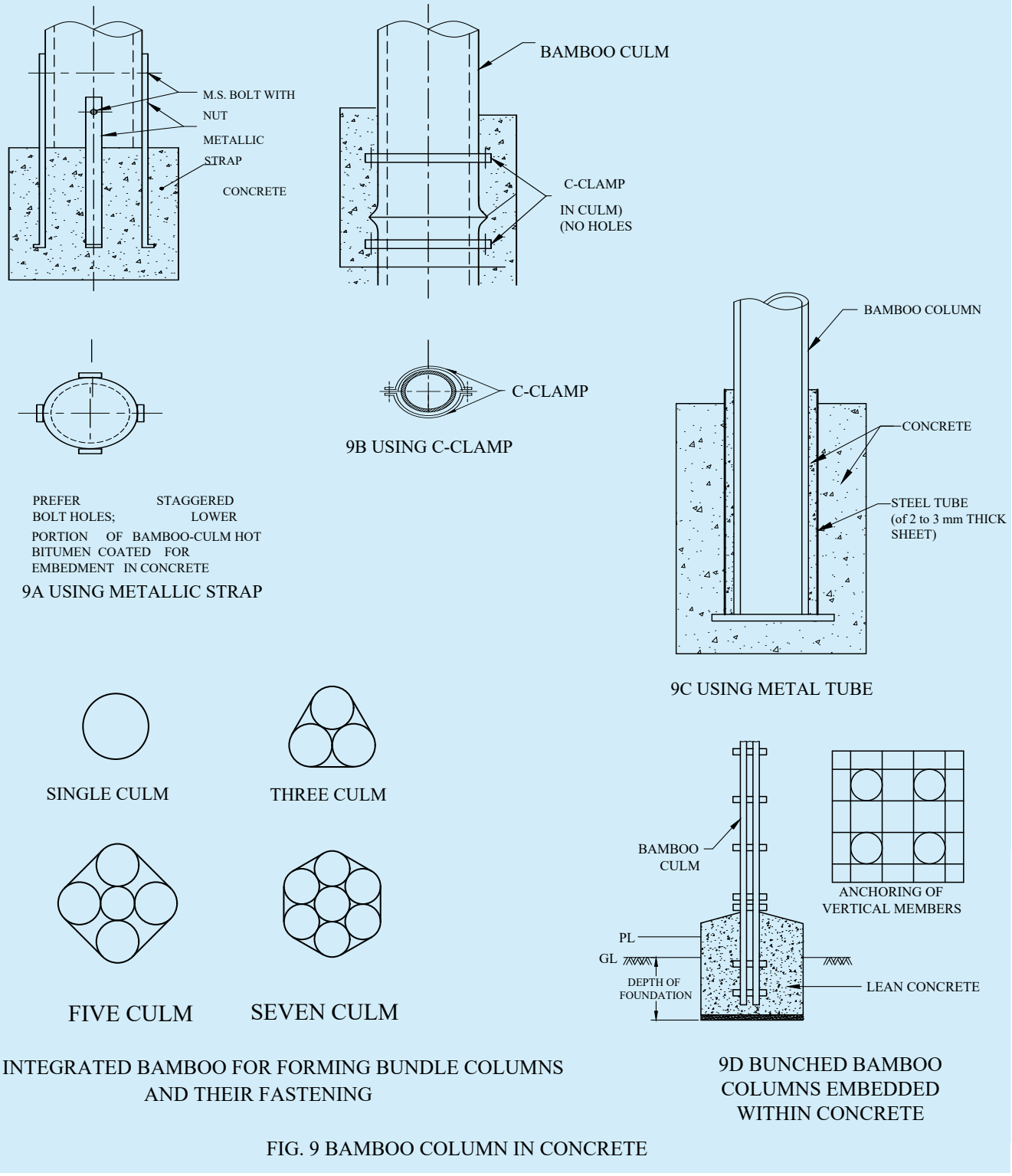

FIG. 9 BAMBOO COLUMN IN CONCRETE

7.3.6 *Floors*

The floor of bamboo may be at ground level with covering of bamboo matting, etc. In elevated floors, bamboo members become an integral part of structural framework of building. The floor will comprise structural bamboo elements and bamboo decking.

7.3.7 *Jointing Techniques*

The jointing techniques in construction using bamboo shall be in accordance with 7.2.

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8 BAMBOO AS REINFORCEMENT IN CONCRETE

8.1 Due to its fibrous texture, bamboo possesses high tensile strength. This can act as an alternate material for reinforcement in concrete within the limitations in design and construction. The ultimate strength of some of the species of bamboo in direct tension is nearly the same as that of steel at its yield point. On an average it varies from 1 400 kg/cm^{2}to 2 000 kg/cm^2. The principles adopted for design of concrete structure with steel is also applicable for the concrete member with bamboo. Strength characteristics of steel, concrete and bamboo are indicated in Table 4 and the design data for balanced section in beams/slabs reinforcement is given in Table 5.

Table 4 Strength Characteristics of Mild Steel, Bamboo and Concrete (For Reinforcement Purposes)

(*Clause* 8.1)

Sl No.	Species	Strength Characteristics kgf/cm^2		
		Mild Steel	Bamboo (Green)	Mass Concrete
(1)	(2)	(3)	(4)	(5)
i)	Modulus of elasticity	2.1× 10^6	175 750 to 196 640 (for all locations)	–
ii)	Tensile stress	1 265.5 to 1 400	158, Bending-tension (all locations)	–
iii)	Compressive stress	–	105 (*D. strictus*)	500
iv)	Bond stress	–	3.5 (green) to 5.6 (dry) reported (likely to increase up to 10)	40 to 100
v)	Shear stress	–	115 to 180	50

Table 5 Design Data for Bamboo and Mild Steel Reinforced Cement Concrete for Balanced Section

(*Clause* 8.1)

Sl No.	Design Parameter	Design Data for Balanced Section M 15 (1 : 2 : 4) Bamboo and Mild Steel Reinforced Cement	
		Mild Steel	Bamboo
(1)	(2)	(3)	(4)
i)	Modular ratio	18	1 to 1.4
ii)	Position of neutral axis from the top	0.4d	0.25d to 0.254d
iii)	Lever arm of the resisting moment (MOR)	0.87d	0.917d to 0.915d
iv)	Constant for MOR (Q)	137	86
	where Q= Bending moment/bd^2 , d=depth or thickness, and b= width.		

8.2 The whole bamboo culms are halved longitudinally and again splitted into four and strips of the size of 20 mm to 25 mm width and 9 mm thickness are prepared for use as reinforcement. Bamboo strips with triangular lugs (formed at nodes) due to conversion of bamboo into strips improve the bond strength because of the protruding

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lugs beings gripped firmly in concrete. Strength properties of split bamboo out of a few species are given in Table 6.

8.3 Bamboo has a tendency to absorb water from concrete which results in initial swelling and subsequent shrinkage leading to cracks in concrete. Following processes are thus recommended as mandatory:

a) Chemical treatment with preference for pressure impregnation (Full cell treatment with warm coal tar creosote is likely to preserve and at the same time act as water repellent); and

b) Treated strips be further given a water repellent treatment with a coating of very hot bitumen 80/100 grade, followed by a liberal coating of coarse sand. Water inhibitor will also help maintain dimensional stability of strips.

8.4 Techniques in Placing the Reinforcement (*see also* Fig. 10)

Reinforcement shall be placed in the following manner:

a) Laid edge wise in sets of twin strips for main reinforcement;

b) Laid flatwise with smooth surface and rough surface alternating as distribution or subsidiary reinforcement (wedge effects increased bond);

c) For negative moments, bamboo strips of length equivalent to $\frac{span}{6}$ laid flatwise in alternate rows of slab reinforcement;

d) For beams and lintels, twin strips are arranged in tiers, placed one above the other depending upon requirement and held in position with the help of vertical strips;

e) Split bamboo strip or preferably steel rod be used for diagonal tension to resist shear in beams, etc; and

f) Bind the reinforcement with the help of galvanized iron binding wire (or nylon thread/bamboo-cane string).

Table 6 Strength Properties of Split Bamboo in Green Condition[1)]

(*Clause* 8.2)

Sl No.	Species	Static Bending			Compression Parallel (Without Node)	Shear Parallel
		Fibre Stress at Elastic Limit	Modulus of Rupture	Modulus of Elasticity	Maximum Crushing Stress	Maximum Shearing Stress
		kg/cm^2	kg/cm^2	$\times 10^3$ kg/cm^2	kg/cm^2	kg/cm^2
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	*Bambusa pallida*[2)]	378	605	32.5	595	97.7
ii)	*B. vulgaris*	464	885	52.3	403	94.4
iii)	*Dendrocalamushamiltonii*	301	524	37.7	414	77.7
iv)	*D. giganteus*	472	848	50.7	360	88.5
v)	*Oxytenanthera abyssinica*[2)]	450	831	33.6	377	112.0

[1)] average of bottom, middle and top portions of culms.

[2)] average of bottom and middle portions only.

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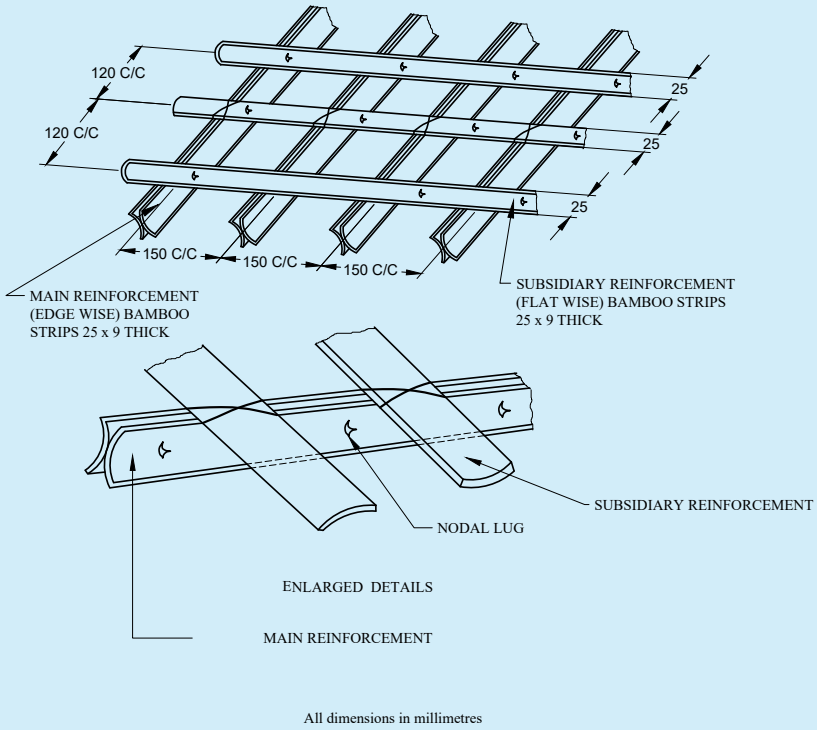

FIG. 10 GENERAL DETAILS OF BAMBOO REINFORCEMENT

9 WALLING, FLOORING, ROOFING, AND IN-FILL PANELS FOR WALL, AND COMMON ROOF COVERING/CLADDING MATERIALS

9.1 Materials

9.1.1 *Bamboo Based Panel Products*

The common roof covering and in-fill materials shall include bamboo mat board, bamboo mat veneer composite, bamboo mat corrugated sheet, bamboo tiles, plastered bamboo reeds, thatch, corrugated galvanized iron sheeting, asphaltic sheets, plain clay tiles, etc. The respective materials shall conform to the following:

a) *Bamboo mat board* — These shall conform to the accepted standard [C-3B(8)];

b) *Bamboo mat veneer composite* — These shall conform to the accepted standard [C-3B(9)];

c) *Bamboo mat corrugated sheet and bamboo mat ridge cap* — These shall conform to the accepted standard [C-3B(5)]; and

d) *Bamboo-jute composite corrugated and semi-corrugated sheets* — These shall conform to the accepted standard [C-3B(10)].

9.1.2 *Reconstituted Wood from Destructured Bamboo*

After lopping, bamboos are cross-cut into small lengths in the round and then split along the axis. The small cross-section strips are then passed between the counter revolving rollers till the material is fully destructured. This is then air dried and resin treated and suitably pressed without much disturbing the natural alignment of fibres thus retaining the directional properties of bamboo.

Some typical strength properties of reconstituted wood from destructured bamboo are given in Table 7. The reconstituted product from bamboo can be sawn, nailed, moulded, bored, screwed, polished and painted. Thus, it offers a good substitute of solid wood for structural uses.

NOTE — Reconstituted boards/wood from destructured bamboo when tested for physical and mechanical properties in accordance with the accepted standards [C-3B(11)] and [C-3B(12)] yielded encouraging results. Gist of mechanical properties of these boards is excellent when compared with *Dendrocalamus strictus* (bamboo), teak and sal.

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Table 7 Gist of Mechanical Properties of Reconstituted Wood from Bamboo (*Dendrocalamus strictus*)

(*Clause* 9.1.2)

Sl No.	Board Material			Density	Modulus of Rupture	Tensile Strength Parallel to Grain	Compressive Strength Parallel to Grain	Izod Impact	Shear Strength	Modulus of Elasticity	Screw Holding Power
	Set	Resin Content	Pressure								
		percent	kgf/cm^2	gm/cm^3	kgf/cm^2	kgf/cm^2	kgf/cm^2	kgf/cm	kgf/cm^2	$\times 10^3$ kgf/cm^2	kgf
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
i)	I	6 to 29	56	0.99 to 1.25	1 769 to 2 255	1 049 to 2 024	1 057 to 1 596	343 to 281	202 to 364	278 to 308	398 to 508
					In the increasing order of resin percentage.						
ii)	II	9	21 to 28	0.62 to 0.93	1 062 to 1 615	6 99 to 1 210	675 to 1 030	186 to 202	128 to 247	155 to 232	361 to 486
	Wood/Bamboo										
iii)	*Shorea robusta* (at 12 percent, dry moisture content)			0.749	1 249	–	685	124	110	152.8	369
iv)	*Tectona grandis* (at 12 percent, dry moisture content)			0.596	959	–	532	68	102	119.6	326
v)	*Dendrocalamus strictus*			0.757	1 310	–	624	–	–	180.3	–

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9.1.3 *Other Panel Products/Infill Materials*

Other panel products/infill materials shall be in accordance with relevant Indian standards. These shall be such that they maintain their integrity and strength in the assigned service class throughout the expected life of the structure.

9.1.4 *Adhesives*

Adhesive shall maintain their integrity in accordance with the accepted standard [C-3B(13)].

9.2 Structural Usage of Bamboo Based Products

9.2.1 *General*

Bamboo mat board (BMB) and bamboo mat veneer composite (BMVC) have high in-plane rigidity and hence high racking strength and are more flexible than equivalent plywood. This property of BMB and BMVC can be advantageously used in many engineering applications such as wall panelling, composite beams, composite floor deck and stress skinned panels. Bamboo mat corrugated sheet (BMCS) is water proof and resistant to decay, termites/insects and fire. These sheets may be used as roofing material. Precast bamboo reinforced cement concrete and reconstituted wood from destructured bamboo can also be used in structural applications.

9.2.2 *Bamboo Composites*

Along with bamboo, bamboo composites such as bamboo mat board, bamboo mat veneer composite, may also be successfully used in combination with timber for making efficient structural components like web because where high shear strength of bamboo mat board and bamboo mat veneer composite can be utilized as web and timber flanges for transmitting tensile and compressive stress. PF, PRF adhesives are commonly used for gluing web and flanges as shown in Fig. 11. Beams can be straight, single tapered, double tapered, haunched, as shown.

Bamboo mat board and bamboo mat veneer composite can also be used for roof and wall sheathing (shear walls) and diaphragms, gussets, arch panels, barrel vaults, etc. It can also be used as skin material in sandwich construction.

Permissible safe working stresses are given in Table 8.

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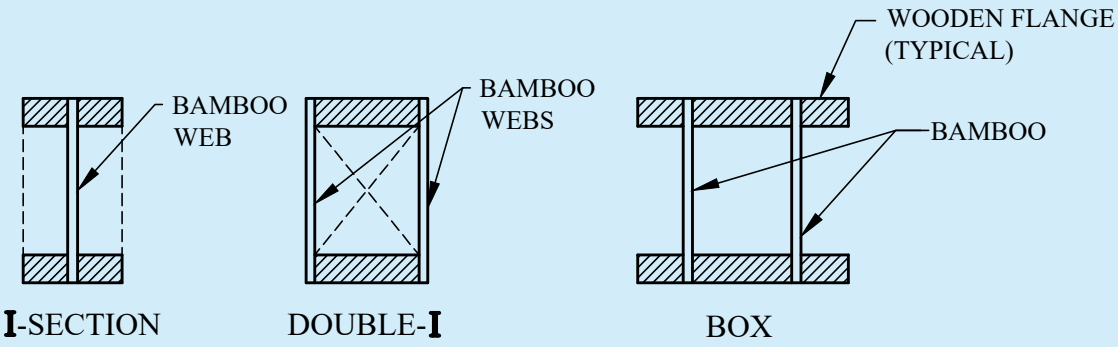
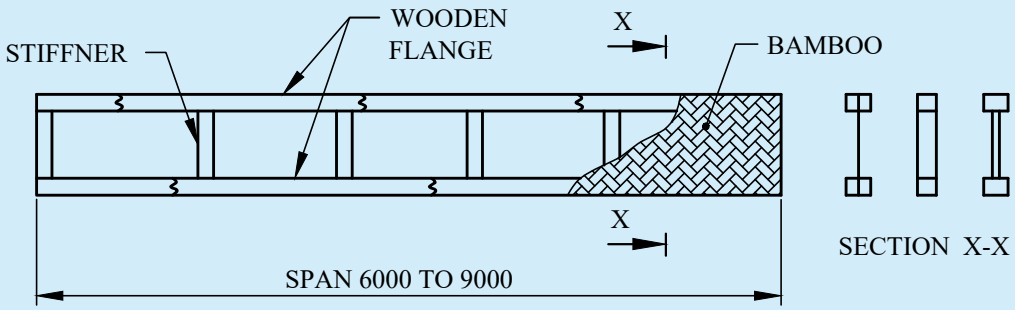
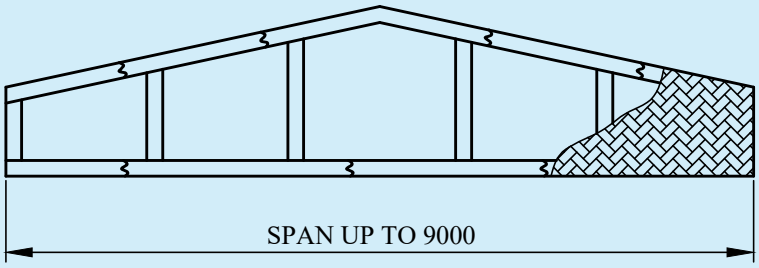


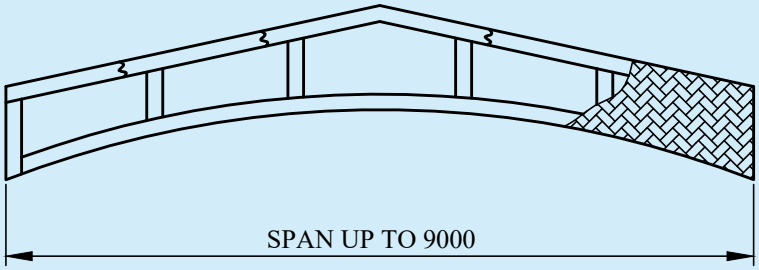
TYPICAL CROSS SECTION OF WEB BEAMS



STRAIGHT



DOUBLE TAPERED



HAUNCHED

WEB BEAM CONFIGURATIONS

All dimensions in millimetres

FIG. 11 BAMBOO MAT BOARD WEB BEAMS

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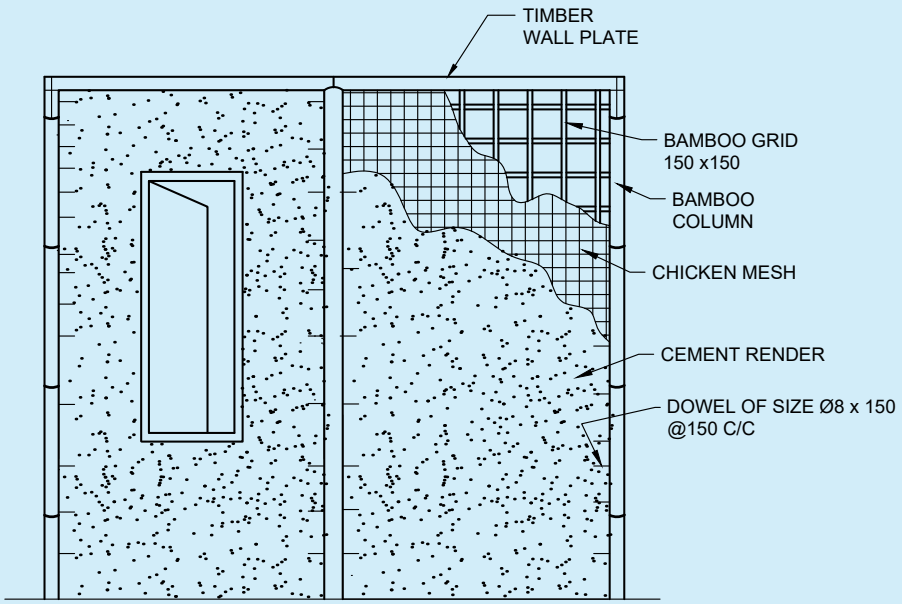
Table 8 Permissible Safe Working Stresses for Bamboo Mat Board and Bamboo Mat Veneer Composite

(*Clause* 9.2.2)

Sl No.	Property	Bamboo Mat Board	Bamboo Mat Veneer Composite in Direction	
			Along	Across
		N/mm^2	N/mm^2	N/mm^2
(1)	(2)	(3)	(4)	(5)
i)	Tensile strength	8.5	10	10
ii)	Compressive strength	10.80	13	6.5
iii)	Modulus of rupture	19.00	19	10.5
iv)	Modulus of elasticity	3 200	4 600	–
v)	Modulus of rigidity	4 970	1 930	–



All dimensions in millimetres.

FIG. 12 BAMBOO STRUCTURAL WALLING SYSTEM

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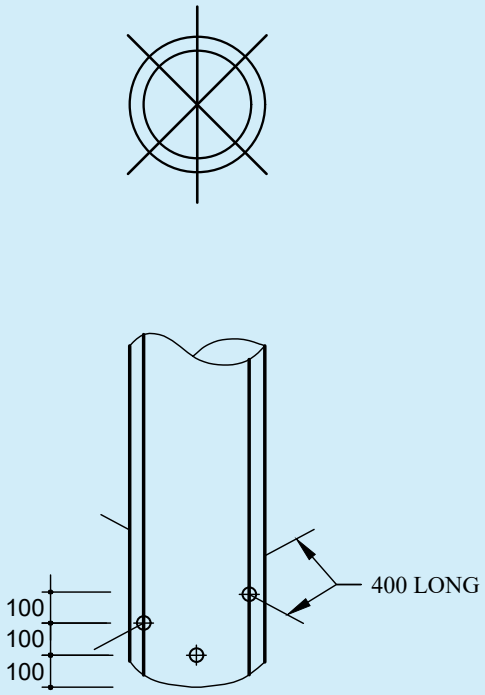
9.2.4.1 *Foundation detail*

The columns pierced by steel dowels at 150 mm centre-to-centre provide connection to the bamboo grid infill. Fix 8 mm diameter MS rod of 400 mm length at 100 mm from one end of the column which has undergone treatment. For anchoring, fix two more MS rods of same diameter at 100 mm centre-to-centre starting from the first hole (*see* Fig. 13). Pour the concrete by giving temporary support. Foundation is limited to individual footings of size 400 mm × 400 mm × 600 mm deep for each bamboo column placed at 1 200 mm apart and two coarse concrete blocks or stone or other locally available material for infill wall foundation.



All dimensions in millimetres

FIG. 13 DETAILS OF BOTTOM OF BAMBOO COLUMN FOR ANCHORING IN CONCRETE FOR FOUNDATION

9.2.5 *Bamboo Concrete Flooring Using Round Bamboo*

Technology of such a flooring (for situations needing raised flooring) is based on the method of rigidly fixing round bamboo, where 30 mm thick M20 grade PCC (with 6 mm down coarse aggregate) is placed over 6 mm to 8 mm thick culm, that are arranged side by side to form bamboo flooring.

The rigid fixing is achieved by using shear connectors in the form of 1.5 mm dia. 20 mm long MS nails partially inserted into the bamboo flooring and balance length with the heads of the nails embedded in the PCC layer, holding the two components together. The sectional properties of the combination are calculated based on the concept of flitched beams for design (*see* Fig. 14).

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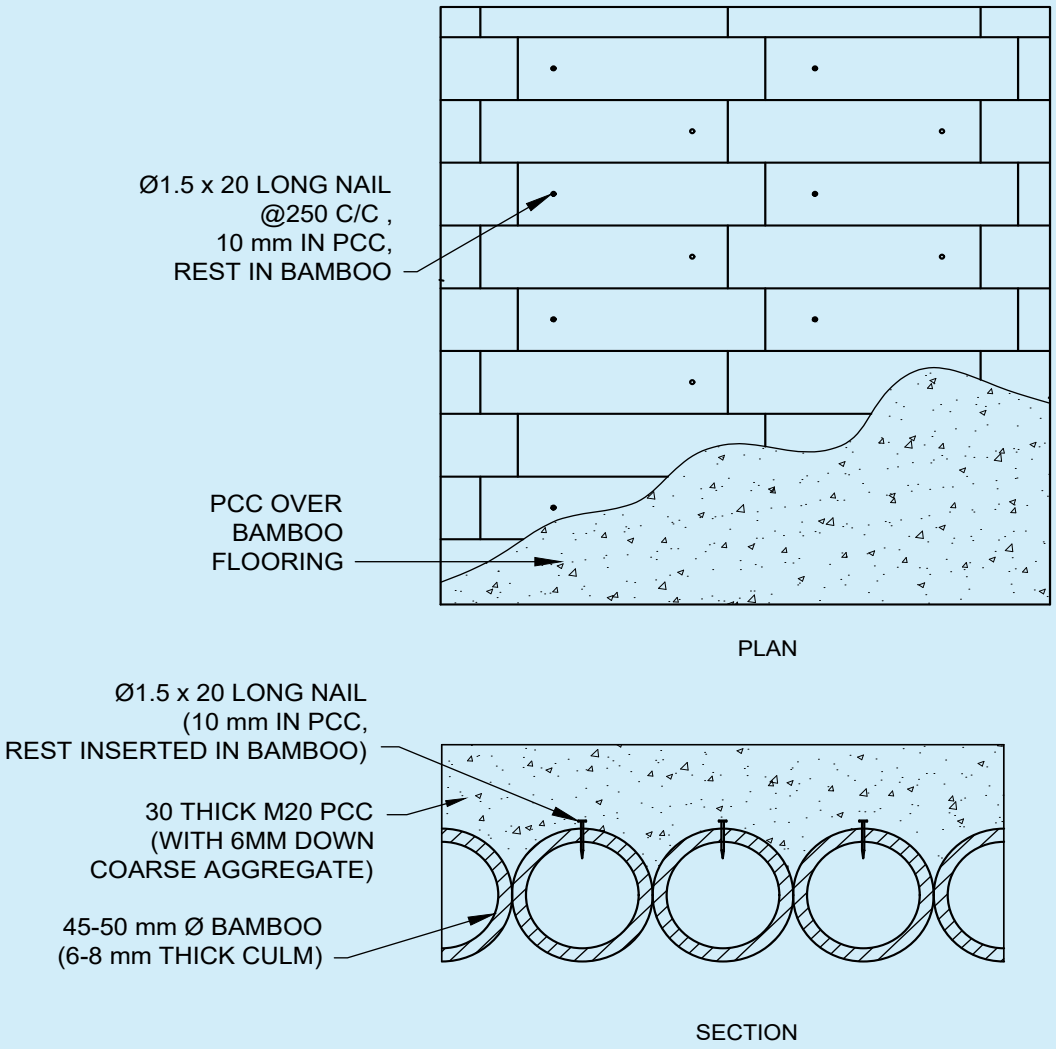

All dimensions in millimetres.

FIG. 14 DETAIL OF A TYPICAL BAMBOO CONCRETE FLOORING

9.2.6 *Bamboo Concrete Walling Using Flat Bamboo*

Loads on walls and roof do not generally justify use of round bamboo as such. So, flat or split bamboo is preferred for use in these cases. The flat bamboo works like a wire-net used conventionally to hold cement plaster in critical applications. Partial plastering is done during the first day ensuring entry of plastering material in the gaps of flat bamboo. This provides the base for cement plastering or PCC laying next day. About 5 mm gap is essential for entry of plastering material of sufficient sectional area to sustain shear force. The gaps are generated by inserting a small wedge of bamboo strip and pulling it out after a part of the gap is filled by plaster. Such gaps have to be discontinuous, to retain the character of a wire-net. About 25 percent width of the flat bamboo is assumed to be gaps, part of it is the organized gap of 5 mm width as described, at an interval of about 50 mm width, to sustain shear force and the balance gap remains to be natural to flat bamboo. Minimum thickness of the flat bamboo is assumed to be 6 mm. Bamboo with thicker wall would be stronger.

Expected strength (using *Bambusa balcooa*) may be considered as follows:

a) Walling of 6 mm bamboo with 18 mm thick cement sand plaster in 1 : 3 would be able to withstand wind pressure of 112 kgf/m^2 if used as wall, either supported continuously 1.5 m apart, or supported simply 1.25 m apart; and

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b) As a roof, the wind suction load would be partly compensated by the dead load of the roof. The imposed load of a mason sitting at the centre of span needs to be considered. This load can be accommodated by use of 18 mm M 20 PCC with 6 mm flat bamboo, supported 1.0 m apart; neatly finished.

The engineered houses for application of the above bamboo concrete walling are designed with stiffeners and diagonals in all the three planes, keeping the deformation of walls along plane of walls almost nil and deflection of columns across the plane of wall at a minimum.

10 LIFE CYCLE ASSESSMENT OF BAMBOO FOR USE IN BUILDINGS

10.1 Timber used in building structures and elements shall undergo a comprehensive life cycle assessment (LCA) to evaluate its environmental impacts across its entire lifecycle, from raw material extraction to disposal or recycling. LCA provides valuable insights into the sustainability of timber and its environmental footprint in building applications. The assessment shall cover the following stages:

a) *Production and Construction* — This stage evaluates the energy and resources required for the extraction, transportation, and processing of raw timber into building materials. It also includes the energy consumption and waste generated during the construction process, as well as the transportation of materials to the construction site.

b) *Use Stage* — The use stage assesses the environmental impacts of timber during the building's operational lifespan. This includes maintenance, repairs, and potential replacement of timber components, considering the durability, performance, and long-term sustainability of the material.

c) *End-of-Life* — At the end of the building's useful life, the timber structure may undergo demolition, with the materials being processed for waste management, recycling, or reuse. This stage assesses the environmental effects of these processes and the potential for timber materials to be repurposed.

d) *Externalized Impacts Beyond the System Boundary* — The incidental effects of recycling, reusing, and recovering energy, water, or materials beyond the system boundaries of the LCA study are included in this stage. These externalized impacts consider how recycled timber or materials might contribute to environmental benefits outside the building's lifecycle.

10.2 Environmental Impact Categories

The LCA may be used to assess the environmental impact of timber buildings in several categories, which encompass emissions and pollutants generated throughout the lifecycle. These categories include:

a) *Global Warming Potential* — The contribution of greenhouse gas emissions to climate change;

b) *Acidification Potential* — The potential for acidic compounds to form in the environment, affecting ecosystems and infrastructure;

c) *Eutrophication Potential* — The impact of nutrient loading on water bodies, potentially causing excessive algal growth;

d) *Ozone Depletion Potential* — The impact of substances that contribute to the depletion of the ozone layer.

e) *Smog Formation Potential* — The potential for air pollution to contribute to the formation of ground-level ozone and smog; and

All relevant material and process quantities shall be included in the LCA inventory, and their impacts shall be calculated using recognized tools and methodologies.

10.3 Requirements for Life Cycle Assessment

In the context of building with timber, LCA may be required for the following purposes:

a) *Decision-Making Support* — Providing building owners and stakeholders with data on the environmental and sustainability aspects of timber, aiding informed decision-making;

b) *Design Evaluation* — Offering insights into material selection and environmental performance across different design possibilities;

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c) *Product Comparison* — Demonstrating the environmental advantages of timber in comparison to alternative building materials;

d) *Environmental Benefit Evaluation* — Helping assess the ecological benefits of innovative timber products or policies; and

e) *Performance Benchmarking* — Comparing the environmental performance of timber buildings against established benchmarks and sustainability goals.

10.4 Methodology for Conducting LCA

The LCA of timber buildings may be conducted iteratively, with increasing detail as the design or construction phases progress. The process should follow these steps:

a) Establish the purpose and scope of the LCA, focusing on the objectives of evaluating timber's environmental impact;

b) Gather data on the timber materials, including their types, quantities, lifespans, and lifecycle stages. Special attention should be given to materials used in the production and use stages;

c) Using LCA tools (for example, SimaPro, MATLAB), assess the environmental impacts of timber buildings. The evaluation shall include biogenic carbon considerations, as timber stores carbon from the atmosphere, contributing to carbon sequestration during its lifecycle;

d) Interpret the LCA results by categorizing environmental impacts by building component, material type, and lifecycle stage. Identify key contributors to the building's overall environmental footprint and visualize the outcomes; and

e) Document all findings, including assumptions, methodology, and conclusions, and present the results in reports for stakeholders. The results shall guide sustainable construction practices and decision-making in timber building projects.

10.5 Reporting

The results of the LCA should be documented, including all assumptions, methodologies, and conclusions. Reports should be prepared in accordance with relevant standards, and may be used to guide sustainable building practices and decision-making in construction.

11 TYPICAL DETAILS OF CONVENTIONAL BAMBOO BUILDING CONSTRUCTION

Typical details of conventional bamboo building construction is given in Annex C, for guidance.

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ANNEX A

(*Foreword*)

SOURCE AND LOCAL NAMES OF SOME OF THE SPECIES OF BAMBOO

Sl No.	*Species*	*Source/Local Names*
(1)	(2)	(3)
i)	*Bambusa auriculata*	Assam, Bangladesh, Myanmar; introduced in Calcutta Botanical Garden.
ii)	*B. balcooa*	Asm – Baluka; Ben – Balku bans; Duars – Bora bans; Garo – Wamnah, beru; Tripura – Barak.
iii)	*B. bambos* (*Syn.B. arundinacea*)	Asm – Kotoba; Ben – Baroowa, behor, ketuas, ketwa; Manip – Saneibi; Mah – Katang bamboo, oowga; Oriya – Daba, katuig; Tel – Mulkasveduru, Mulluvedurn; English – Spiny bamboo.
iv)	*B. burmanica*	Asm – Thaikawa.
v)	*B. multiplex* *Syn. B. glancescens* (*Syn. B. nana*)	Sans – keu-fa; Burmese – Pa-lau-pinan-wa; Malay – Bamboo tjeenah; China – Bamboo hower tjeenah.
vi)	*B. nutans*	Asm – Deobans, jotia-makal; Asm – Bidhuli, mukial; Ben – Makia; Bhutia – Jiu; Hin – Malabans; Kangra – Nal; Khasi - Seringjai; Kuki – Wa malang; Lepcha – Malubans, mahlu, mallo; Oriya – Badia bansa; Sylhet (Bangladesh) – Peechli; Tripura – Kali.
vii)	*B. pallida*	Asm – Bijli, jowa, makal, walkthai; Cachar – Bakhal, burwal; Khasi – Seskien, skhen, ineng, usker;

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Sl No. (1)	*Species* (2)	*Source/Local Names* (3)
		Lepcha – Pashipo, pshi, pushee;
		Mikir – Loto;
		Naga – Tesero, watoi;
		Tripura – Makal.
viii)	*B. polymorpha*	Asm – Jama betwa, betwa;
		Ben – Batua, jaibarouwa, jama;
		Burma – Kyathaung-wa;
		MP-Korku – Narangi bhas;
		Tripura – Basi.
ix)	*B. tulda*	Asm – Wamunna, wagi, nal-bans;
		Ben – Tulda, jowa;
		Duars – Karanti, matela;
		Garo – Watti;
		Hin – Peka;
		Kamrup – Bijuli, jati, joo, ghora;
		Tripura – Mirtinga.
x)	*B. vulgaris*	Ben and Manipuri – Bakal;
		Oriya – Sunarkania bans.
xi)	*B. wamin Syn. B.ventricosa* (*Syn. B. vulgaris var. wamin*)	Common name – Pitcher bamboo.
xii)	*Cephalostachyum pergracile*	MP – Bhalan bans;
		Manip – Wootang;
		Naga – Latang;
		Oriya – Darrgi.
xiii)	*Dendrocalamus giganteous*	English – Giant Bamboo;
		Asm – Worra;
		Manip – Maroobeb.
xiv)	*D. hamiltonii*	Nep – Tamo;
		Asm – Kokwa;
		Tripura – Pecha.
xv)	*D. Longispathus*	Tripura – Rupai.
xvi)	*D. membranaceus*	Native of Myanmar; introduced in Kerala.
xvii)	*D. strictus*	English – Male bamboo;
		Ben – Karail;
		Guj – Nakur bans;

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Sl No.	*Species*	*Source/Local Names*
(1)	(2)	(3)
		Kan – Kiri bidiru; Mah – Male bamboo, nanvel; Oriya – Salia; Tam – Kalmungil; Tel – Sadanapa vedur; Tripura – Lathi bans; Hin – Bans kaban, Nav bans;
xviii)	*Melocanna baccifera*	Asm – Tarai; Ben – Muli; Cachar – Wati; Garo – Watrai; Manip – Moubi; Mikir – Artem; Naga – Turiah.
xix)	*Oxytenanthera abyssinicia*	Native of tropical Africa; cultivated at FRI, Dehradun.
xx)	*Thyrsostachys oliveri*	Native of Myanmar; Planted in Haldwani (Uttaranchal), Arunachal Pradesh, Kerala and Tamil Nadu.

NOTES

1 The following abbreviations have been used in the above table:

Asm	Assam
Ben	Bengali
Guj	Gujarati
Hin	Hindi
Kan	Kannada
Mah	Maharashtra
Manip	Manipur
M.P.	Madhya Pradesh
Nep	Nepali
Sans	Sanskrit
Tam	Tamil
Tel	Telugu

2 The above table does not provide an exhaustive list. It only attempts to enlist some of the information readily available in regard to species of bamboo from India and some of the neighbouring countries, and some connected information.

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ANNEX B

(*Foreword*)

RECOMMENDATIONS FOR BAMBOO STRUCTURES IN HIGH WIND/CYCLONE PRONE AREAS

B-1 Given below are some recommendations for bamboo structures in high wind/cyclone prone areas, which are based on rapid visual screening (RVS) of the damages of past events:

a) Prefer a square plan of the building symmetrical to the arrangement of partitions and openings for balanced distribution of forces within the structure. Window positioning be planned in such a way that wind entering through window could pass through similar opening on opposite wall in order to reduce built-up pressure, and incorporate ridge vents;

b) A hip-shaped, pyramidal or a conical roofing be preferred to gable end or mono-pitch ones for least wind resistance;

c) Wide mono-pitched *verandas* abutting the entrance to residence should be structurally separated from the main building;

d) Roof overhangs be limited to 450 mm as they contribute to higher negative suction. Large overhangs should be properly tied back to wall frame-work;

e) Roof structure shall be properly secured with vertical load-bearing members (wooden or bamboo columns, brick piers or walling) for eliminating the chances of being blown away from the building. A continuous beam connecting all the columns would increase the framing action;

f) Roof cladding (G.I. sheets, clay tiles or thatch) be properly connected with holding down U-bolts, or sturdy anchorages to ensure safety. Thatch roof be tied by nylon ropes in diagonal pattern;

g) Sufficient depth for anchorage be provided; columns be embedded in pits within the foundation and secured by poured concrete. Wetting of column base be checked by raising it from the floor level;

h) Highest importance be given to strengthening the junctions between structural elements at different levels, that is, foundation, eaves and roof. Walls be linked together suitably by a continued collar beam at floor, sill, lintel and roof level; and

j) Fastening any two mutually perpendicular load bearing members should be done in accordance with good practice of spacing the connecting bolts with reference to loads direction and the centre of stiffness. This will increase the moment-resisting capacity of the joint and prevent hinge formation. A minimum of two bolts be provided to prevent rotation of members at a joint.

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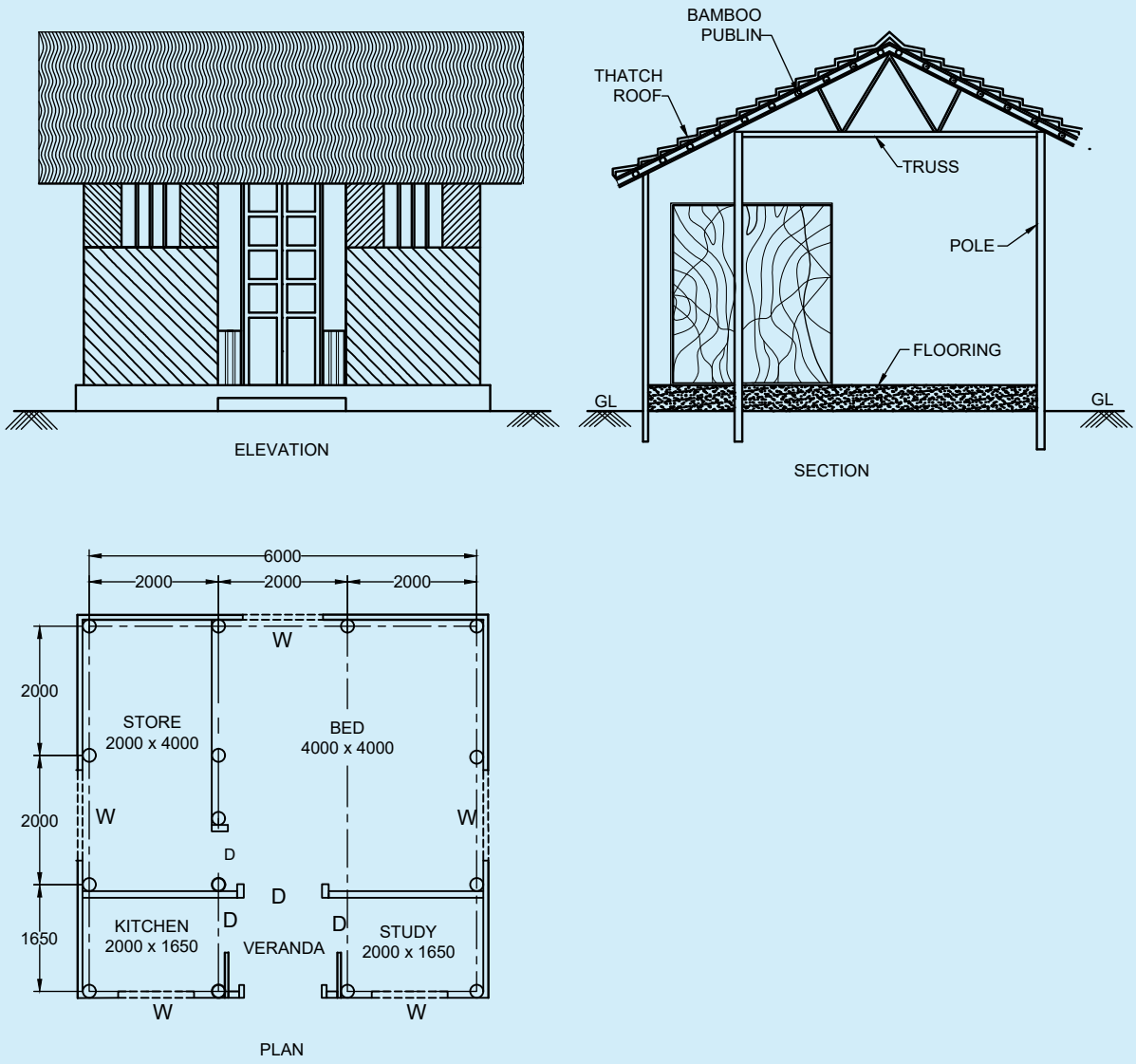
ANNEX C

(*Foreword*, *Clause* 11)

REPRESENTATIVE DESIGNS AND CONSTRUCTION DETAILS FOR BAMBOO STRUCTURES

C-1 For guidance in conventional bamboo construction, typical details are presented in the Fig. 15 These details illustrate common applications of bamboo in trusses, roofing, walls, and flooring. They are intended to serve as indicative examples and not as substitutes for structural design.



All dimensions are in millimetres

FIG. 15 BAMBOO HUT

C-1.1 Figure 16 shows a typical bamboo truss arrangement used in conventional bamboo structures. It highlights jointing methods generally adopted in practice. The illustration is indicative and not a design substitute.

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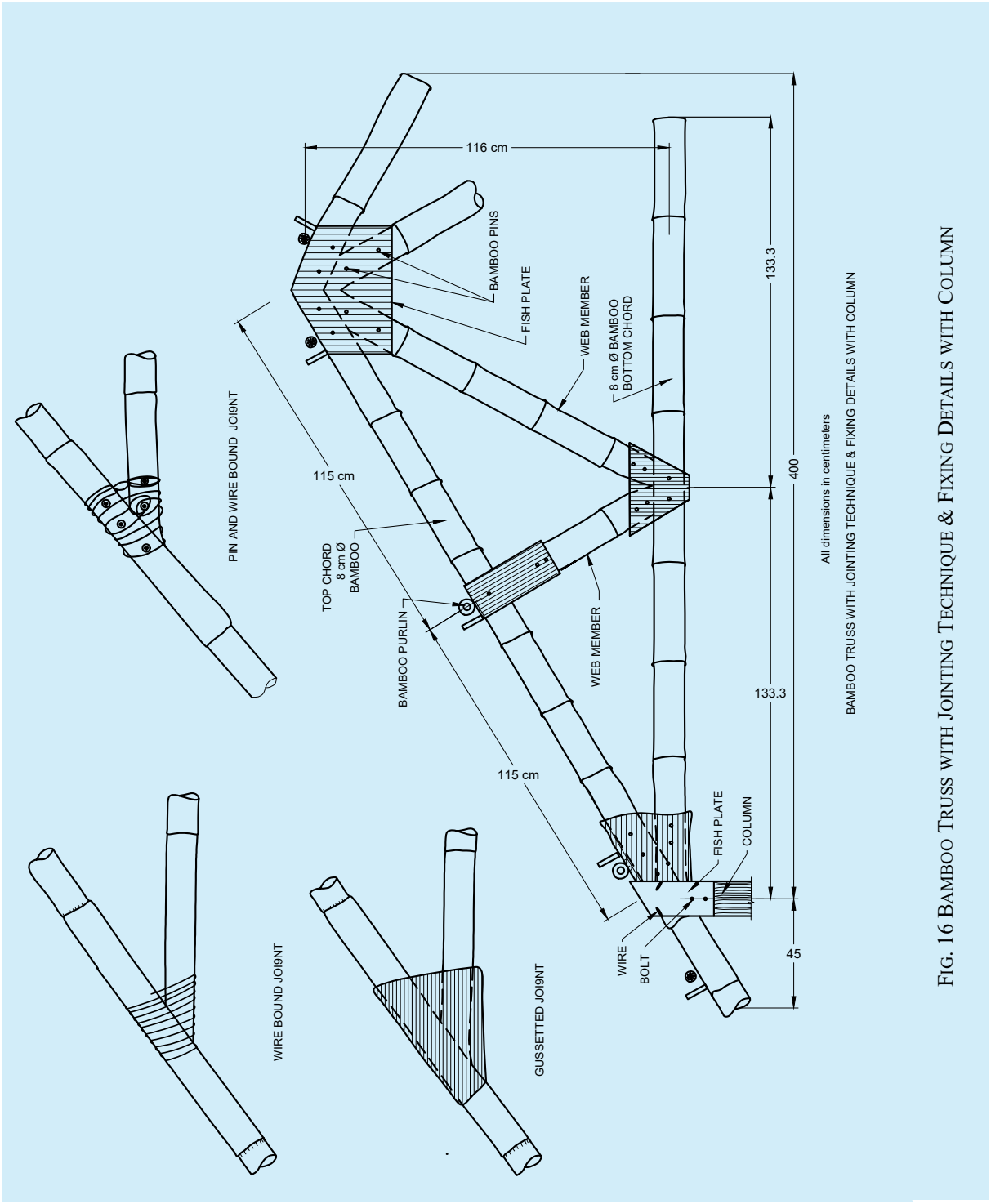

FIG. 16 BAMBOO TRUSS WITH JOINTING TECHNIQUE & FIXING DETAILS WITH COLUMN

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C-1.2 Figure 17 presents a representative bamboo roofing system for conventional bamboo structures. It demonstrates the use of bamboo members in roof framing. The figure is for reference only, not a structural design.

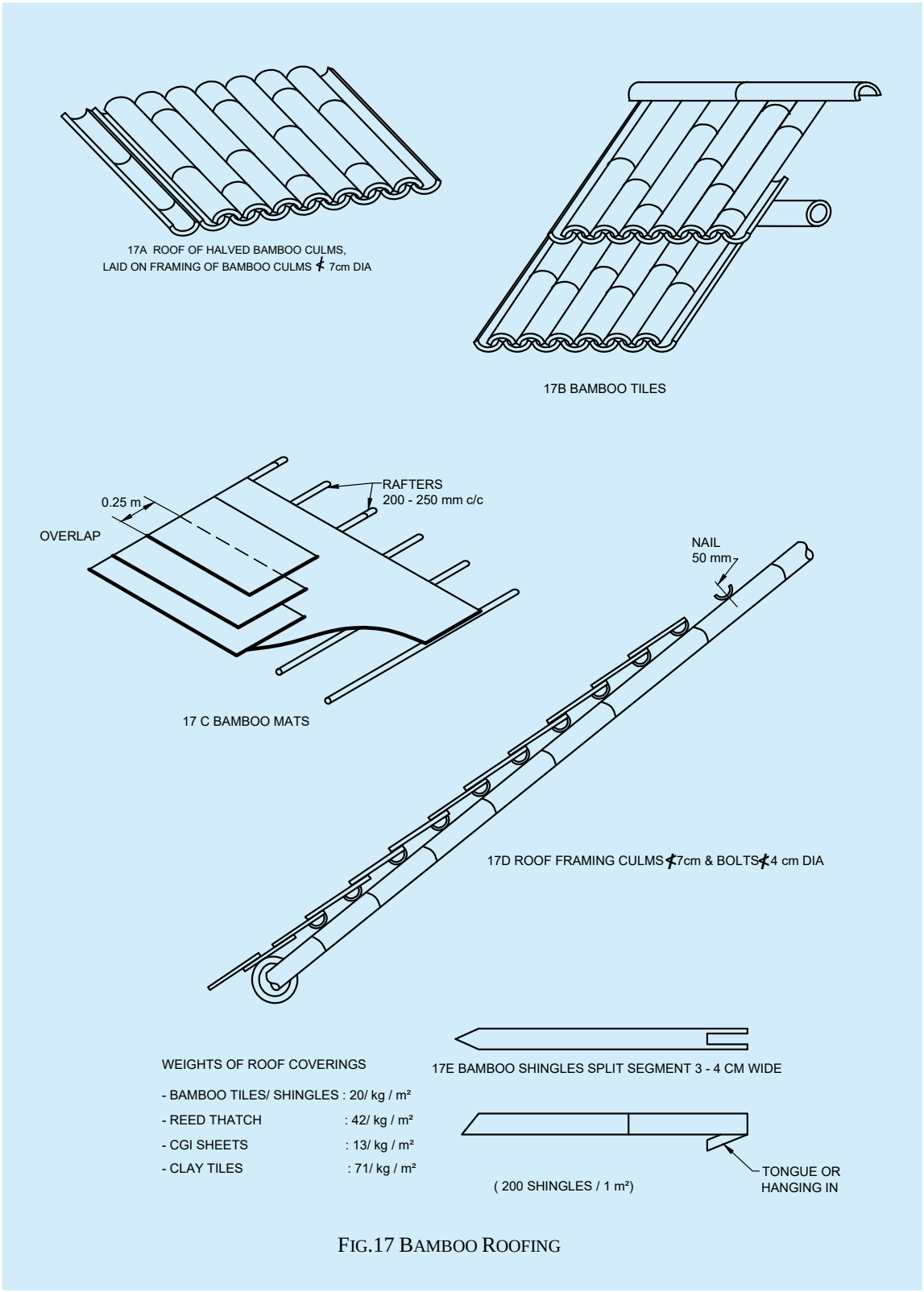

FIG.17 BAMBOO ROOFING

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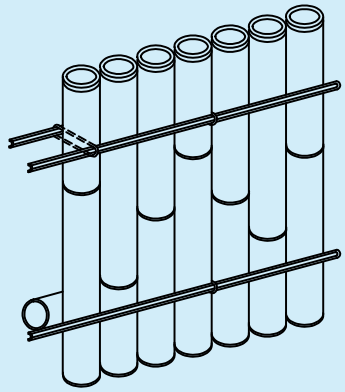
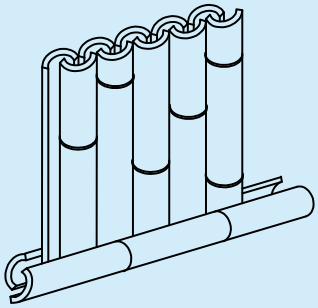
C-1.3 Figure 18 depicts bamboo used as walling elements in conventional bamboo structures. It illustrates construction arrangements commonly followed. The drawing is illustrative and not a substitute for design.



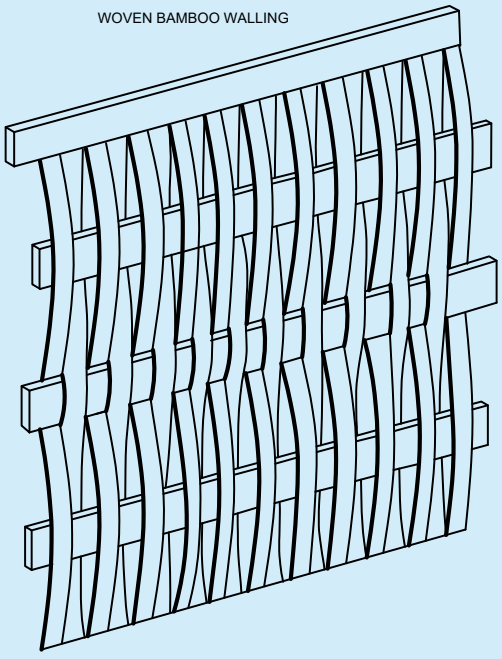
18A WHOLE BAMBOO CULMS AS VERTICALS AND FACED WITH WOVEN BAMBOO MATS

INFILLS BETWEEN FRAMING MEMBERS (IN- PLANE BRACING)



18B HALVED BAMBOO CULMS

WOVEN BAMBOO WALLING



THE PLASTER CONSISTS OF MUD, CLAY, SAND, LIME, COW DUNG, CEMENT AND ORGANIC FIBRES WITH THE ADDITION OF AN INSECTICIDE

FIG.18 BAMBOO WALLS

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C-1.4 Figure 19 shows the application of bamboo in floor decking for conventional bamboo structures. It highlights a representative arrangement for structural flooring. The figure is illustrative only and not for design purposes.

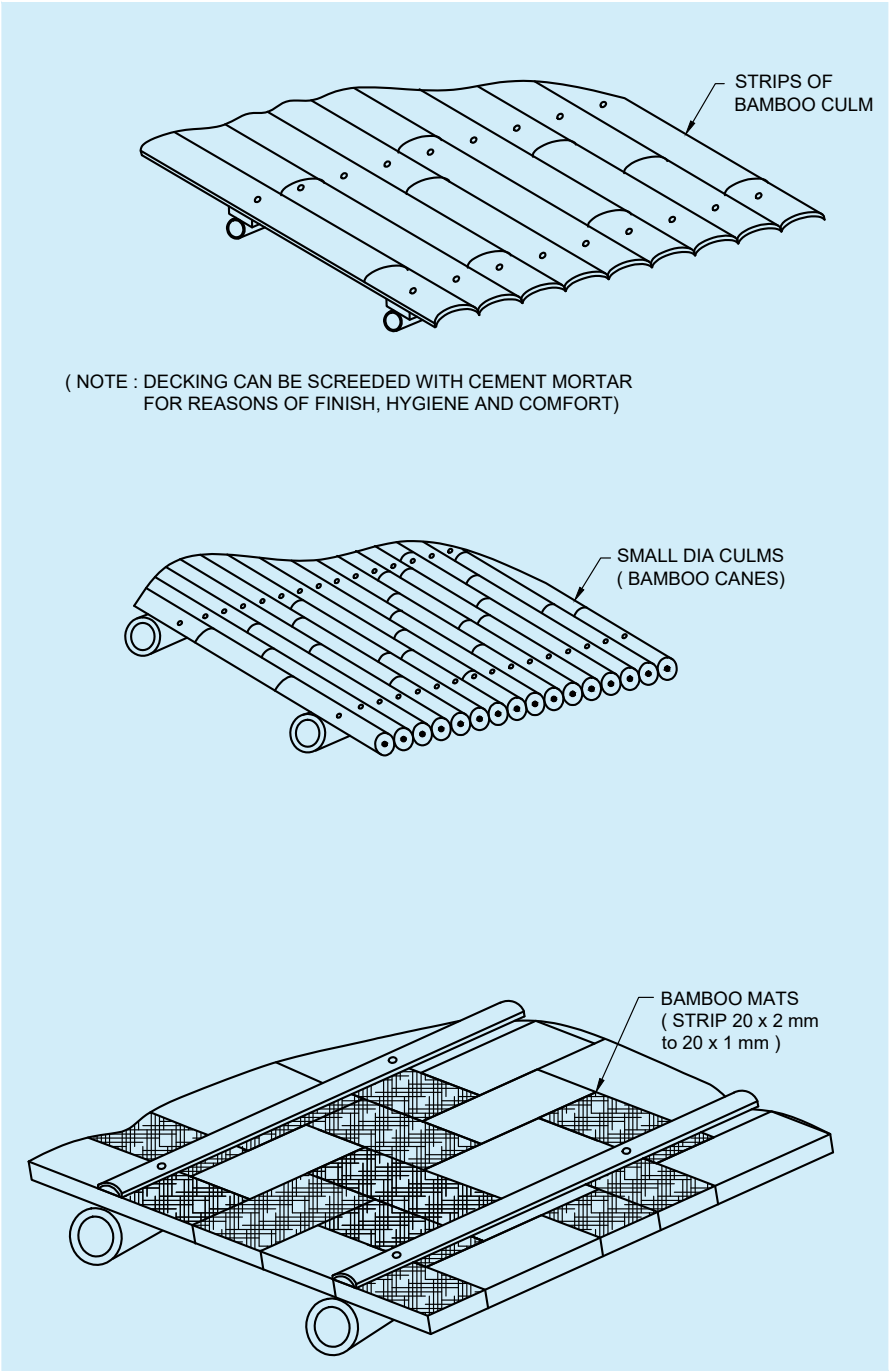

FIG. 19 BAMBOO FLOOR DECKING

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LIST OF STANDARDS

The following list records those standards which are acceptable as 'good practice' and 'accepted standards' in the fulfilment of the requirements of this NBCS. The latest version of the standard shall be adopted at the time of enforcement of this NBCS. The standards listed may be used for conformance with the requirements of the referred clauses in this NBCS.

In the following list, the number appearing in the first column within parenthesis indicates the number of the reference in this Subsection.

	IS No.	*Title*
(1)	IS 9096 : 2006	Preservation of bamboo for structural purposes — Code of practice (*first revision*)
(2)	IS 6874 : 2008	Method of tests for bamboo (*first revision*)
(3)	IS 1902 : 2006	Preservation of bamboo and cane for non-structural purposes — Code of practice (*second revision*)
(4)	IS 8242 : 1976	Bamboo mat corrugated sheets — Specification
(5)	IS 15476 : 2004	Specification for bamboo mat corrugated sheets
(6)	IS 2366 : 1983	Code of practice for nail-jointed timber construction (*first revision*)
(7)	IS 4924	Method of test for nail-jointed timber trusses
	(Part 1) : 1968	Destructive test
	(Part 2) : 1968	Proof test
(8)	IS 13958 : 1994	Bamboo mat board for general purposes — Specification
(9)	IS 14588 : 1999	Bamboo mat-veneer composite for general purposes — Specification
(10)	IS 15972 : 2012	Bamboo-jute composite corrugated and semi-corrugated sheets — Specification
(11)	IS 1734 (Parts 1 to 20) : 1983	Methods of test for plywood (*second revision*)
(12)	IS 1708 (Parts 1 to 18) : 1986	Methods of testing of small clear specimens of timber (*second revision*)
(13)	IS 848 : 2025	Synthetic resin adhesives for plywood (phenolic, aminoplastic and bio-materials based) — Specification (*third revision*)

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 4 MASONRY

BUREAU OF INDIAN STANDARDS

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National Building Code Sectional Committee, CED 46

FOREWORD

This NBCS (Part C/Section 4) covers the structural design of unreinforced and reinforced masonry elements in buildings. Provisions on reinforced brick and reinforced brick concrete floors and roofs have also been dealt with.

This Section was first published in 1970 and subsequently revised in 1983, 2005 and 2016 as a chapter of National Building Code of India. The first revision of this Section was modified in 1987 through Amendment No. 2 to bring this Section in line with the latest revised masonry code. In the amendment, certain provisions were updated following the revision of IS 1905 'Code of practice for structural use of unreinforced masonry' on which the earlier version was based. In the amendment, requirements of masonry element for stability were modified; in the design of free standing wall, provisions were made for taking advantage of the tensile resistance in masonry under certain conditions; provision regarding effective height of masonry wall between openings was modified; method of working out effective height of wall with a membrane type DPC was modified; the criteria for working out effective length of wall having openings was modified; some general guidelines for dealing with concentrated loads for design of walls were included; and provision of cutting and chases in walls were amplified. The Section was thereafter revised in 2005.

In the second revision of this Section in 2005, the provisions of special considerations in earthquake zones were aligned in line with the revised Indian Standard, IS 4326 : 1993 'Code of practice for earthquake resistant design and construction of buildings'; a new clause covering guidelines for improving earthquake resistance of low strength masonry buildings was added; and reference to design of reinforced brick and reinforced brick concrete floors and roofs was included.

In the third revision of this Section in 2016, the detailed provisions relating to reinforced masonry and durability criteria for selection of masonry mortars had been included. Also, the number of storeys of masonry construction had been limited to 4, in line with the seismic design standards. A new masonry wall construction using rat-trap bond had been included.

In this revision, this publication has been renamed as National Building Construction Standards (NBCS).

As a result of experience gained in the implementation of this Section and feedback received, as well as in view of formulation of new varieties of cement and new types of masonry blocks, a need to revise this Section was again felt. This section has, therefore, been revised to incorporate the following significant changes:

1) New masonry blocks have been included as masonry unit.
2) The admissibility of different structural systems in the earthquake zones is clarified. The following structural systems are admissible to construct earthquake resistant masonry buildings:
    a) Masonry walls with prescriptive bands (MWB);
    b) Masonry walls with prescriptive bands and vertical reinforcement (MWBR);
    c) Confined masonry walls (CMW); and
    d) Reinforced masonry walls (RMW).
3) The clause on confined masonry has been updated.
4) The clause on reinforced masonry has been updated.
5) Reference to all the concerned Indian Standards has been updated.

Structural design requirements of this section are largely based on following Indian Standards:

a) IS 1905 : 1987 'Code of practice for structural use of unreinforced masonry (*third revision*)';
b) IS 4326 : 2013 'Earthquake resistant design and construction of buildings — Code of practice (*third revision*)';

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c) IS 10440 :1983 ‘Code of practice for construction of RB and RBC floors and roofs’; and

d) IS 17848 : 2022 ‘Confined masonry for earthquake resistance — Code of practice’.

Buildings with MWB and MWBR systems shall not be of height more than 15 m subject to a maximum of four storeys, measured from the base of the buildings to the roof slab or ridge level.

A reference to SP 20 : 1991 ‘Handbook on masonry design and construction (*first revision*)’, may be useful. In the formulation of this Section, inputs were also derived from the IITK-GSDMA’s publication, ‘Guidelines for structural use of reinforced masonry’.

All standards, whether given herein above or cross-referred to in the main text of this section, are subject to revision. The parties to agreement based on this section are encouraged to investigate the possibility of applying the most recent editions of the standards.

As the regulation of land development and buildings is a State subject and is dealt by the States and local bodies such as urban local bodies within their jurisdiction through building regulatory documents like building regulations, building byelaws, and fire regulations, this document is voluntary in nature and is non-binding. The implementation depends on adoption by concerned parties or stipulations in a contract or by appropriate adoption by the concerned authorities.

For the purpose of deciding whether a particular requirement of this Section is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 2022 ‘Rules for rounding off numerical values (*second revision*)’. The number of significant places retained in the rounded off value should be the same as that of the specified value in this Section.

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Important Explanatory Note for Users of the NBCS

In any Part/Section of this NBCS, where reference is made to **'good practice'** in relation to design, constructional procedures or other related information, and where reference is made to **'accepted standard'** in relation to material specification, testing, or other related information, the Indian Standards listed at the end of the Part/Section shall be used as a guide to the interpretation.

At the time of publication, the editions indicated in the standards were valid. All standards are subject to revision and parties to agreements based on any Part/Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

In the list of standards given at the end of a Part/Section, the number appearing within parentheses in the first column indicates the number of the reference of the standard in the Part/Section. For example:

a) Accepted standards [C-4(1)] refers to the Indian Standard(s) give at serial number (1) of the list of standards given at the end of this Part C/Section 4, that is, IS 1077 : 1992 'Specification for common burnt clay building bricks (*fifth revision*)', IS 1725 : 2023 'Specification for stabilized soil blocks used in general building construction (*third revision*)', IS 2180 : 1988 'Specification for heavy duty burnt clay building bricks (*third revision*)', IS 2185 (Part 1) : 2005 'Concrete masonry units — Specification: Part 1 Hollow and solid concrete blocks (*third revision*)', IS 2185 (Part 2) : 1983 'Concrete masonry units — Specification: Part 2 Hollow and solid light weight concrete blocks (*first revision*) (superseding IS 3590)', IS 2185 (Part 3) : 1984 'Concrete masonry units— Specification: Part 3 Autoclaved cellular (aerated) concrete blocks (*first revision*)(superseding IS 5482)', IS 2185 (Part 4) : 2008 'Concrete masonry units — Specification: Part 4 Cellular preformed foam cellular concrete blocks', IS 2222 : 1991 'Specification for burnt clay perforated building bricks (*fourth revision*)', IS 2849 : 1983 'Specification for non-load bearing gypsum partition blocks (solid and hollow types)(*first revision*)', IS 3115 : 1992 'Specification for lime based blocks (*second revision*)', IS 3316 : 1974 'Specification for structural granite (*first revision*)', IS 3620 : 1979 'Specification for laterite stone block for masonry (*first revision*)', IS 3952 : 2013 'Burnt clay hollow bricks and blocks for walls and partitions — Specification (*third revision*)', IS 4139 : 1989 'Specification for calcium silicate bricks (*second revision*)', IS 12440 : 1988 'Specification for precast concrete stone masonry blocks', IS 12894 : 2002 'Specification for pulverized fuel ash lime bricks (*first revision*)', IS 13757 : 1993 'Specification for burnt clay fly ash building bricks' and IS 16720 : 2018 'Pulverized fuel ash-cement bricks — Specification'.

b) Good practice [C-4(4)] refers to the Indian Standard(s) give at serial number (4) of the list of standards given at the end of this Part C/Section 4, that is,IS 2250 : 1981 'Code of practice for preparation and use of masonry mortars (*first revision*)'.

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 4 MASONRY

1 SCOPE

1.1 This NBCS (Part C/Section 4) covers the structural design aspects of unreinforced load bearing and non-load bearing walls, constructed with masonry units permitted in accordance with this Section.

1.2 This Section also deals with the selection of materials, special features of design and construction for masonry construction using rectangular masonry units. It also covers guidelines regarding earthquake resistant buildings constructed using masonry of low strength.

1.3 This Section also covers provisions for design of reinforced brick and reinforced brick concrete floors and roofs.

1.4 This Section also gives the recommendations for structural design aspects of reinforced load bearing and non-load bearing walls, constructed with different types of bricks and blocks.

1.5 The recommendations of the Section do not apply to walls constructed in mud mortars.

2 TERMINOLOGY

For the purpose of this Section, the following definitions shall apply.

2.1 Bed Block — A block bedded on a wall, column or pier to disperse a concentrated load on a masonry element.

2.2 Bond — Arrangement of masonry units in successive courses to tie the masonry together both longitudinally and transversely; the arrangement is usually worked out to ensure that no vertical joint of one course is exactly over the one in the next course above or below it, and there is maximum possible amount of lap.

2.3 Column, Pier and Buttress

a) *Column* — An isolated vertical load bearing member, width of which does not exceed four times the thickness;

b) *Pier* — A thickened section forming integral part of a wall placed at intervals along the wall, to increase the stiffness of the wall or to carry a vertical concentrated load. Thickness of a pier is the overall thickness including the thickness of the wall or, when bonded into a leaf of a cavity wall, the thickness obtained by treating that leaf as an independent wall (*see* Fig. 1)

c) *Buttress* — A pier of masonry built as an integral part of wall and projecting from either or both surfaces, decreasing in cross-sectional area from base to top.

2.4 Cross-Sectional Area of Masonry Unit — Net cross-sectional area of a masonry unit shall be taken as the gross cross-sectional area minus the area of cellular space. Gross cross-sectional area of cored units shall be determined to the outside of the coring but cross-sectional area of grooves shall not be deducted from the gross cross-sectional area to obtain the net cross-sectional area.

2.5 Curtain Wall — A non-load bearing wall subject to lateral loads. It may be laterally supported by vertical or horizontal structural members, where necessary (*see* Fig. 2).

2.6 Effective Height — The height of a wall or column, to be considered for calculating slenderness ratio.

2.7 Effective Length — The length of a wall to be considered for calculating slenderness ratio.

2.8 Effective Thickness — The thickness of a wall or column to be considered for calculating slenderness ratio.

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2.9 Hollow Unit — A masonry unit of which net cross-sectional area in any plane parallel to the bearing surface is less than 75 percent of its gross cross-sectional area measured in the same plane.

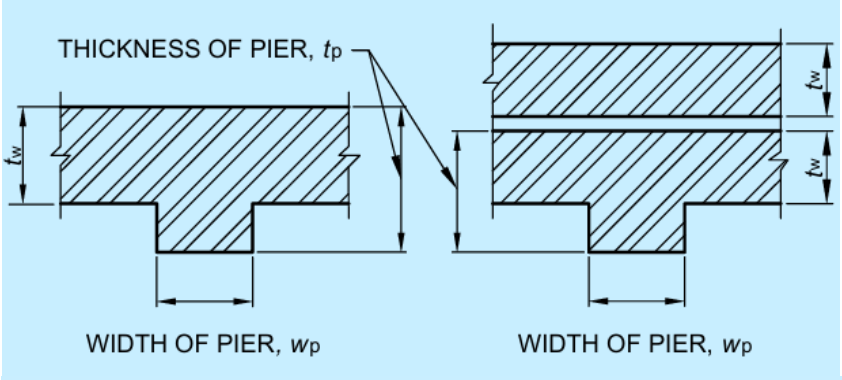
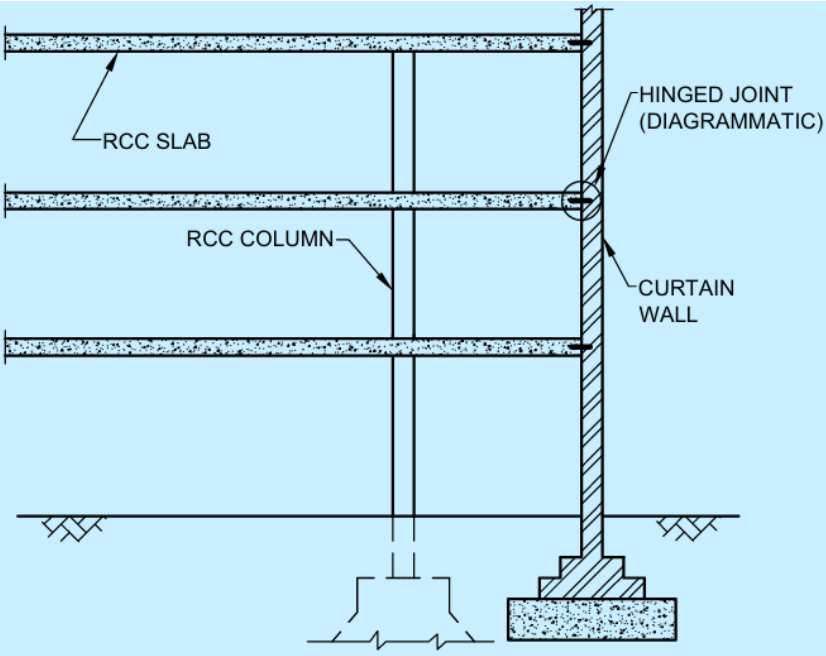


FIG. 1 DEFINITION OF PIER



NOTE — The figure is only illustrative to explain about the term curtain wall and is not intended to be an exhaustive provision.

FIG. 2 TYPICAL MASONRY CURTAIN WALL

2.10 Grout — Mortar of pourable consistency.

2.11 Joint — A junction of masonry units, which can be classified as:

a) *Bed Joint* — A horizontal mortar joint upon which masonry units are laid;

b) *Cross Joint* — A vertical joint, normal to the face of the wall; and

c) *Wall Joint* — A vertical joint parallel to the face of the wall.

2.12 Leaf — Inner or outer section of a cavity wall.

2.13 Lateral Support — A support which enables a masonry element to resist lateral load and/or restrains lateral deflection of a masonry element at the point of support.

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2.14 Load Bearing Wall — A wall designed to carry an imposed vertical load in addition to its own weight, together with any lateral load.

2.15 Masonry — An assemblage of masonry units properly bonded together with mortar.

2.16 Masonry Unit — Individual units which are bonded together with the help of mortar to form a masonry element such as wall, column, pier, buttress, etc.

2.17 Partition Wall — An interior non-load bearing wall, one storey or part storey in height.

2.18 Panel Wall — An exterior non-load bearing wall in framed construction, wholly supported at each storey but subjected to lateral loads.

2.19 Shear Wall — A wall designed to carry horizontal forces acting in its plane with or without vertical imposed loads.

2.20 Slenderness Ratio — Ratio of effective height or effective length to effective thickness of a masonry element.

2.21 Types of Walls

a) *Cavity Wall* — A wall comprising two leaves, each leaf being built of masonry units and separated by a cavity and tied together with metal ties or bonding units to ensure that the two leaves act as one structural unit, the space between the leaves being either left as continuous cavity or filled with a non-load bearing insulating and water-proofing material;

b) *Faced Wall* — A wall in which facing and backing of two different materials are bonded together to ensure common action under load (*see* Fig. 3).

NOTE — To ensure monolithic action in faced walls, shear strength between the facing and the backing shall be provided by toothing, bonding or other means; and

c) *Veneered Wall* — A wall in which the facing is attached to the backing but not so bonded as to result in a common action under load.

2.22 Reinforced Brick Slab (RB Slab) — Reinforced brick slab is particularly same as reinforced concrete slab in all its essential features except that brickwork is substituted partly for cement concrete. *See* Fig. 4(A) for details.

2.23 Reinforced Brick Concrete Slab (RBC Slab) — These are reinforced brick slab using concrete in the joints and on the top of bricks. *See* Fig. 4(B) for details.

3 MATERIALS

3.1 General

The materials used in masonry construction shall be in accordance with Part B ‘Building Materials’ of this NBCS. Storage of materials shall be in accordance with good practice [C-4(2)].

3.2 Masonry Units

Masonry units used in construction shall conform to accepted standards [C-4(1)].

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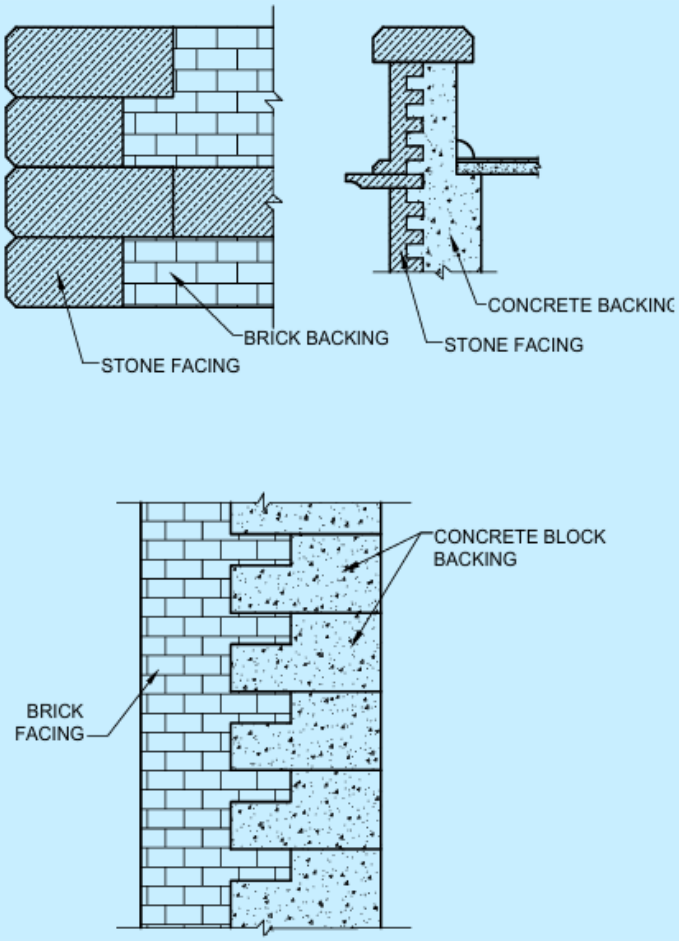

FIG. 3 TYPICAL FACED WALL

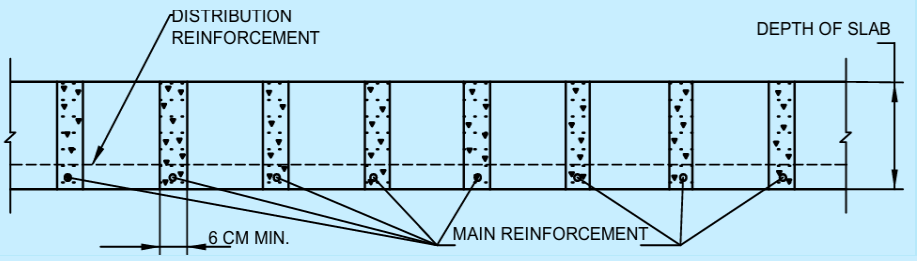

FIG. 4(A) CROSS-SECTION OF RB SLAB

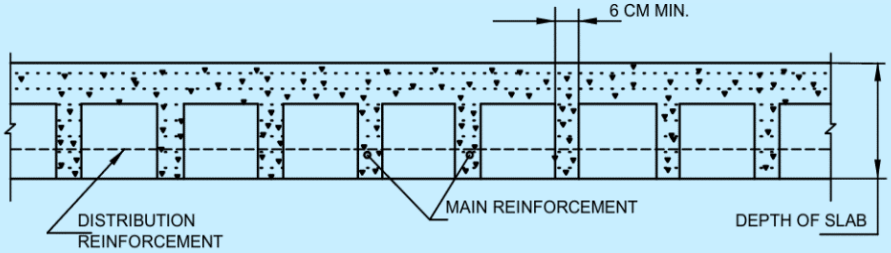

FIG. 4(B) CROSS-SECTION OF RBC SLAB

FIG. 4 CROSS-SECTIONS OF RB AND RBC SLABS

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3.2.1 Masonry units may be of the following types:

a) Common burnt clay building bricks;

b) Burnt clay fly ash building bricks;

c) Pulverized fuel ash lime bricks;

d) Pulverized fuel ash cement bricks;

e) Stones (in regular sized units);

f) Sand-lime bricks;

g) Concrete blocks (solid and hollow);

h) Lime based blocks;

j) Burnt clay hollow blocks;

k) Gypsum partition blocks,

m) Autoclaved cellular concrete blocks;

n) Stabilized soil blocks; and

p) Concrete stone masonry blocks.

NOTES

1 Gypsum partition blocks are used only for construction of non-load bearing partition walls.

2 Use of other masonry units such as precast stone blocks, fly-ash-lime-gypsum bricks, stabilized soil blocks and other bricks/blocks not covered by the above specifications may also be permitted based on test results.

3.2.2 Masonry units that have been previously used shall not be reused in brickwork or blockwork construction, unless they have been thoroughly cleaned and conform to this Section for similar new masonry units.

3.3 Mortar

Mortar for masonry shall conform to accepted standard [C-4(3)].

3.3.1 Minimum compressive strength for different grades of mortar on the basis of mix proportions as per **3.3.2** are given in Table 1.

3.3.2 Mix proportions and compressive strengths of some of the commonly used mortars are given in Table 2.

Table 1 Compressive Strength for Different Grades of Mortar

(*Clauses* 3.3.1, 3.4.1, 5.4.3, G-4.1 *and* Table 9)

Sl No.	Grade of Mortar	Minimum Compressive Strength at 28 Days N/mm^2
(1)	(2)	(3)
i)	H1	10
ii)	H2	6.0
iii)	M1	5.0
iv)	M2	3.0
v)	M3	1.5
vi)	L1	0.7
vii)	L2	0.5

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3.4 Durability Criteria for Selection of Masonry Mortars

3.4.1 The selection of masonry mortars from durability considerations shall cover both the loading and exposure conditions of the masonry. The requirements for masonry mortar shall generally be as given in **3.4.2** to **3.4.6** (*see also* Table 1).

3.4.2 In the case of masonry exposed frequently to rains and where there is further protection by way of plastering or rendering or other finishes, the grade of mortar shall not be less than M3 but shall preferably be of grade M2. Where no protection is provided, the grade of mortar for external walls shall not be less than M2.

3.4.3 In the case of load bearing internal walls, the grade of mortar shall preferably be M3 or more for high durability but in no case less than L1.

3.4.4 In the case of masonry in foundations laid below damp-proof course, the grades of mortar for use in masonry shall be as specified below:

a) Where soil has little moisture, masonry mortar of grade not less M3 shall be used;

b) Where soil is very damp, masonry mortar of grade preferably M2 or more shall be used. But in no case shall the grade of mortar be less than L1 and M3; and

c) Where soil is saturated with water, masonry mortar of grade M1 shall be used but in no case shall the grade of mortar be less than M2.

3.4.5 For masonry in buildings subject to vibration of machinery, the grade of mortar shall not be less than M1.

3.4.6 For parapets, where the height is greater than three times the thickness, the grade of masonry mortar used shall not be less than M1. In the case of low parapets, the grade of mortar shall be the same as that used in the wall masonry below.

3.4.7 The grade of mortar for bedding joints in masonry with large concrete blocks shall not be less than M1.

4 DESIGN CONSIDERATIONS

4.1 General

Masonry structures gain stability from the support offered by cross walls, floors, roof and other elements such as piers and buttresses. Load bearing walls are structurally more efficient when the load is uniformly distributed and the structure is so planned that eccentricity of loading on the members is as small as possible. Avoidance of eccentric loading by providing adequate bearing of floor/roof on the walls providing adequate stiffness in slabs and avoiding fixity at the supports, etc, is especially important in load bearing walls in multistorey structures. These matters should receive careful consideration during the planning stage of masonry structures.

4.2 Lateral Supports and Stability

4.2.1 *Lateral Supports*

Lateral supports for a masonry element such as load bearing wall or column are intended,

a) to limit slenderness of a masonry element so as to prevent or reduce possibility of buckling of the member due to vertical loads; and

b) to resist horizontal components of forces so as to ensure stability of a structure against overturning.

4.2.1.1 Lateral support may be in the vertical or horizontal direction, the former consisting of floor/roof bearing on the wall or properly anchored to the same and latter consisting of cross walls, piers or buttresses.

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Table 2 Mix Proportions and Strength of Mortars for Masonry

(*Clause* 3.3.2 *and* Table 23)

Sl No.	Mix Proportions (By Mass)					Minimum Compressive Strength at 28 Days in N/mm² (with Cement Compressive Strength at 28 Days, in N/mm², between 33 and 43)	Minimum Compressive Strength of Cement : Sand Mortar at 28 Days in N/mm² (with Cement Compressive Strength at 28 Days, in N/mm² ≥ 43)
	Cement	Lime	Lime - Pozzolana Mixture	Pozzolana	Sand		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	1	¼ C or B	0	0	3	10	—
2(a)	1	¼ C or B	0	0	4	7.5	—
2(b)	1	½ C or B	0	0	4 ½	6	—
2(c)	1	0	0	0	4	6	7
3(a)	1	0	0	0	5	5	6
3(b)	1	1 C or B	0	0	6	3	—
3(c)	0	0	1(LP–40)	0	1 ½	3	—
4(a)	1	0	0	0	6	3	4
4(b)	1	2B	0	0	9	2	—
4(c)	0	1A	0	0	2	2	—
4(d)	0	1 B	0	1	1	2	—
4(e)	0	1 C or B	0	2	0	2	—
4(f)	0	0	1(LP–40)	0	1 ¾	2	—
5(a)	1	0	0	0	7	2	3
5(b)	1	3 B	0	0	12	1.5	—
5(c)	0	1 A	0	0	3	1.5	—
5(d)	0	1 B	0	2	1	1.5	—

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Table 2 (*Concluded*)

Sl No.	Mix Proportions (By Mass)					Minimum Compressive Strength at 28 Days in N/mm² (with Cement Compressive Strength at 28 Days, in N/mm², between 33 and 43)	Minimum Compressive Strength of Cement : Sand Mortar at 28 Days in N/mm² (with Cement Compressive Strength at 28 Days, in N/mm² ≥ 43)
	Cement	Lime	Lime - Pozzolana Mixture	Pozzolana	Sand		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
5(e)	0	1 C or B	0	3	0	1.5	—
5(f)	0	0	1(LP-40)	0	2	1.5	—
6(a)	1	0	0	0	8	1	2
6(b)	0	1 B	0	1	2	0.7	—
6(c)	0	1 C or B	0	2	1	0.7	—
6(d)	0	0	1(LP-40)	0	2 ¼	0.7	—
6(e)	0	0	1(LP-20)	0	1 ½	0.7	—
7(a)	0	1 B	0	0	3	0.5	—
7(b)	0	1 C or B	0	1	2	0.5	—
7(c)	0	0	1(LP-7)	0	1 ½	0.5	—

NOTES

1 Sand for making mortar should be well graded. In case sand is not well graded, its proportions shall be reduced in order to achieve the minimum specified strength.

2 For mixes in Sl No. 1 and Sl No. 2, use of lime is not essential from consideration of strength as it does not result in increase in strength. However, its use is highly recommended since if improves workability.

3 For mixes in Sl No. 3(a), 4(a), 5(a) and 6(a) either lime C or B to the extent of ¼ part of cement (by mass) or some plasticizer should be added for improving workability.

4 For mixes in Sl No. 4(b) and 5(b), lime and sand should first be ground in mortar mill and then cement added to coarse stuff.

5 It is essential that mixes in Sl No. 4(c), 4(d), 5(d), 5(e), 6(b), 6(c), 7(a) and 7(b) are prepared by grinding in a mortar mill.

6 A, B and C denote eminently hydraulic lime, semi-hydraulic lime and fat lime respectively, as specified in appropriate standards listed in Part B ‘Building Materials’ of this NBCS.

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4.2.1.2 Requirements of 4.2.1(a) from consideration of slenderness may be deemed to have been met with, if:

a) in case of a wall, where slenderness ratio is based on effective height, any of the following constructions are provided;

1) RCC floor/roof slab (or beams and slab) irrespective of the direction of span, bears on the supported wall as well as cross walls, to the extent of at least 90 mm;

2) RCC floor/roof slab not bearing on the supported wall or cross wall is anchored to it with non-corrodible metal ties of 600 mm length and of section not less than 6 mm × 30 mm, and at intervals not exceeding 2 m, as shown in Fig. 5; and

3) Timber floor/roof, anchored by non-corrodible metal ties of length 600 mm and of minimum section 6 mm × 30 mm, securely fastened to joists and built into walls as shown in Fig. 6 and Fig. 7. The anchors shall be provided in the direction of span of timber joists as well as in its perpendicular direction, at intervals of not more than 2 m in buildings up to two storeys and 1.25 m for buildings more than two storeys in height.

NOTES

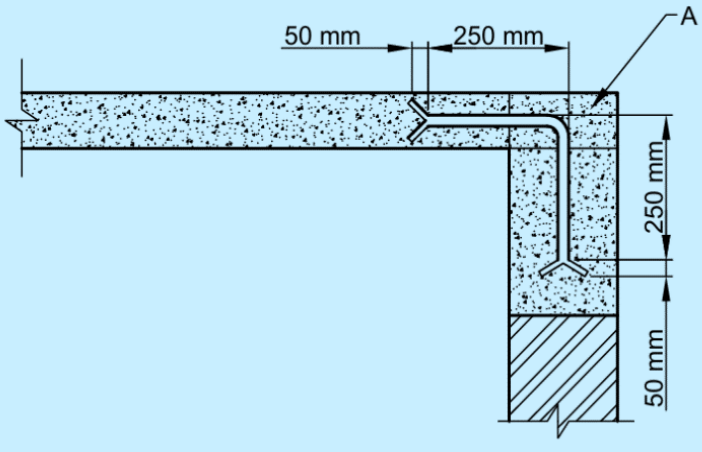
1 In case precast RCC units are used for floors and roofs, it is necessary to interconnect them and suitably anchor them to the cross walls so that they can transfer lateral forces to the cross walls.

2 In case of small houses of conventional designs, not exceeding two storeys in height, stiffening effect of partitions and cross walls is such that metal anchors are normally not necessary in case of timber floor/roof and precast RCC floor/roof units.

b) in case of a wall, when slenderness ratio is based on its effective length; a cross wall/pier/buttress of thickness equal to or more than half the thickness of the supported wall or 90 mm, whichever is more, and length equal to or more than one fifth of the height of wall, is built at right angle to the wall (*see* Fig. 8) and bonded to it according to provision of 4.2.2.2 (d);

c) in case of a column, an RCC or timber beam/RS joist/roof truss, is supported on the column. In this case, the column will not be deemed to be laterally supported in the direction at right angle to it; and

d) in case of a column, an RCC beam forming a part of beam and slab construction is supported on the column, and slab adequately bears on stiffening walls. This construction will provide lateral support to the column, in the direction of both horizontal axes.



A = Cement concrete only at places where anchors are provided
(200 mm in width in the direction perpendicular to the plane of paper)

FIG. 5 ANCHORING OF RCC SLAB WITH MASONRY WALL (WHEN SLAB DOES NOT BEAR ON WALL)

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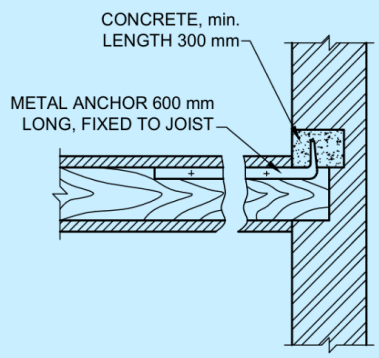

FIG. 6(A) JOISTS AT RIGHT ANGLE TO WALL

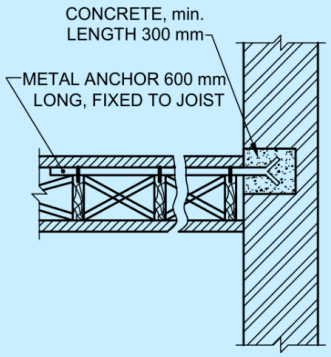

FIG. 6(B) JOISTS PARALLEL TO WALL

FIG. 6 TYPICAL DETAILS FOR ANCHORAGE OF SOLID WALLS

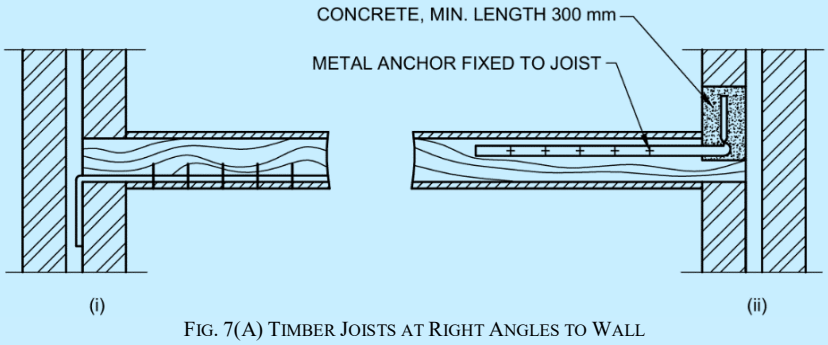

FIG. 7(A) TIMBER JOISTS AT RIGHT ANGLES TO WALL

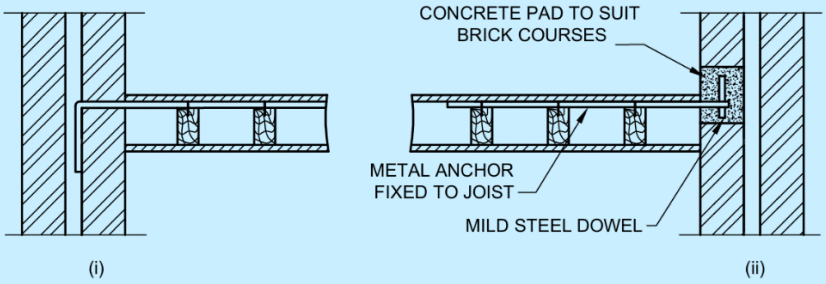

FIG. 7(B) TIMBER JOISTS PARALLEL TO WALL

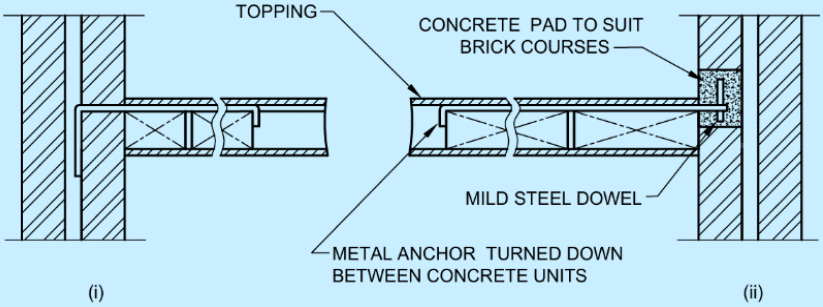

FIG. 7(C) PRECAST CONCRETE FLOOR UNITS PARALLEL TO WALL

FIG. 7 TYPICAL DETAILS FOR ANCHORAGE OF CAVITY WALLS

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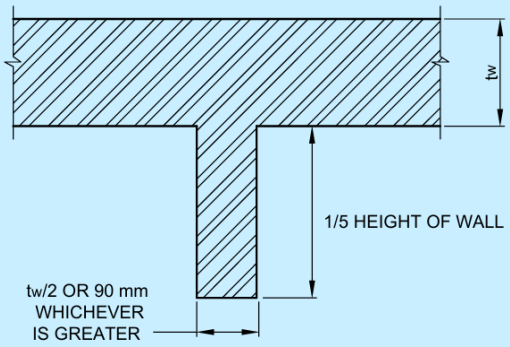

FIG. 8 MINIMUM DIMENSION FOR MASONRY WALL OR BUTTRESS EFFECTIVE LATERAL SUPPORT

4.2.2 *Stability*

A wall or column subjected to vertical and lateral loads may be considered to be provided with adequate lateral support from consideration of stability, if the construction providing the support is capable of resisting some of the following forces:

a) Simple static reactions at the point of lateral support to all the lateral loads; plus

b) 2.5 percent of the total vertical load that the wall or column is designed to carry at the point of lateral support.

4.2.2.1 For the purpose specified in **4.2.2**, if the lateral supports are in the vertical direction, these should meet the requirements given in **4.2.1.2** (a) and should also be capable of acting as horizontal girders duly anchored to the cross wall so as to transmit the lateral loads to the foundations without exceeding the permissible stresses in the cross walls.

4.2.2.2 In case of load bearing buildings up to four storeys, stability requirements of **4.2.2** may be deemed to have been met with, if:

a) height to width ratio of building does not exceed 2;

b) cross walls acting as stiffening walls continuous from outer wall to outer wall or outer wall to a load bearing inner wall, and of thickness and spacings as given in Table 3 are provided. If stiffening wall or walls that are in a line, are interrupted by openings, length of solid wall or walls in the zone of the wall that is to be stiffened shall be at least one-fifth of height of the opening as shown in Fig. 9;

c) floors and roof either bear on cross walls or are anchored to those walls as in **4.2.1.2** such that all lateral loads are safely transmitted to those walls and through them to the foundation; and

d) cross walls are built jointly with the bearing walls and are jointly mortared, or the two interconnected by toothing. Alternatively, cross walls may be anchored to walls to be supported by ties of non-corrodible metal of minimum section 6 mm × 35 mm and length 600 mm with ends bend at least 50 mm; maximum vertical spacing of ties being 1.2 m (*see* Fig. 10).

4.2.2.3 In case of halls exceeding 8.0 m in length, safety and adequacy of lateral supports shall always be checked by structural analysis.

4.2.2.4 A trussed roofing may not provide lateral support unless special measures are adopted to brace and anchor the roofing. However, in case of residential and similar buildings of conventional design with trussed roofing having cross walls, it may be assumed that stability requirements are met with by the cross walls and structural analysis for stability may be dispensed with.

4.2.2.5 Capacity of a cross wall, also called shear wall, sometimes to take horizontal loads and consequently bending moments increases, when parts of bearing walls act as flanges to the cross wall. Maximum overhanging length of bearing wall which could effectively function as a flange should be taken as $12t$ or $H/6$, whichever is less in case of *T/I* shaped walls, and $6t$ or $H/16$, whichever is less in case of *L/U* shaped walls, where t is the thickness of bearing wall and H is the total height of wall above the level being considered, as shown in Fig. 11.

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4.2.2.6 *External walls of basement and plinth*

In case of external walls of basement and plinth, stability requirements of **4.2.2** may be deemed to have been met with, if:

a) bricks used in basement and plinth have a minimum crushing strength of 5 N/mm^2 and mortar used in masonry is of Grade M1 or better;

b) clear height of ceiling in basement does not exceed 2.6 m;

c) walls are stiffened according to provisions of **4.2.2.1**;

d) in the zone of action of soil pressure on basement walls, traffic load excluding any surcharge due to adjoining buildings does not exceed 5 kN/m^2 and terrain does not rise; and

e) minimum thickness of basement walls is in accordance with Table 4. However, if percentage opening in the basement wall for purposes like ventilation are more than 25 percent, then stability analysis/check need to be performed particularly for resistance to seismic loads.

NOTE — In case there is surcharge on basement walls from adjoining buildings, thickness of basement walls shall be based on structural analysis.

4.2.2.7 *Walls mainly subjected to lateral loads*

a) *Free standing wall* — A free standing wall such as compound wall or parapet wall is acted upon by wind force which tends to overturn it. This tendency to over-turning is resisted by gravity force due to self-weight of wall, and also by flexural moment of resistance on account of tensile strength of masonry. Free standing walls shall thus be designed as in **5.5.2.1**. If mortar used for masonry cannot be relied upon for taking flexural tension (*see* **5.4.2**), stability of free standing wall shall be ensured such that stability moment of wall due to self-weight equals or exceeds 1.5 times the overturning moment; and

b) *Retaining wall* — Stability for retaining walls shall normally be achieved through gravity action but flexural moment of resistance could also be taken advantage of under special circumstances at the discretion of the designer (*see* **5.4.2**).

Table 3 Thickness and Spacing of Stiffening Walls

[*Clause* 4.2.2.2(b) and G-4.6.2]

Sl No.	Thickness of Load Bearing Wall to be Stiffened	Height[1)] of Storey Not to Exceed	Stiffening Wall[1)]		
			Minimum Thickness		Maximum Spacing
			For 1 to 3 Storeys	For 4 Storeys	
	mm	m	mm	mm	m
(1)	(2)	(3)	(4)	(5)	(6)
i)	100	3.2	100	—	4.5
ii)	200	3.2	100	200	6.0
iii)	300	3.4	100	200	8.0
iv)	Above 300	5.0	100	200	8.0

[1)] Storey height and maximum spacings as given are centre-to-centre dimensions.

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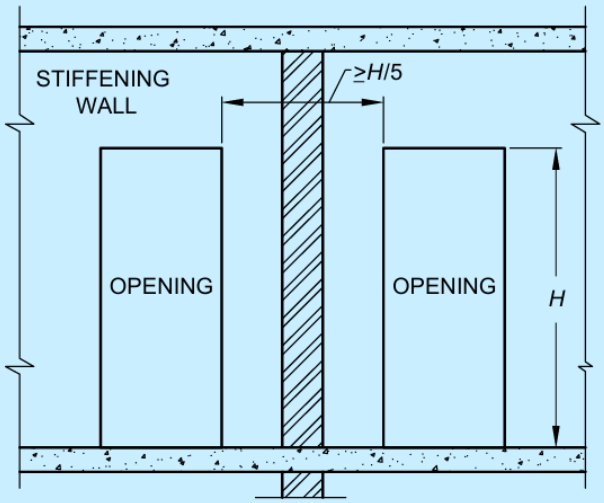

FIG. 9 OPENING IN STIFFENING WALL

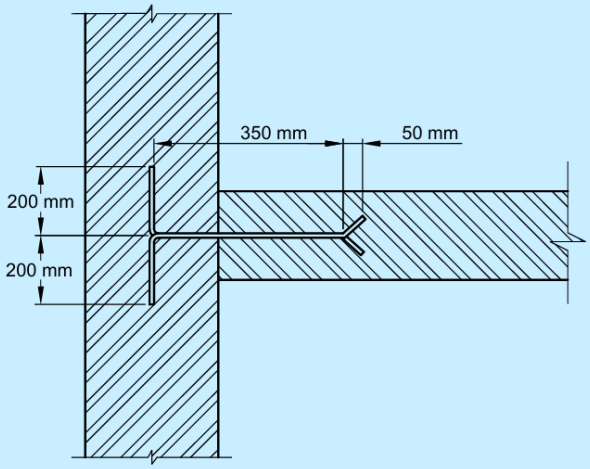

FIG. 10 ANCHORING OF STIFFENING WALL WITH SUPPORT WALL

Table 4 Minimum Thickness of Basement Walls

[*Clause* 4.2.2.6(e)]

Sl No.	Height of the Ground above Basement Floor Level		Minimum Nominal Thickness of Basement Wall
	Wall loading (permanent load) less than 50 kN/m	Wall loading (permanent load) more than 50 kN/m	
(1)	(2)	(3)	(4)
i)	Up to 1.4 m	Up to 1.75 m	300 mm
ii)	Up to 2 m	Up to 2.5 m	400 mm

4.3 Effective Height

4.3.1 *Wall*

Effective height of a wall shall be taken as shown in Table 5 (*see* Fig. 12).

NOTE — A roof truss or beam supported on a column meeting the requirements of **4.2.2.1** is deemed to provide lateral support to the column only in the direction of the beam/truss.

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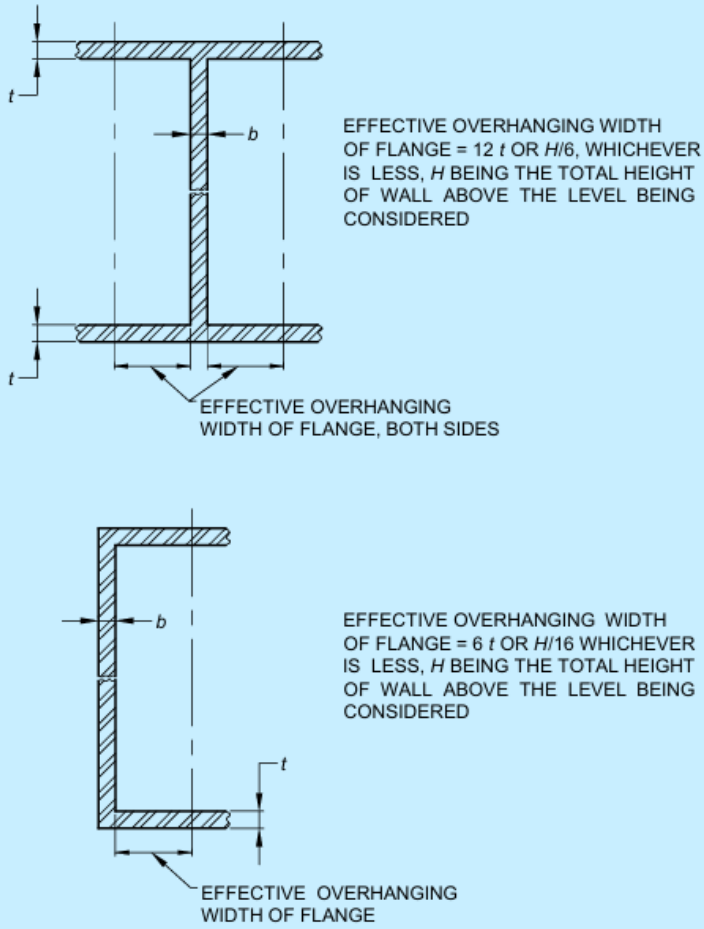

FIG. 11 TYPICAL DETAILS FOR ANCHORAGE OF SOLID WALLS

Table 5 Effective Height of Walls

(*Clause* 4.3.1, 8.7.9.3, Table 22 *and* Table 23)

Sl No.	Condition of Support	Effective Height
(1)	(2)	(3)
i)	Lateral as well as rotational restraint (that is, full restraint) at top and bottom. For example, when the floor/roof spans on the walls so that reaction to load of floor/roof is provided by the walls, or when an RCC floor/roof has bearing on the wall (minimum 90 mm), irrespective of the direction of the span foundation footings of a wall give lateral as well as rotational restraint	0.75 *H*
ii)	Lateral as well as rotational restraint (that is, full restraint) at one end and only lateral restraint (that is, partial restraint) at the other. For example, RCC floor/roof at one end spanning or adequately bearing on the wall and timber floor/roof not spanning on wall, but adequately anchored to it, on the other end	0.85 *H*
iii)	Lateral restraint, without rotational restraint (that is, partial restraint) on both ends. For example, timber floor/roof, not spanning on the wall but adequately anchored to it on both ends of the wall, that is, top and bottom	1.00 *H*
iv)	Lateral restraint as well as rotational restraint (that is, full restraint) at bottom but have no restraint at the top. For example, parapet walls with RCC roof having adequate bearing on the lower wall, or a compound wall with proper foundation on the soil.	1.50 *H*

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Table 5 (*Concluded*)

NOTES

1 *H* is the height of wall between centres of support in case of RCC slabs and timber floors. In case of footings or foundation block, height (*H*) is measured from top of footing or foundation block. In case of roof truss, height (*H*) is measured up to bottom of the tie beam. In case of beam and slab construction, height should be measured from centre of bottom slab to centre of top beam. All these cases are illustrated by means of examples shown in Fig. 12.

2 For working out effective height, it is assumed that concrete DPC, when properly bonded with masonry, does not cause discontinuity in the wall.

3 Where membrane type damp-proof course or termite shield causes a discontinuity in bond, the effective height of wall may be taken to be greater of the two values calculated as follows:

a) Consider *H* from top of footing ignoring DPC and take effective height as 0.75 *H*; and

b) Consider *H* from top of DPC and take effective height as 0.85 *H*.

4 When assessing effective height of walls, floors not adequately anchored to walls shall not be considered as providing lateral support to such walls.

5 When thickness of a wall bonded to a pier is at least two-thirds of the thickness of the pier measured in the same direction, the wall and pier may be deemed to act as one structural element.

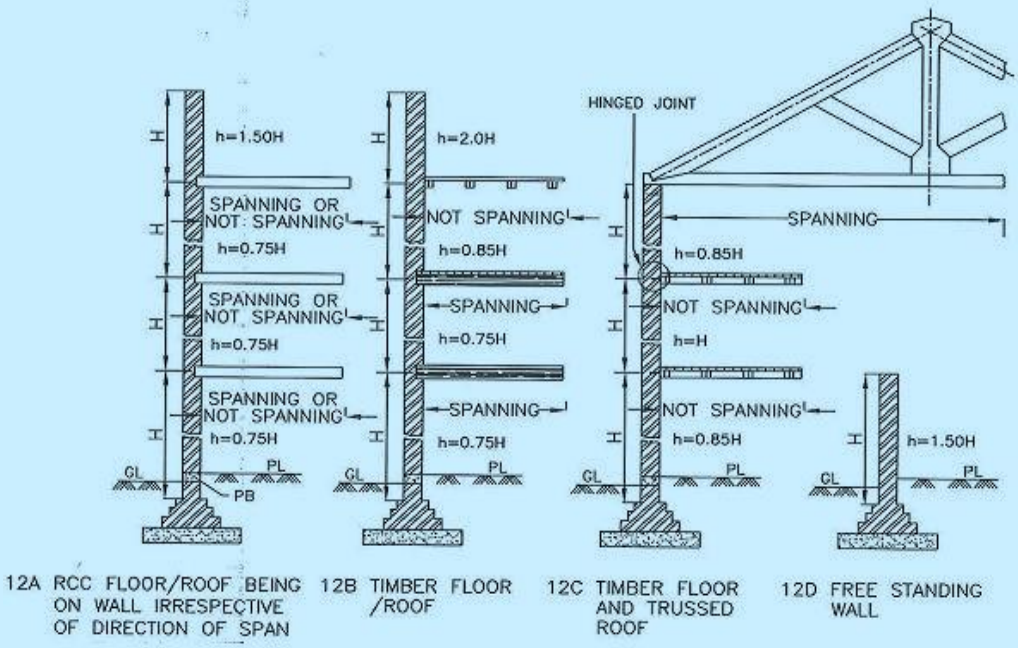
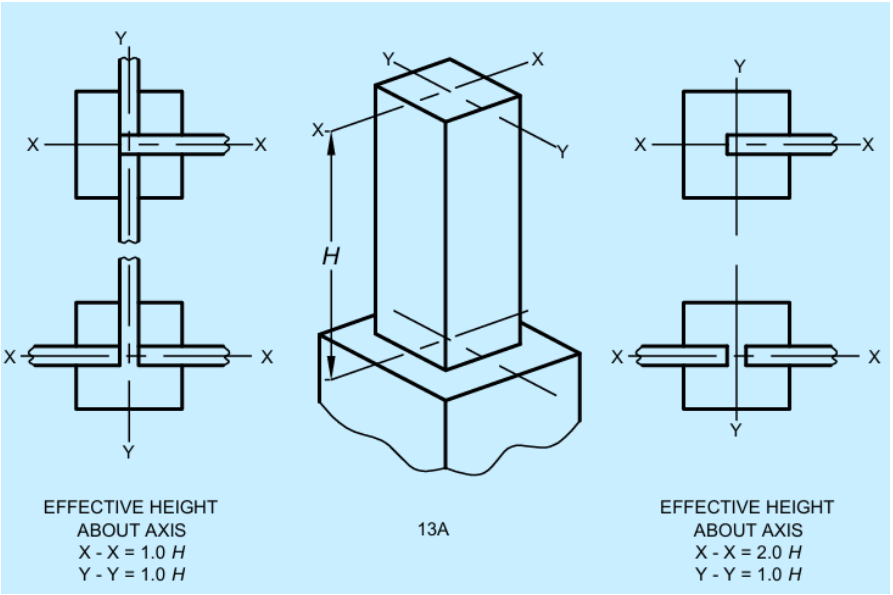


FIG. 12 EFFECTIVE HEIGHT OF WALL

NOTE — If Plinth band (PB) is equal to or more than 150 mm, height can be taken from above the plinth band.



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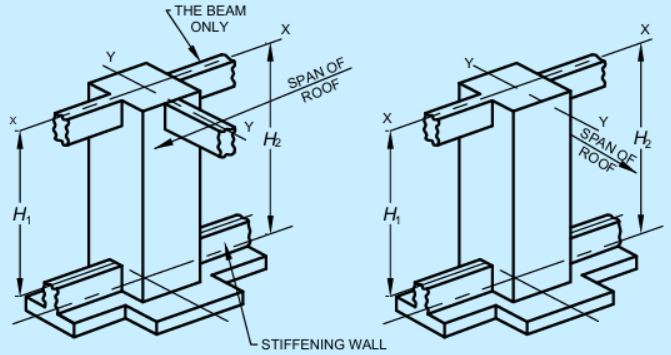

ROOF CONSTRUCTION	EFFECTIVE HEIGHT ABOVE AXIS FIG. 13B	EFFECTIVE HEIGHT ABOVE AXIS FIG. 13C
WITH PRECAST CONCRETE UNITS OF OF IN- SIDE CONCRETE FLOOR	X - X = 1.0 H_2 Y - Y = 1.0 H_1 Y - Y = 1.5 H_1 (NO TIES)	X - X = 1.5 H_2 Y - Y = 1.0 H_1
WITH LIGHT DECK OR SIMILAR ROOF	X - X = 1.0 H_2 Y - Y = 1.0 H_1 Y - Y = 2.0 H_1 (NO TIES)	X - X = 2.0 H_2 Y - Y = 1.0 H_1

FIG. 13 EFFECTIVE HEIGHT OF WALL

4.3.2 *Column*

In case of a column, effective height shall be taken as actual height for the direction it is laterally supported and as twice the actual height for the direction it is not laterally supported (*see* Fig. 13).

NOTES

1 A roof truss or beam supported on a column meeting the requirements of **4.2.2.1** is deemed to provide lateral support to the column only in the direction of the beam/truss.

2 When floor or roof consisting of RCC beams and slabs is supported on columns, the columns would be deemed to be laterally supported in both directions.

4.3.3 *Openings in Walls*

When openings occur in a wall such that masonry between the openings is by definition a column, effective height of masonry between the openings shall be reckoned as follows:

a) *When wall has full restraint at the top*:

1) Effective height for the direction perpendicular to plane of wall equals 0.75 H plus 0.25 H_1, where H is the distance between supports and H_1 is the height of the taller opening; and

2) Effective height for the direction parallel to the wall equals H, that is, the distance between the supports.

b) *When wall has partial restraint at the top and bottom:*

1) Effective height for the direction perpendicular to plane of wall equals H when height of neither opening exceeds 0.5 H and it is equal to 2 H when height of any opening exceeds 0.5 H; and

2) Effective height for the direction parallel to the plane of the wall equals 2 H.

4.4 Effective Length

Effective length of a wall shall be as given in Table 6.

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4.5 Effective Thickness

Effective thickness to be used for calculating slenderness ratio of a wall or column shall be obtained as in **4.5.1** to **4.5.5**.

4.5.1 For solid walls, faced walls or columns, effective thickness shall be the actual thickness.

4.5.2 For solid walls adequately bonded into piers, buttresses, effective thickness for determining slenderness ratio based on effective height shall be the actual thickness of wall multiplied by stiffening coefficient as given in Table 7. No modification in effective thickness, however, shall be made when slenderness ratio is to be based on effective length of walls.

Table 6 Effective Length of Walls

(*Clauses* 4.4)

Sl No.	Conditions of Support (*See* Fig. 14)	Effective Length
(1)	(2)	(3)
i)	Where a wall is continuous and is supported by cross wall and there is no opening within a distance of $H/8$ from the face of cross wall (*see* Fig. 14) or Where a wall is continuous and is supported by piers/buttresses conforming to **4.2.1.2 (b)**	0.8 L
ii)	Where a wall is supported by a cross wall at one end and continuous with cross wall at other end or Where a wall is supported by a pier/buttress at one end and continuous with pier/buttress at other end conforming to **4.2.1.2 (b)**	0.9 L
iii)	Where a wall is supported at each end by cross wall or Where a wall is supported at each end by a pier/buttress conforming to **4.2.1.2 (b)**	1.0 L
iv)	Where a wall is free at one end and continuous with a pier/buttress at the other end or Where a wall is free at one end and continuous with a pier/buttress at the other end conforming to **4.2.1.2 (b)**	1.5 L
v)	Where a wall is free at one end and supported at the other end by a cross wall or Where a wall is free at one end and supported at the other end by a pier/buttress conforming to **4.2.1.2 (b)** where L = length of wall from or between centres of cross wall, piers or buttress; and H = actual height of wall between centres of adequate lateral support.	2.0 L

NOTE — In case there is an opening taller than 0.5 H in a wall, ends of the wall at the opening shall be considered as free. Cross walls shall conform to **4.2.2.2 (d)**.

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4.5.3 For solid walls or faced walls stiffened by cross walls, appropriate stiffening coefficient may be determined from Table 7 on the assumption that the cross walls are equivalent to piers of width equal to the thickness of the cross wall and of thickness equal to three times the thickness of stiffened wall.

4.5.4 For cavity walls with both leaves of uniform thickness throughout, effective thickness shall be taken as two-thirds of the sum of the actual thickness of the two leaves.

4.5.5 For cavity walls with one or both leaves adequately bonded into piers, buttresses or cross walls at intervals, the effective thickness of the cavity wall shall be two-thirds of the sum of the effective thickness of each of the two leaves; the effective thickness of each leaf being calculated using **4.5.1** or **4.5.2** as appropriate.

Table 7 Stiffening Coefficient for Walls Stiffened by Piers, Buttresses or Cross Walls

(*Clauses* 4.5.2 *and* 4.5.3)

Sl No.	Ratio $\frac{S_p}{w_p}$	Stiffening Coefficient		
		$\frac{t_p}{t_w} = 1$	$\frac{t_p}{t_w} = 2$	$\frac{t_p}{t_w} = 3$ or more
(1)	(2)	(3)	(4)	(5)
i)	6	1.0	1.4	2.0
ii)	8	1.0	1.3	1.7
iii)	10	1.0	1.2	1.4
iv)	15	1.0	1.1	1.2
v)	20 or more	1.0	1.0	1.0

where

S_p = centre-to-centre spacing of the pier or cross wall;

t_p = thickness of pier as defined in **2.3** (**b**) (*see* Fig. 1);

t_w = actual thickness of the wall proper (*see* Fig. 1); and

w_p = width of the pier in the direction of the wall or the actual thickness of the cross wall.

NOTE — Linear interpolation between the values given in this table is permissible but not extrapolation outside the limits given.

4.6 Slenderness Ratio

4.6.1 *Walls*

For a wall, slenderness ratio shall be effective height divided by effective thickness or, effective length divided by the effective thickness, whichever is less. In case of a load bearing wall, slenderness ratio shall not exceed that given in Table 8. To reduce the slenderness ratio one and a half brick thick wall shall be up to ground level.

4.6.2 *Columns*

For a column, slenderness ratio shall be taken to be the greater of the ratios of effective heights to the respective effective thickness, in the two principal directions. Slenderness ratio for a load bearing column shall not exceed **12**.

4.7 Eccentricity

Eccentricity of vertical loading at a particular junction in a masonry wall shall depend on factors, such as extent of bearing, magnitude of loads, stiffness of slab or beam, fixity at the support and constructional details at junctions. No exact calculations are possible to make accurate assessment of eccentricity. Extent of eccentricity

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under any particular circumstances has, therefore, to be decided according to the best judgment of the designer. Some guidelines for assessment of eccentricity are given in Annex A.

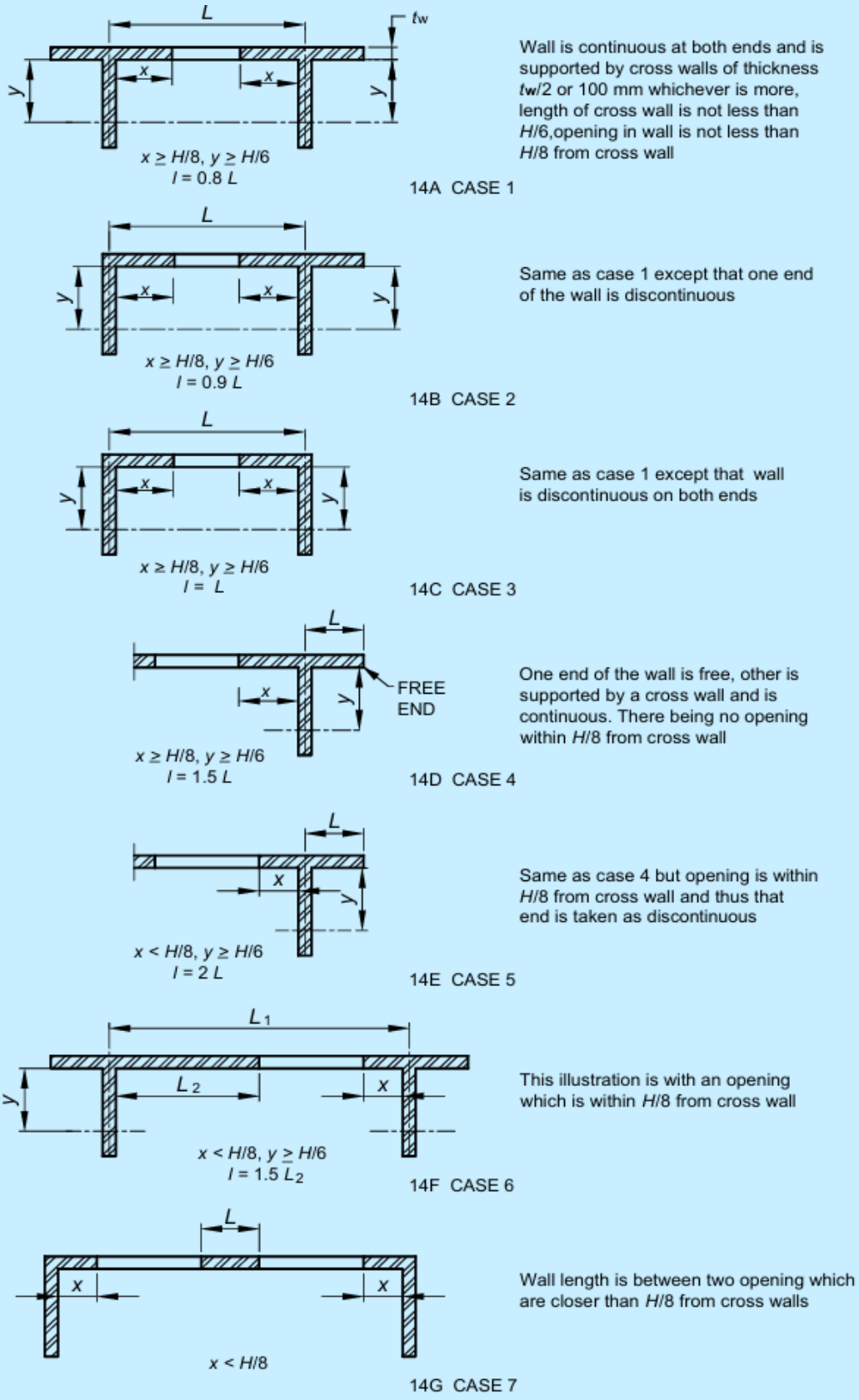

FIG. 14 EFFECTIVE LENGTH OF WALL

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Table 8 Maximum Slenderness Ratio for a Load Bearing Wall

(*Clause* 4.6.1)

Sl No.	Number of Storeys	Maximum Slenderness Ratio	
		Using Portland Cement or Portland Pozzolana Cement in Mortar	Using Lime Mortar
(1)	(2)	(3)	(4)
i)	Not exceeding 2	27	20
ii)	Exceeding 2	27	13

5 STRUCTURAL DESIGN

5.1 General

The building as a whole shall be analysed by accepted principles of mechanics to ensure safe and proper functioning in service of its component parts in relation to the whole building. All component parts of the structure shall be capable of sustaining the most adverse combinations of loads, which the building may be reasonably expected to be subjected to during and after construction.

5.2 Design Loads

Loads to be taken into consideration for designing masonry components of a structure are:

a) dead loads of walls, columns, floors and roofs;

b) imposed loads of floors and roof;

c) wind loads on walls and sloping roof; and

d) seismic forces.

NOTE — When a building is subjected to other loads, such as vibration from railways; machinery, etc, these should be taken into consideration accordingly to the best judgment of the designer (*see also* Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS).

5.2.1 The design loads and other forces to be taken for the design of masonry structures shall conform to Part C/Section 1 of this NBCS.

NOTE — During construction, suitable measures shall be taken to ensure that masonry is not liable to damage or failure due to action of wind forces, back filling behind walls or temporary construction loads.

5.3 Load Dispersion

5.3.1 *General*

The angle of dispersion of vertical load on walls shall be taken as not more than 30° from the vertical.

5.3.2 *Arching Action*

Account may also be taken of the arching action of well-bonded masonry walls supported on lintels and beams, in accordance with established practice. Increased axial stresses in the masonry associated with arching action in this way, shall not exceed the permissible stresses given in **5.4**.

5.3.3 *Lintels*

Lintels that support masonry construction shall be designed to carry loads from masonry (allowing for arching and dispersion where applicable), and loads received from any other part of the structure. Length of bearing of lintel at each end shall not be less than 90 mm or one-tenth of the span, whichever is more, and area of the bearing

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shall be sufficient to ensure that stresses in the masonry (combination of wall stresses, stresses due to arching action and bearing stresses from the lintel) do not exceed the stresses permitted in **5.4** (*see* Annex C).

5.4 Permissible Stresses

5.4.1 *Permissible Compressive Stress*

Permissible compressive stress in masonry shall be based on value of basic compressive stress (f_b) as given in Table 9 and multiplying this value by factors known as stress reduction factor (k_s), area reduction factor (k_a) and shape modification factor (k_p) as detailed in **5.4.1.1** to **5.4.1.3**. Values of basic compressive stress given in Table 9 take into consideration crushing strength of masonry unit and grades of mortar, and hold good for values of slenderness ratio not exceeding 6, zero eccentricity and masonry unit having height to width ratio (as laid) equal to 0.75 or less.

Alternatively, basic compressive stress may be based on results of prism test given in Annex B on masonry made from masonry units and mortar to be actually used in a particular job.

Table 9 Basic Compressive Stresses for Masonry (After 28 days)

(*Clauses* 5.4.1, 5.4.5, 6.3.1 *and Table* 14)

Sl No.	Mortar Type (Ref to, Table 1)	Basic Compressive Stresses, in N/mm², Corresponding to Masonry Units of which Height to Width Ratio does not Exceed 0.75 and Crushing Strength, in N/mm², is not Less than											
		3.5	5.0	7.5	10	12.5	15	17.5	20	25	30	35	40
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
i)	H1	0.35	0.50	0.75	1.00	1.16	1.31	1.45	1.59	1.91	2.21	2.5	3.05
ii)	H2	0.35	0.50	0.74	0.96	1.09	1.19	1.30	1.41	1.62	1.85	2.1	2.5
iii)	M1	0.35	0.50	0.74	0.96	1.06	1.13	1.20	1.27	1.47	1.69	1.9	2.2
iv)	M2	0.35	0.44	0.59	0.81	0.94	1.03	1.10	1.17	1.34	1.51	1.65	1.9
v)	M3	0.25	0.41	0.56	0.75	0.87	0.95	1.02	1.10	1.25	1.41	1.55	1.78
vi)	L1	0.25	0.36	0.53	0.67	0.76	0.83	0.90	0.97	1.11	1.26	1.4	1.06
vii)	L2	0.25	0.31	0.42	0.53	0.58	0.61	0.65	0.69	0.73	0.78	0.85	0.95

NOTES

1 The table is valid for slenderness ratio up to 6 and loading with zero eccentricity.

2 The values given for basic compressive stress are applicable only when the masonry is properly cured.

3 Linear interpolation is permissible for units having crushing strengths between those given in the table.

4 The permissible stress for random rubble masonry may be taken as 75 percent of the corresponding stress for coarsed walling of similar materials.

5 The strength of ashlar masonry (natural stone masonry of massive type with thin joints) is closely related to intrinsic strength of the stone and allowable working stress in excess of those given in the table may be allowed for such masonry at the discretion of the designer.

6 For calculation of basic compressive stress of stabilized soil block having thickness 100 mm or more, reference to specialist literature may be made.

7 If the work is inadequately supervised, strength should be reduced to three-fourth or less at the discretion of the designer.

5.4.1.1 *Stress reduction factor*

This factor, as given in Table 10, takes into consideration the slenderness ratio of the element and also the eccentricity of loading.

5.4.1.2 *Area reduction factor*

This factor takes into consideration smallness of the sectional area of the element and is applicable when sectional area of the element is less than 0.2 m². The factor $k_a = 0.7 + 1.5A$, A being the area of section, in m².

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Table 10 Stress Reduction Factor for Slenderness Ratio and Eccentricity

(*Clause* 5.4.1.1, 9.2.2.3, 9.2.2.9 *and* 10.4.1)

Sl No.	Slenderness Ratio	Eccentricity of Loading Divided by the Thickness of the Member					
		0	1/24	1/12	1/6	1/4	1/3
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	6	1.00	1.00	1.00	1.00	1.00	1.00
ii)	8	0.95	0.95	0.94	0.93	0.92	0.91
iii)	10	0.89	0.88	0.87	0.85	0.83	0.81
iv)	12	0.84	0.83	0.81	0.78	0.75	0.72
v)	14	0.78	0.76	0.74	0.70	0.66	0.66
vi)	16	0.73	0.71	0.68	0.63	0.58	0.53
vii)	18	0.67	0.64	0.61	0.55	0.49	0.43
viii)	20	0.62	0.59	0.55	0.48	0.41	0.34
ix)	22	0.56	0.52	0.48	0.40	0.32	0.24
x)	24	0.51	0.47	0.42	0.33	0.24	—
xi)	26	0.45	0.40	0.35	0.25	—	—
xii)	27	0.43	0.38	0.33	0.22	—	—

NOTES

1 Linear interpolation between values is permitted.

2 Where in special cases the eccentricity of loading lies between 1/3 and 1/2 of the thickness of the member, the stress reduction factor should vary linearly between unity and 0.20 for slenderness ratio of 6 and 20, respectively.

3 Slenderness ratio of a member for sections within 1/8 of the height of the member above or below a lateral support may be taken to be 6.

5.4.1.3 *Shape modification factor*

This factor takes into consideration the shape of the unit, that is, height to width ratio (as laid) and is given in Table 11. This factor is applicable for units of crushing strength up to 15 N/mm^2.

5.4.1.4 *Increase in permissible compressive stresses allowed for eccentric vertical loads, lateral loads under certain conditions*

In members subjected to eccentric and/or lateral loads, increase in permissible compressive stress is allowed as follows:

a) When resultant eccentricity ratio exceeds 1/24 but does not exceed 1/6, 25 percent increase in permissible compressive stress is allowed in design; and

b) When resultant eccentricity ratio exceeds 1/6, 25 percent increase in permissible stress is allowed but the area of the section under tension shall be disregarded for computing the load carrying capacity of the member.

NOTE — When resultant eccentricity ratio of loading is 1/24 or less, compressive stress due to bending shall be ignored and only axial stress need to be computed for the purpose of design.

5.4.1.5 *Increase in permissible compressive stress for walls subjected to concentrated loads*

When a wall is subjected to a concentrated load (a load being taken to be concentrated when area of supporting wall equals or exceeds three times the bearing area), certain increase in permissible compressive stress may be allowed because of dispersal of the load. Since, according to the present state of art, there is diversity of views in regard to manner and extent of dispersal, design of walls subjected to concentrated loads may, therefore, be worked out as per the best judgment of the designer. Some guidelines in this regard are given in Annex C.

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Table 11 Shape Modification Factor for Masonry Units

(*Clause* 5.4.1.3)

Sl No.	Height to Width Ratio of Units (as Laid)	Shape Modification Factor (k_p) For Units Having Crushing Strength in N/mm^2			
		Up to 5.0	7.5	10.0	15.0
(1)	(2)	(3)	(4)	(5)	(6)
i)	Up to 0.75	1.0	1.0	1.0	1.0
ii)	1.0	1.2	1.1	1.1	1.0
iii)	1.5	1.5	1.3	1.2	1.1
iv)	2.0 to 4.0	1.8	1.5	1.3	1.2

NOTE — Linear interpolation between values is permissible.

5.4.2 *Permissible Tensile Stress*

As a general rule, design of masonry shall be based on the assumption that masonry is not capable of taking any tension. However, in case of lateral loads normal to the plane of wall, which causes flexural tensile stress, as for example, panel, curtain partition and free standing walls, flexural tensile stresses as follows may be permitted in the design for masonry:

a) *Grade M1 or better mortar*:

1) 0.07 N/mm^2 for bending in the vertical direction where tension developed is normal to bed joints; and

2) 0.14 N/mm^2 for bending in the longitudinal direction where tension developed is parallel to bed joints, provided crushing strength of masonry units is not less than 10 N/mm^2.

b) *Grade M2 mortar*:

1) 0.05 N/mm^2 for bending in the vertical direction where tension developed is normal to bed joints; and

2) 0.10 N/mm^2 for bending in the longitudinal direction where tension developed is parallel to bed joints, provided crushing strength of masonry units is not less than 7.5 N/mm^2.

NOTES

1 No tensile stress is permitted in masonry in case of water-retaining structures in view of water in contact with masonry. Also no tensile stress is permitted in earth-retaining structures, in view of the possibility of presence of water at the back of such walls.

2 Allowable tensile stress in bending in the vertical direction may be increased to 0.1 N/mm^2 for M1 mortar and 0.07 N/mm^2 for M2 mortar in case of boundary walls/compound at the discretion of the designer, since there is not much risk to life and property in the event of failure of such walls.

5.4.3 *Permissible Shear Stress*

In case of walls built in mortar not leaner than Grade M1 (*see* Table 1) and resisting horizontal forces in the plane of the wall, permissible shear stress calculated on the area of bed joints, shall not exceed the value obtained by the formula given below, subject to a maximum of 0.5 N/mm^2:

$$f_s = 0.1 + f_d/6$$

where

f_s = permissible shear stress, in N/mm^2; and

f_d = compressive stress due to dead loads, in N/mm^2.

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5.4.4 If there is tension in any part of a section of masonry, the area under tension shall be ignored while working out shear stress on the section.

5.4.5 *Specified Compressive Strength*

The specified compressive strength $\boldsymbol{f_m}$ (in MPa) shall be determined by:

a) testing prism specimens as per Annex B; and

b) the expression:

$$f_m = 0.433 f_b^{0.64} f_{m0}^{0.36}$$

where

f_b = compressive strength (in MPa) of clay brick as per Table 9; and

f_{m0} = compressive strength of mortar as per **5.4.1.**

c) as per **5.4.1**.

5.4.6 *Elastic Modulus*

Elastic modulus, E_m of masonry shall be determined by testing masonry prisms as per **5.4.5(a)**; the chord modulus of elasticity shall be taken as the slope of the chord between 0.05 Max. and 0.33 of the Max. compressive strength of each prism:

The expressions:

$$E_m = \begin{cases} 550 f_m & \text{clay brick masonry} \\ 850 f_m & \text{concrete block masonry} \end{cases}$$

5.4.7 *Shear Modulus*

The shear modulus, G_m shall be determined by:

$$G_m = 0.2 E_m$$

5.5 Design Thickness/Cross-Section

5.5.1 *Walls and Columns Subjected to Vertical Loads*

Walls and columns bearing vertical loads shall be designed on the basis of permissible compressive stress. Design consists in determining thickness in case of walls and section in case of columns in relation to strength of masonry units and grade of mortar to be used, taking into consideration various factors, such as slenderness ratio, eccentricity, area of section, workmanship, quality of supervision, etc, subject further to provisions of **5.5.1.1** to **5.5.1.4**.

5.5.1.1 *Solid walls*

Thickness used for design calculation shall be the actual thickness of masonry computed as the sum of the average dimensions of the masonry units specified in the relevant standard, together with the specified joint thickness. In masonry with raked joints, thickness shall be reduced by the depth of raking of joints for plastering/pointing.

5.5.1.2 *Cavity walls*

a) Thickness of each leaf of a cavity wall shall not be less than 75 mm;

b) Where the outer leaf is half masonry unit in thickness, the uninterrupted height and length of this leaf shall be limited so as to avoid undue loosening of ties due to differential movements between the two leaves. The outer leaf shall, therefore, be supported at least at every third storey or at every 10 m of height, whichever is less and at every 10 m or less along the length;

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c) Where the load is carried by both leaves of a wall of a cavity construction, the permissible stress shall be based on the slenderness ratio derived from the effective thickness of the wall as given in 4.5.4 or 4.5.5. The eccentricity of the load shall be considered with respect to the centre of gravity of the cross-section of the wall; and

d) Where the load is carried by one leaf only, the permissible stress shall be the greater of values calculated by the following two alternative methods:

1) The slenderness ratio is based on the effective thickness of the cavity wall as a whole as given in 4.5.4 or 4.5.5 and on the eccentricity of the load with respect to the centre of gravity of the cross-section of the whole wall (both leaves) (This is the same method as where the load is carried by both the leaves but the eccentricity will be more when the load is carried by one leaf only); and

2) The slenderness ratio is based on the effective thickness of the loaded leaf only using 4.5.1 and 4.5.2, and the eccentricity of the load will also be with respect to the centre of gravity of the loaded leaf only.

In either alternative, only the actual thickness of the load bearing leaf shall be used in arriving at the cross-sectional area resisting the load (*see* 5.5.1.1).

5.5.1.3 *Faced wall*

The permissible load per length of wall shall be taken as the product of the total thickness of the wall and the permissible stress in the weaker of the two materials. The permissible stress shall be found by using the total thickness of the wall when calculating the slenderness ratio.

5.5.1.4 *Veneered wall*

The facing (veneer) shall be entirely ignored in calculations of strength and stability. For the purpose of determining the permissible stress in the backing, the slenderness ratio shall be based on the thickness of the backing alone.

5.5.2 *Walls and Columns Mainly Subjected to Lateral Loads*

5.5.2.1 *Free standing walls*

a) Free standing walls, subjected to wind pressure or seismic forces shall be designed on the basis of permissible tensile stress in masonry or stability as in 4.2.2.4. However, in seismic Zones II, free-standing walls may be apportioned without making any design calculations with the help of Table 12 provided the mortar used is of grade not leaner than M1; and

b) If there is a horizontal damp-proof course near the base of the wall, that is, not capable of developing tension vertically, the minimum wall thickness should be the greater of that calculated from either:

1) the appropriate height to thickness ratio given in Table 12 reduced by 25 percent, reckoning the height from the level of the damp-proof course; or

2) the appropriate height to thickness ratio given in Table 12 reckoning the height from the lower level at which the wall is restrained laterally.

Table 12 Height to Thickness Ratio of Free Standing Walls Related to Wind Speed

[*Clause* 5.5.2.1(a) *and* (b)]

Sl No.	Design Wind Pressure N/mm^2	Height to Thickness Ratio
(1)	(2)	(3)
i)	Up to 285	10
ii)	575	7
iii)	860	5
iv)	1 150	4

NOTES

1 For intermediate values, linear interpolation is permissible.

2 Height is to be reckoned from 150 mm below ground level or top of footing/foundation block, whichever is higher, and up to the top edge of the wall.

3 The thickness should be measured including the thickness of the plaster.

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5.5.2.2 *Retaining walls*

Normally masonry of retaining walls shall be designed on the basis of zero-tension, and permissible compressive stress. However, in case of retaining walls for supporting horizontal thrust from dry materials, retaining walls may be designed on the basis of permissible tensile stress at the discretion of the designers.

5.5.3 *Walls and Columns Subjected to Vertical as well as Lateral Loads*

For walls and columns, stress worked out separately for vertical loads as in **5.5.1** and lateral loads as in **5.5.2** shall be combined and elements designed on the basis of permissible stress.

5.5.4 *Walls Subjected to In-Plane Bending and Vertical Loads* (*Shear Walls*)

Walls subjected to in-plane bending and vertical loads, that is, shear walls shall be designed on the basis of no tension with permissible shear stress and permissible compressive stress.

5.5.5 *Non-Load Bearing Walls*

Non-load bearing walls, such as panel walls, curtain walls and partition walls which are mainly subjected to lateral loads, according to present state of art, are not capable of precise design and only approximate methods based on some tests are available. Guidelines for approximate design of these walls are given in Annex D.

6 GENERAL REQUIREMENTS

6.1 Methods of Construction

6.1.1 *General*

Construction of the following types of load bearing and non-load bearing masonry walls shall be carried out in accordance with good practice [C-4(4)]:

a) brickwork;

b) stone masonry;

c) hollow concrete block masonry;

d) gypsum partition blocks;

e) autoclaved cellular concrete block masonry; and

f) lightweight concrete block masonry.

6.1.2 *Construction of Buildings in Seismic Zones*

Special features of construction for earthquake resistant masonry buildings in zones III and above shall be applicable according to **8**.

6.2 Minimum Thickness of Walls from Consideration Other than Structural

Thickness of walls determined from consideration of strength and stability may not always be adequate in respect of other requirements, such as resistance to fire, thermal insulation, sound insulation and resistance to damp penetration for which reference may be made to the appropriate Parts/Sections of the NBCS, and thickness suitably increased, where found necessary.

6.3 Workmanship

6.3.1 *General*

Workmanship has considerable effect on strength of masonry and bad workmanship may reduce the strength of brick masonry to as low as half the intended strength. The basic compressive stress values for masonry as given in Table 9 holds good for commercially obtainable standards of workmanship with reasonable degree of supervision.

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6.3.2 *Bedding of Masonry Units*

Masonry units shall be laid on a full bed of mortar with frog, if any, upward such that cross-joints and wall joints are completely filled with mortar. Masonry units which are moved after initial placement shall be re-laid in fresh mortar, discarding the disturbed mortar.

6.3.3 *Bond*

Cross-joints in any course of one brick thick masonry wall shall be not less than one-fourth of a masonry unit in horizontal direction from the cross-joints in the course below. In masonry walls more than one brick in thickness, bonding through the thickness of wall shall be provided by either header units or by other equivalent means in accordance with good practice [C-4(5)].

6.3.4 *Verticality and Alignment*

All masonry shall be built true and plumb within the tolerances prescribed below; care shall be taken to keep the perpends properly aligned:

a) deviation from vertical within a storey shall not exceed 6 mm per 3 m height;

b) deviation in verticality in total height of any wall of a building more than one storey in height shall not exceed 12.5 mm.;

c) deviation from position shown on plan of any brickwork shall not exceed 12.5 mm;

d) relative displacement between load bearing walls in adjacent storeys intended to be in vertical alignment shall not exceed 6 mm;

e) deviation of bed-joint from horizontal in a length of 12 m shall not exceed 6 mm subject to a maximum deviation of 12 mm; and

f) deviation from the specified thickness of bed joints, cross-joints and perpends shall not exceed one-fifth of the specified thickness.

NOTE — These tolerances have been specified from the point of view of their effect on the strength of masonry. The permissible stress recommended in 5.3 may be considered applicable only, if these tolerances are adhered to.

6.4 Joints to Control Deformation and Cracking

Special provision shall be made to control or isolate thermal and other movements so that damage to the fabric of the building is avoided and its structural sufficiency preserved. Design and installation of joints shall be done according to the appropriate recommendations in accordance with good practice [C-4(6)].

6.5 Chases, Recesses and Holes

6.5.1 Chases, recesses and holes are permissible in masonry only if these do not impair strength and stability of the structure.

6.5.2 In masonry, designed by structural analysis, all chases, recesses and holes shall be considered in structural design and detailed in building plans.

6.5.3 When chases, recesses and holes have not been considered in structural design and are not shown in drawings, these may be provided, subject to the constraints and precautions specified in 6.5.3.1 to 6.5.3.10.

6.5.3.1 As far as possible, services should be planned with the help of vertical chases and use of horizontal chases should be avoided.

6.5.3.2 For load bearing walls, depth of vertical and horizontal chases shall not exceed one-third and one-sixth of the wall thickness, respectively.

6.5.3.3 Vertical chases shall not be closer than 2 m in any stretch of wall and shall not be located within 345 mm of an opening or within 230 mm of a cross wall that serves as a stiffening wall for stability. Width of a vertical chase shall not exceed thickness of wall in which it occurs.

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6.5.3.4 When unavoidable horizontal chases of width not exceeding 60 mm in a wall having slenderness ratio not exceeding 15 may be provided. These shall be located in the upper or lower middle third height of wall at a distance not less than 600 mm from a lateral support. No horizontal chase shall exceed 1 m in length and there shall not be more than 2 chases in any one wall. Horizontal chases shall have minimum mutual separation distance of 500 mm. Sum of lengths of all chases and recesses in any horizontal plane shall not exceed one-fourth the length of the wall.

6.5.3.5 Holes for supporting put-logs of scaffolding shall be kept away from bearings of beams, lintels, and other concentrated loads. If unavoidable, stresses in the affected area shall be checked to ensure that these are within safe limits.

6.5.3.6 No chase, recess or hole shall be provided in any stretch of a masonry wall, the length of which is less than four times the thickness of wall, except when found safe by structural analysis.

6.5.3.7 Masonry directly above a recess or a hole, if wider than 300 mm, shall be supported on a lintel. No lintel, however, is necessary in case of a circular recess or hole exceeding 300 mm in diameter provided upper half of the recess or hole is built as a semi-circular arch of adequate thickness and there is an adequate length of masonry on the sides of openings to resist the horizontal thrust.

6.5.3.8 As far as possible, chases, recesses and holes in masonry should be left (inserting sleeves, where necessary) at the time of construction of masonry so as to obviate subsequent cutting. If cutting is unavoidable, it should be done without damage to the surrounding or residual masonry. It is desirable to use such tools for cutting which depend upon rotary and not on heavy impact for cutting action.

6.5.3.9 No chase, recess or hole shall be provided in half-brick load bearing wall, excepting the minimum number of holes needed for scaffolding.

6.5.3.10 Chases, recesses or holes shall not be cut into walls made of hollow or perforated units, after the units have been incorporated in masonry.

6.6 Corbelling

6.6.1 Where corbelling is required for the support of some structural element, maximum projection of masonry unit should not exceed one-half of the height of the unit or one-half of the built-in part of the unit and the maximum horizontal projection of the corbel should not exceed one-third of the wall thickness.

6.6.2 The load per unit length on a corbel shall not be greater than half of the load per unit length on the wall above the corbel. The load on the wall above the corbel, together with four times the load on the corbel, shall not cause the average stress in the supporting wall or leaf to exceed the permissible stresses given in **5.4**.

6.6.3 It is preferable to adopt header courses in the corbelled portion of masonry from considerations of economy and stability.

7 REINFORCED BRICK AND REINFORCED BRICK CONCRETE FLOORS AND ROOFS

The construction and design of reinforced brick and reinforced brick concrete floors and roof shall be in accordance with good practices [C-4(7)].

8 SPECIAL CONSIDERATION FROM EARTHQUAKE POINT OF VIEW

The design and construction of masonry buildings shall be governed by provisions of **4**, except as modified by the provisions of this section for those elements participating in lateral force resistance. The design and construction for earthquake resistant masonry buildings are given in **8**, **9** and **10**. The earthquake resistant design and detailing of masonry buildings having the following structural systems:

a) Masonry Walls with Prescriptive Bands (MWB);

b) Masonry Walls with Prescriptive Bands and Vertical Reinforcements (MWBR);

c) Confined Masonry Walls (CMW) (*see* **9**); and

d) Reinforced Masonry Walls (RMW) (*see* **10**).

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Buildings with MWB and MWBR systems shall not be of height more than 15 m subject to a maximum of four storeys, measured from the base of the buildings to the roof slab or ridge level.

8.1 For the purpose of **8** and **9**, the following definitions shall apply in addition to those given in **2**.

8.1.1 *Separation Section*

The gap of specified width between adjacent buildings or parts of the same building either left uncovered or covered suitably to permit movement to avoid pounding due to earthquake.

8.1.1.1 *Crumple section*

The separation gap filled with appropriate material that crumples or factures in the event of an earthquake.

8.1.2 *Centre of Rigidity*

The point at the floor level of a structure where application of lateral force in a particular principal direction produces equal deflections of the floor components along that direction.

8.1.3 *Load Bearing Wall*

A wall designed to resist axial force, shear force in its own plane and bending moment about its major axis.

8.1.4 *Box System*

A building made of masonry load bearing walls and horizontal floors.

8.1.5 *Band*

A wooden (in low strength masonry buildings), reinforced concrete or reinforced brick runner provided in the walls to tie them together and to impart horizontal bending strength in them.

8.1.6 *Earthquake Zone*

The earthquake zones as classified in **5** of Part C/Section 1 of this NBCS.

8.1.7 *Design Horizontal Acceleration Coefficient*

The horizontal acceleration coefficient A_h computed considering the importance factor and soil-foundation system as specified in **5** of Part C/Section 1 of this NBCS.

8.1.8 *Concrete Strength*

The 28-day compressive strength (in MPa) of concrete cubes of 150 mm size.

8.1.9 *Cross-Sectional Area of Masonry Unit*

The net cross-sectional area of a masonry unit shall be taken as the gross cross-sectional area minus the area of cellular space (if any). Gross cross-sectional area of cored units shall be determined to the outside of the coring, but cross-sectional area of grooves (if any) shall not be deducted from the gross cross-sectional area to obtain the net cross-sectional area.

8.1.10 *Grout*

A mixture of cement, sand, and water of pourable consistency for filling small voids.

8.1.11 *Grouted Masonry*

Masonry in which the masonry units with holes are filled with a mixture of cement and sand.

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8.1.11.1 *Grouted hollow-unit masonry*

A form of grouted masonry construction in which certain designated cells of hollow units are continuously filled with grout.

8.1.11.2 *Grouted multi-wythe masonry*

A form of grouted masonry construction in which the space between the wythes is solidly or periodically filled with grout.

8.1.12 *Joint Reinforcement*

A prefabricated reinforcement in the form of lattice truss which has been hot dip galvanized after fabrication and is to be laid in the mortar bed joint.

8.1.13 Pier

An isolated vertical member whose horizontal dimension measured at right angles to its thickness is not less than 4 times its thickness and whose height is less than 5 times its length.

8.1.14 *Prism*

An assemblage of masonry units bonded by mortar with or without grout used as a test specimen for determining properties of masonry.

8.1.15 *Reinforced Masonry*

Masonry which is reinforced (as specified in 2) and grouted (if necessary) so that the two materials act together in resisting forces.

8.1.16 *Grouted Cavity Reinforced Masonry*

Masonry with two parallel single-leaf walls spaced at least 50 mm apart, and effectively tied together with wall ties. The intervening cavity contains steel reinforcement and is filled with infill concrete so as to result in common action with masonry under load.

8.1.17 *Pocket type Reinforced Masonry*

Masonry reinforced primarily to resist lateral loading where the main reinforcement is concentrated in vertical pockets formed in the tension face of the masonry and is surrounded by in situ concrete.

8.1.18 *Quetta Bond Reinforced Masonry*

Masonry that is at least one and half units thick, in which vertical pockets containing reinforcement and mortar or concrete infill occur at intervals along its length.

8.1.19 *Specified Compressive Strength of Masonry*

Minimum compressive strength (in MPa) is force per unit of net cross-section area, which is required of the masonry used in construction and upon which the design is based.

8.1.20 *Wall Tie*

A metal fastener, which connects wythes of masonry to each other or to other materials.

8.1.21 *Wythe*

A continuous vertical layer of masonry wall of one unit in thickness.

NOTE — For low strength masonry buildings, wooden runners may also be used as runner.

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8.2 General Principles

The general principles given in 8.2.1 to 8.2.9 shall be observed in construction of earthquake resistant buildings.

8.2.1 *Lightness*

Since the earthquake force is a function of mass, the weight of the building shall be as minimum as possible, consistent with structural safety and functional requirements. Roofs and upper storeys of buildings, in particular, should be designed as light as possible.

8.2.2 *Continuity of Construction*

8.2.2.1 As far as possible, the parts of the building should be tied together in such a manner that the building acts as one unit.

8.2.2.2 For parts of buildings between separation or crumple sections to expansion joints, floor slabs shall be continuous throughout as far as possible. Concrete slabs shall be rigidly connected or integrally cast with the support beams.

8.2.2.3 Additions to the structures shall be accompanied by the provision of separation or crumple sections between the new and the existing structures as far as possible, unless positive measures are taken to establish continuity between the existing and the new construction.

8.2.2.4 Alteration to the building structure shall be done by maintaining its structural stability by ensuring proper load path.

8.2.3 *Projecting and Suspended Parts*

8.2.3.1 Projecting parts shall be avoided as far as possible. If the projecting parts cannot be avoided, they shall be properly reinforced and firmly tied to the main structure, and their design shall be in accordance with **5** of Part C/Section 1 of this NBCS.

8.2.3.2 Ceiling plaster shall preferably be avoided. When it is unavoidable, the plaster shall be as thin as possible.

8.2.3.3 Suspended ceiling shall be avoided as far as possible. Where provided they shall be light, adequately framed and secured to which electrical fixtures shall be fully secured.

8.2.4 *Building Configuration*

8.2.4.1 Masonry buildings shall have walls placed uniformly along the two principal plan directions. Buildings with the irregularities listed in **5** of Part C/Section 1 of this NBCS perform poorly during earthquake shaking, and should be avoided. When these irregularities appear in a building, the guidance given in **5** of Part C/Section 1 of this NBCS for the respective irregularity shall be followed. In order to minimize torsion and stress concentration, provisions given in 8.2.4.2 to 8.2.4.4 should be complied with as relevant.

8.2.4.2 The building should have a simple rectangular plan and be symmetrical both with respect to mass and rigidity so that the centre of mass and rigidity of the building coincide with each other in which case no separation sections other than expansion joints are necessary. For provision of expansion joints reference may be made to [C-4(6)].

8.2.4.3 If symmetry of the structure is not possible in plan, elevation or mass, provision shall be made for torsional and other effects due to earthquake forces in the structural design or the parts of different rigidities may be separated through crumple sections. The length of such building between separation sections shall not preferably exceed three times the width.

NOTE — As an alternative to separation section to reduce torsional moments, the centre of rigidity of the building may be brought close or coincident to the centre of mass by adjusting the locations and/or sizes of columns and walls.

8.2.4.4 Buildings having plans with shapes like L, T, E and Y shall preferably be separated into rectangular parts by providing separation sections at appropriate places. Typical examples are shown in Fig. 15.

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NOTES

1 The buildings with small lengths of projections forming L, T, E or Y shapes need not be provided with separation section. In each cases the length of the projection may not exceed 15 percent to 20 percent of the total dimension of the building in the direction of the projection (*see* Fig. 16).

2 For buildings with minor asymmetry in plan and elevation, separation sections may be omitted.

8.2.5 *Strength in Various Directions*

The structure shall be designed to have adequate strength against earthquake effects along both the horizontal axes. The design shall also be safe considering the reversible nature of earthquake forces.

8.2.6 *Foundations*

The structure shall not be founded on such soils which shall subside or liquefy during an earthquake, resulting in large differential settlements (*see also* **8.3.3.3**).

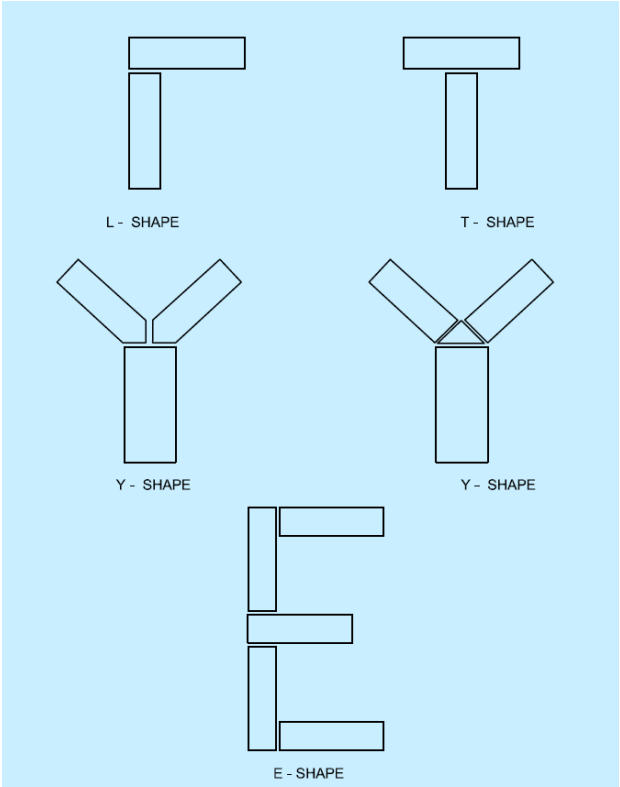

FIG. 15 TYPICAL SHAPES OF BUILDING WITH SEPARATION SECTIONS

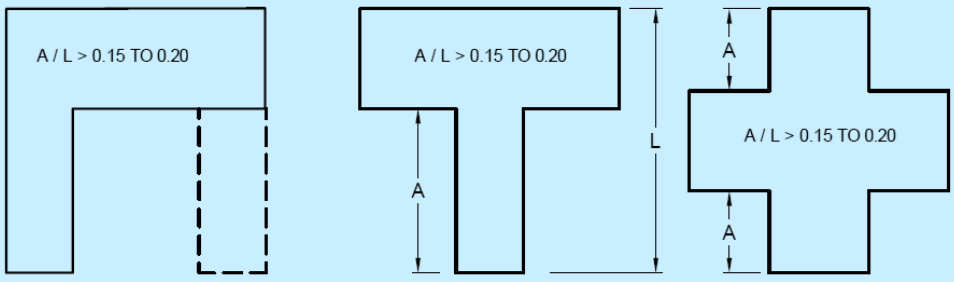

FIG. 16 (A) PLAN IRREGULARITIES

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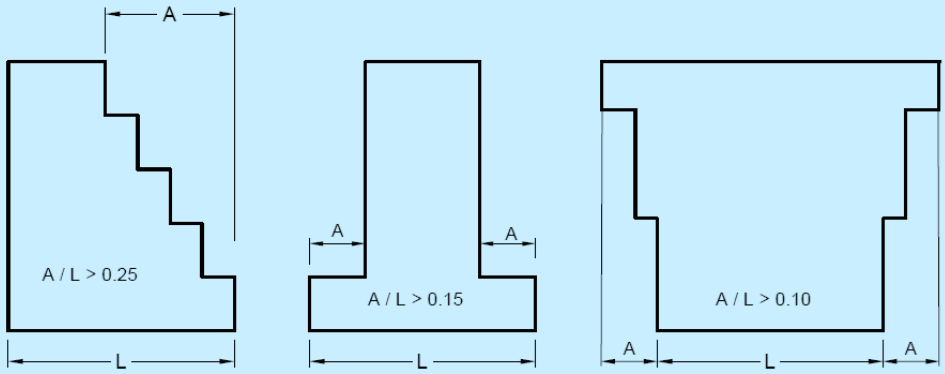

FIG. 16(B) VERTICAL IRREGULARITIES

FIG. 16 PLAN AND VERTICAL IRREGULARITIES

8.2.7 *Ductility*

The main structural elements and their connection shall be designed to have a ductile failure. This shall enable the structure to absorb energy during earthquakes to avoid sudden collapse of the structure. Providing reinforcing steel in masonry at critical sections, as specified in this Section shall not only increase strength and stability but also ductility. The details for achieving ductility in reinforced concrete structures is given in Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS.

8.2.8 *Damage to Non-Structural Parts*

Suitable details shall be worked out to connect the non-structural parts with the structural framing so that the deformation of the structural frame leads to minimum damage of the non-structural elements.

8.2.9 *Fire Safety*

Fire frequently follows an earthquake and therefore, buildings shall be constructed to make them fire resistant in accordance with the provisions of Part F 'Fire and Life Safety' of this NBCS.

8.3 Special Construction Features

8.3.1 *Separation of Adjoining Structures*

8.3.1.1 Separation of adjoining structures or parts of the same structure is required for structures having different total heights or storey heights and different dynamic characteristics. This is to avoid collision during an earthquake.

8.3.1.2 Minimum width of separation gaps as mentioned in **8.3.1.1**, shall be as specified in Table 13. The design seismic coefficient to be used shall be in accordance with **5** of Part C/Section 1 of this NBCS.

8.3.1.3 Where separation is necessary, a complete separation of the parts shall be made except below the plinth level. The plinth beams, foundation beams and footings may be continuous. Where separation sections are provided in a long building, they shall take care of movement owing to temperature changes also.

8.3.2 *Separation or Crumple Section*

8.3.2.1 In case of framed construction, members shall be duplicated on either side of the separation or crumple section. As an alternative, in certain cases, such duplication may not be provided, if the portions on either side can act as cantilevers to take the weight of the building and other relevant loads.

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8.3.2.2 Typical details of separation and crumple sections are shown in Fig. 17. For other types of joint details, reference may be made to [C-4(5)].

Table 13 Gap Width for Adjoining Structures

(*Clause* 8.3.1.2)

Sl No.	Buildings Categories for Earthquake Resisting Feature	Gap Width per Storey for Adjoining Masonry Building
		mm
(1)	(2)	(3)
i)	Frames with shear walls	15
ii)	Category B[1)]	10
iii)	Category C[1)]	10
iv)	Category D[1)]	15
v)	Category E[1)]	20

NOTE — Minimum total gap shall be 25 mm. For any other value of A_h the gap width shall be determined proportionately.

[1)] *See* Table 14.

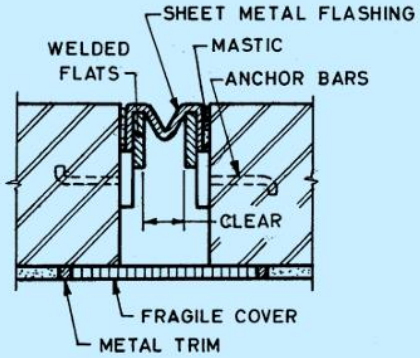

FIG.17(A) BRICK WALL

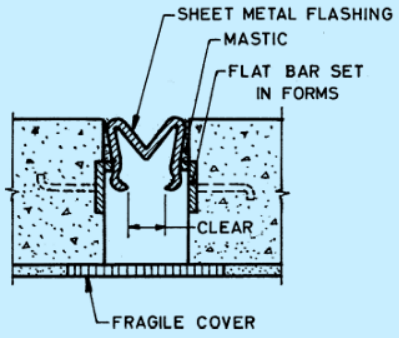

FIG. 17(B) CONCRETE WALLS

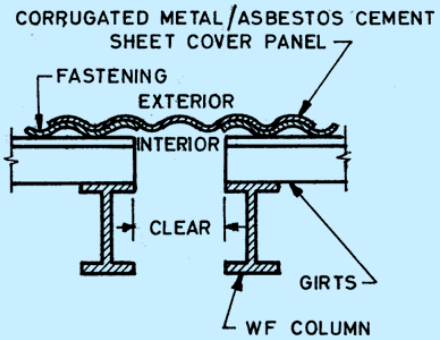

FIG. 17(C) METAL SLIDING INDUSTRIAL WORK

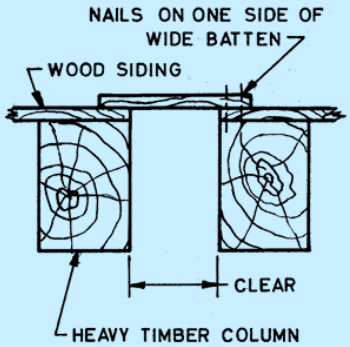

FIG. 17(D) WOOD SHEATHING INDUSTRIAL WORK

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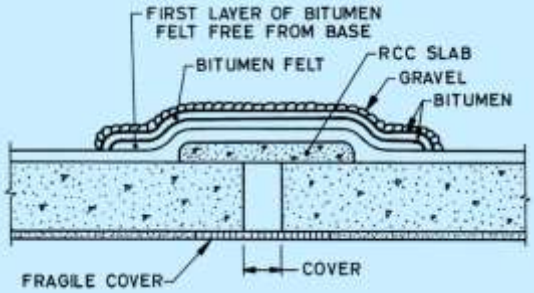

FIG.17(E) RCC SLAB ON ROOF SURFACE

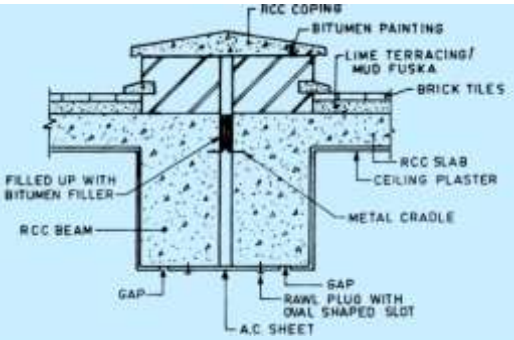

FIG. 17(F) SEPARATION JOINT DETAILS AT ROOF

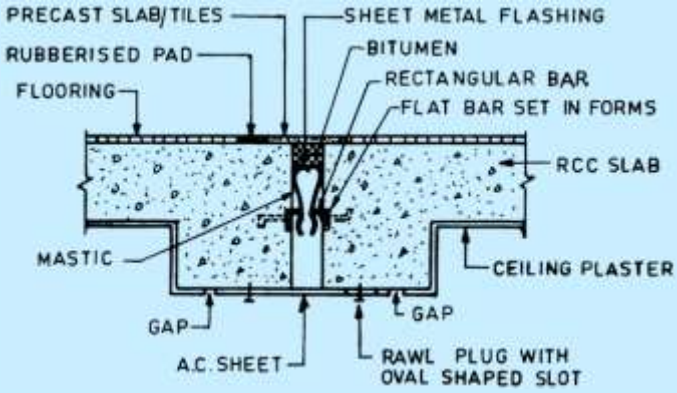

FIG.17(G) SEPARATION AT FLOOR LEVEL

FIG. 17 TYPICAL DETAILS OF SEPARATION OR CRUMPLE SECTION

8.3.3 *Foundations*

8.3.3.1 For the design of foundations, the provisions of Part C/Section 1 and Section 2 of this NBCS shall generally be followed.

8.3.3.2 The subgrade below the entire area of the building shall preferably be of the same type of the soil. Wherever this is not possible, a suitably located separation or crumple section shall be provided.

8.3.3.3 Loose fine sand, soft silt and expansive clays should be avoided. If unavoidable, the building shall rest either on a rigid raft foundation or on piles taken to a firm stratum. However, for light constructions the following measures may be taken to improve the soil on which the foundation of the building may rest:

a) sand piling; and

b) soil stabilization.

8.3.3.4 *Isolated footings for columns*

All the individual footings or pile caps where used in Type III Soft soils, shall be connected by reinforced concrete ties at least in two directions approximately at right angles to each other. For buildings with no basement, the ties may be placed at or below the plinth level and for buildings with basement they may be placed at the level of basement floor. They shall need to be designed to carry the load of the panel walls also.

NOTE — The ties may not be necessary where structural floor connects the columns at or below the plinth level.

8.3.3.4.1 Where ties are used, their sections shall be designed to carry in tension as well as in compression, an axial load not less than the earthquake force in the direction the tie acting on the heavier of the columns connected, but the sections shall not be less than 200 mm × 200 mm with M20 concrete reinforced with 4 bars of 12 mm diameter plain mild steel bars or 10 mm diameter high strength deformed bars, one at each corner, bound by 8 mm diameter stirrups not more than 150 mm apart.

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NOTE — In working out the buckling strength of ties, the lateral support provided by the soil may be taken into account. Calculations show that for such buried ties, lateral buckling is not a problem and the full section of the tie may be taken effective as a short column.

8.3.3.4.2 In the case of reinforced concrete slab, the thickness shall not be less than 1/50th of the clear distance between the footings, but not less than 100 mm in any case. It shall be reinforced with not less than 0.15 percent mild steel bars or 0.12 percent high strength deformed bars in each direction placed symmetrically at top and bottom.

8.3.4 *Roofs and Floors*

8.3.4.1 Flat roof or floor shall not preferably be made of tiles of ordinary bricks supported on steel, timber or reinforced concrete joists, nor they shall be of a type which in the event of an earthquake is likely to be loosened and parts or all of which may fall. If this type of construction cannot be avoided, the joists should be blocked at ends and bridged at intervals such that their spacing is not altered during an earthquake.

8.3.4.1.1 For pitched roofs, corrugated iron or asbestos sheets shall be used in preference to country, Allahabad or Mangalore tiles or other loose roofing units. All roofing materials shall be properly tied to the supporting members. Heavy roofing materials shall generally be avoided.

8.3.4.2 *Pent roofs*

8.3.4.2.1 All roof trusses shall be supported on reinforced concrete or reinforced brick band (*see* **8.5**). The holding down bolts shall have adequate size and length as required for earthquake forces in accordance with **5** of Part C/Section 1 of this NBCS.

Where a trussed roof adjoins a masonry gable, the ends of the purlins shall be carried on and secured to a plate or bearer which shall be adequately bolted to reinforced concrete or reinforced brick band at the top of gable end masonry.

NOTE — Hipped roof in general have shown better structural behaviour during earthquakes than gable ended roofs.

8.3.4.2.2 At tie level all the trusses and the gable end shall be provided with diagonal braces in plan so as to transmit the lateral shear due to earthquake force to the gable walls acting as shear walls at the ends as specified in **8.5**.

8.3.4.3 *Jack arches*

Jack arched roofs or floors, where used shall be provided with mild steel ties in all spans along with diagonal braces in plan to ensure diaphragms actions.

8.3.5 *Staircases*

8.3.5.1 The inter-connection of the stairs with the adjacent floors should be appropriately treated by providing sliding joints at the stairs to eliminate their bracing effect on the floors. Large stair halls shall preferably be separated from the rest of the building by means of separation or crumple sections.

8.3.5.2 Three types of stair construction may be adopted as described below:

a) *Separated staircases* — One end of the staircase rests on a wall and the other end is carried by columns and beams which have no connection with the floors. The gap at the vertical joints between the floor and the staircase may be covered either with a tread plate attached to one side of the joint and sliding on the other side, or covered with some appropriate material which could crumple or fracture during an earthquake without causing structural damage. The supporting members, columns or walls, are isolated from the surrounding floors by means of separation or crumple sections. A typical example is shown in Fig. 18;

b) *Built-in staircase* — When stairs are built monolithically with floors, they can be protected against damage by providing rigid walls at the stair opening. An arrangement, in which the staircase is enclosed by two walls, is given in Fig. 19. In such cases, the joints, as mentioned in respect of separated staircases, shall not be necessary;

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The two walls mentioned above, enclosing the staircase, shall extend through the entire height of the stairs and to the building foundations; and

c) *Staircases with sliding joints* — In case it is not possible to provide rigid walls around stair openings for built-in staircase or to adopt the separated staircases, the staircases shall have sliding joints so that they shall not act as diagonal bracing.

8.3.6 *Framing of Thin Load Bearing Walls* (*see* Fig. 20)

Load bearing walls can be made thinner than 200 mm say 150 mm inclusive of plastering on both sides. Reinforced concrete framing columns and collar beams shall be necessary to be constructed to have full bond with the walls. Columns are to be located at all corners and junctions of walls and spaced not more than 1.5 m apart but so located as to frame up the doors and windows. The horizontal bands or ring beams are located at all floors, roof as well as lintel levels of the openings. The sequence of construction between walls and columns shall be first to build the wall up to 4 to 6 courses height leaving toothed gaps (tooth projection being about 40 mm only) for the columns and second to pour M20 (1 : 1½ : 3) concrete to fill the columns against the walls using wood forms only on two sides. The columns steel should be accurately held in position all along. The band concrete should be cast on the wall masonry directly so as to develop full bond with it.

Such construction may be limited to only two storeys maximum in view of its vertical load carrying capacity. The horizontal length of walls between cross walls shall be restricted to 7 m and the storey height to 3 m.

8.3.7 *Reinforcing Details for Hollow Block Masonry*

The following details may be followed in placing the horizontal and vertical steel in hollow block masonry using cement-sand or cement-concrete blocks.

8.3.7.1 *Horizontal band*

U-shaped blocks may be used for construction of horizontal bands at various levels of the storeys as shown in Fig. 21, where the amount of horizontal reinforcement shall be taken 25 percent more than that given in Table 18 and provided by using four bars and 6 mm diameter stirrups. Other continuity details shall be followed, as shown in Fig. 28.

NOTES

1 Detailing of steel around the window openings will be same as for door openings with additional 75 mm band at sill level.

2 Figures are for just explanation purposes and the same shall not be used as design guide.

8.3.7.2 *Vertical reinforcement*

Bars, as specified in Table 19 shall be located inside the cavities of the hollow blocks, one bar in each cavity (*see* Fig. 29). Where more than one bar is planned these can be located in two or three consecutive cavities. The cavities containing bars are to be filled by using micro-concrete 1 : 2 : 3 or cement coarse sand mortar 1 : 3, and properly rodded for compaction. The vertical bars should be spliced by welding or overlapping for developing full tensile strength. For proper bonding, the overlapped bars should be tied together by winding the binding wire over the lapped length. To reduce the number of overlaps, the blocks may be made U-shaped as shown in Fig. 29 which shall avoid lifting and threading of bars into the hollows.

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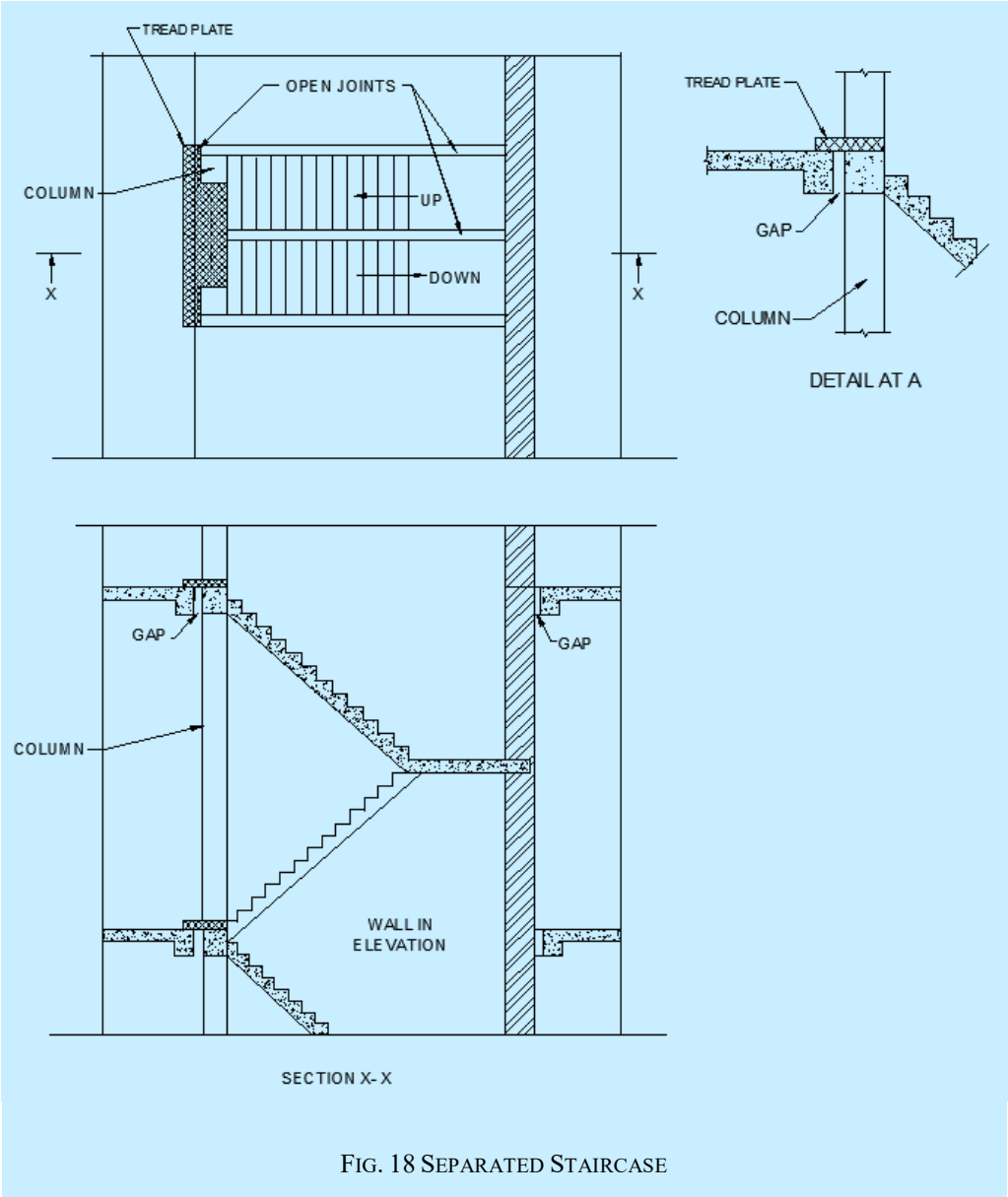

FIG. 18 SEPARATED STAIRCASE

8.4 DESIGN CONSIDERATIONS

8.4.1 *Design Philosophy*

The design approach adopted for masonry buildings is the working stress method at serviceability loads, with structural materials assumed to behave in linear elastic manner. Safety is ensured by restricting stresses in material induced by design loads imposed on the structure as per the load combinations specified in **5** of Part C/Section 1 of this NBCS.

8.4.1.1 *Working stress method*

In the working stress method of design of members, permissible stresses given in Table 14 shall govern the earthquake resistant design of members of the masonry building systems specified in this section.

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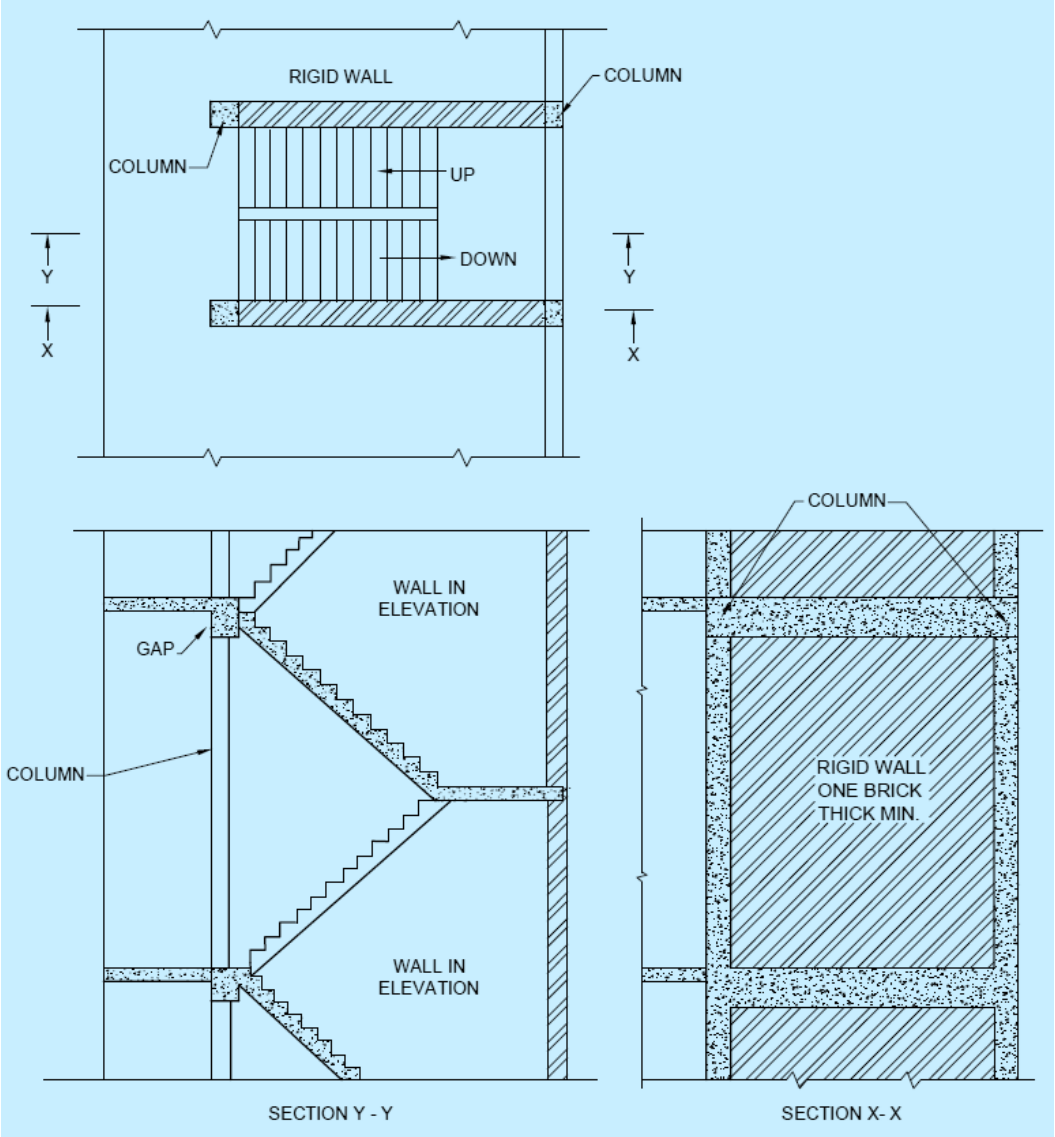

FIG. 19 RIGIDLY BUILT-IN STAIRCASE

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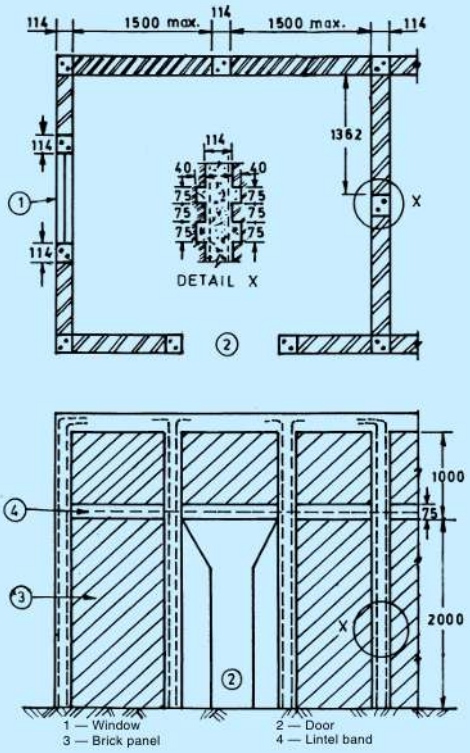

All dimensions in millimeters.

FIG. 20 FRAMING OF THIN LOAD-BEARING BRICK WALLS

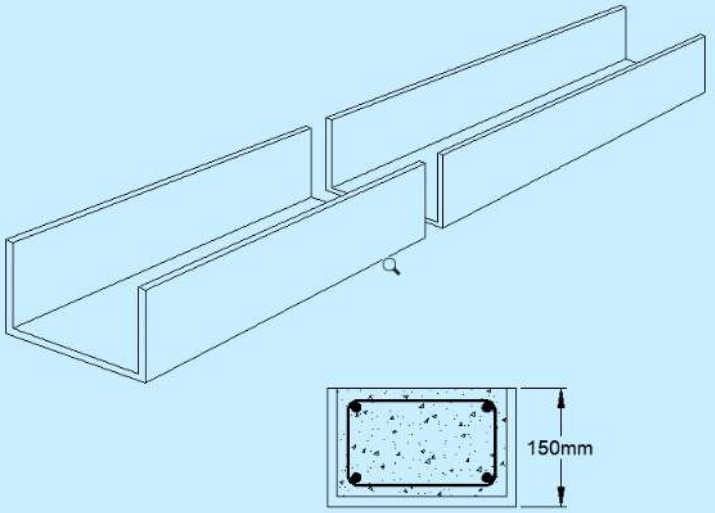

FIG. 21 U - BLOCKS FOR HORIZONTAL BANDS

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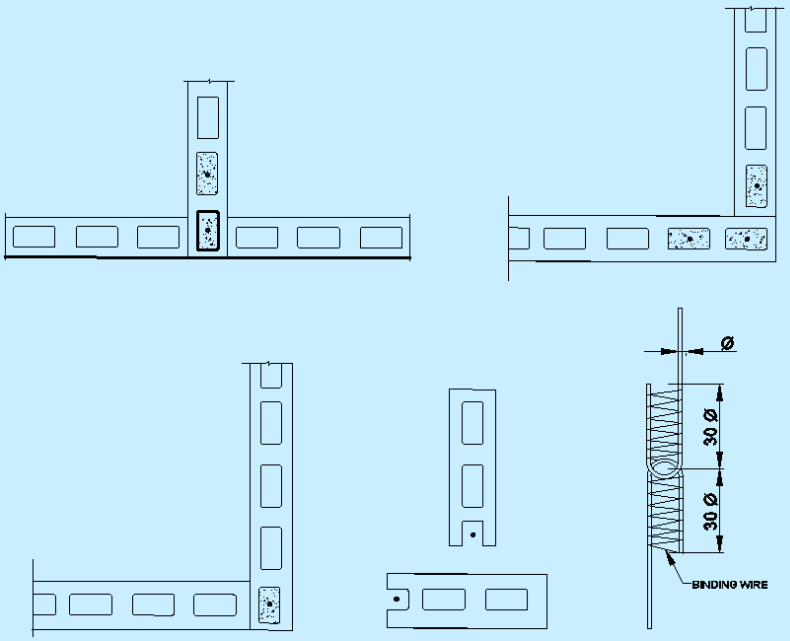

FIG. 22 VERTICAL REINFORCEMENT IN CAVITIES

Table 14 Permissible Stresses in Materials to be Used in the Design of Members of Masonry Buildings

(*Clauses* 8.4.1.1, 11.5 *and* 11.6.1)

Sl No.	Compression		Tension	Shear
	Axial	Axial and Flexural	Axial and Flexural	
(1)	(2)	(3)	(4)	(5)
Masonry Wall with Prescriptive Bands (MWB) Masonry Wall with Prescriptive Bands and Vertical Reinforcements (MWBR)				
i)	5.4.1 and Table 9 apply	Increase in permissible compressive stress permitted as per 5.4.1.4 applies	5.4.2 applies. Masonry walls subjected to in-plane bending and vertical loads shall be designed based on no-tension, permissible shear, and compressive stresses	5.4.3 applies.
Confined Masonry Wall (CMW)				
ii)	Compressive force in confined masonry due to axial load shall not exceed: $P_0 = k_s(0.25f_mA_m + 0.65f_sA_t + f_{c0}A_c)$	Compressive stress in masonry due to combined action of axial load and bending shall not exceed $1.25\ f_a$	5.4.2 applies.	Allowable shear stress shall be according to 5.4.3
Reinforced Masonry Wall (RMW)				
iii)	Compressive force due to axial load shall not exceed: $P_0 = k_s(0.25f_mA_m + 0.65f_sA_t)$	Compressive stress in masonry due to combined action of axial load and bending shall not exceed $1.25f_a$	5.4.2 applies.	For flexural members: a) without web reinforcement: $f_v = 0.083\sqrt{f_m} \leq 0.75\text{MPa}$

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Table 14 (*Concluded*)

Table 14 Permissible Stresses in Materials to be Used in the Design of Members of Masonry Buildings

(*Clauses* 8.4.1.1, 11.5 *and* 11.6.1)

Sl No.	Compression		Tension	Shear
	Axial	Axial and Flexural	Axial and Flexural	
(1)	(2)	(3)	(4)	(5)
				b) with web reinforcement: $f_v = 0.25\sqrt{f_m} \leq 0.75\text{MPa}$ For walls, shall be as per **5.4.3**

8.4.2 Bands and Vertical Elements

Masonry buildings, other than those with reinforced masonry walls and confined masonry walls, shall be provided with horizontal bands and vertical reinforcing elements:

a) the horizontal bands shall run along the full perimeter of the external and internal walls, within the full thickness of the walls; and

b) the vertical reinforcing elements shall pass through vertically at all corners and adjoining all openings (door, window, and ventilator).

8.4.2.1 *Types*

Five types of bands are admissible, namely:

a) Gable band (when masonry buildings have gable walls);

b) Roof band;

c) Lintel band;

d) Sill band; and

e) Plinth band.

8.4.2.2 *Geometry*

The width of RC band shall be same as the thickness of the wall. Wall thickness shall be at least 200 mm. A clear cover of 20 mm shall be maintained for the reinforcement.

The vertical thickness of RC Band shall be at least:

a) 75 mm, where two longitudinal bars are specified, one on each face; and

b) 150 mm, where four bars are specified, one at each corner.

8.4.2.3 *Ductile detailing*

The bands shall be made of reinforced concrete of grade not leaner than M25 or reinforced brickwork in cement mortar not leaner than 1 : 3. In coastal areas, the filling mortar with water proofing admixture shall be used.

8.4.3 *Floor Slabs*

The roof and floor slabs in MWBs, MWBR, CMBs and RMBs shall be designed as beam supported slabs as per the provisions of Part C/Section 5 of this NBCS.

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8.4.4 *Building Structure Type*

Masonry buildings shall be categorized into four types (Table 15), namely Types B, C, D and E, based on the earthquake and the category of the building and the importance of building I, where I is importance factor applicable to the building. Type E buildings shall not be more than 3 storeys tall.

Table 15 Building Categories of Building for Earthquake Resisting Features

(*Clause* 8.4.4)

Sl No.	Importance Factor	Earthquake Zone			
		II	III	IV	V
(1)	(2)	(3)	(4)	(5)	(6)
i)	1.0	*B*	*C*	*D*	*E*
ii)	1.5	C	D	*E*	*E*

8.4.4.1 *Slenderness of walls*

The masonry walls shall meet the slenderness limit specified in Table 16.

Table 16 Allowable Effective Height to Effective Thickness Ratios of Unreinforced Masonry Walls

(*Clause* 8.4.4.1)

Sl No.	Wall	Building Structure Type			
		B	C	D	E
(1)	(2)	(3)	(4)	(5)	(6)
i)	MWB and MWBR	15	15	12	12
ii)	CMW	15	15	15	15
iii)	RMW	15	15	15	15
iv)	URM parapet walls	12	12	6	6

8.4.5 *Footings*

Strip footings under load bearing walls shall be designed as per the provisions of [C-4(8)] in conjunction with Part C/Section 1 of this NBCS.

8.5 Masonry Walls with Prescriptive Bands (MWB), and Masonry Walls with Prescriptive Bands and Vertical Reinforcement (MWBR)

8.5.1 *Masonry Walls*

8.5.1.1 The bearing walls in both directions shall be straight and symmetrical in plan.

8.5.1.2 The wall panels formed between cross walls and floors, or roof shall be checked for their strength in bending as a plate or as a vertical strip subjected to the earthquake force acting on its own mass. This check need not be complied with in walls of 200 mm or larger thickness and of storey height less than 3.5 m

8.5.1.3 *Masonry bond*

For achieving full strength of masonry, the usual bonds specified for masonry should be followed so that the vertical joints are broken properly from course to course. To obtain full bond between perpendicular walls, it is necessary to make a slopping (stepped) joint by making the corners first to a height of 600 mm and then building

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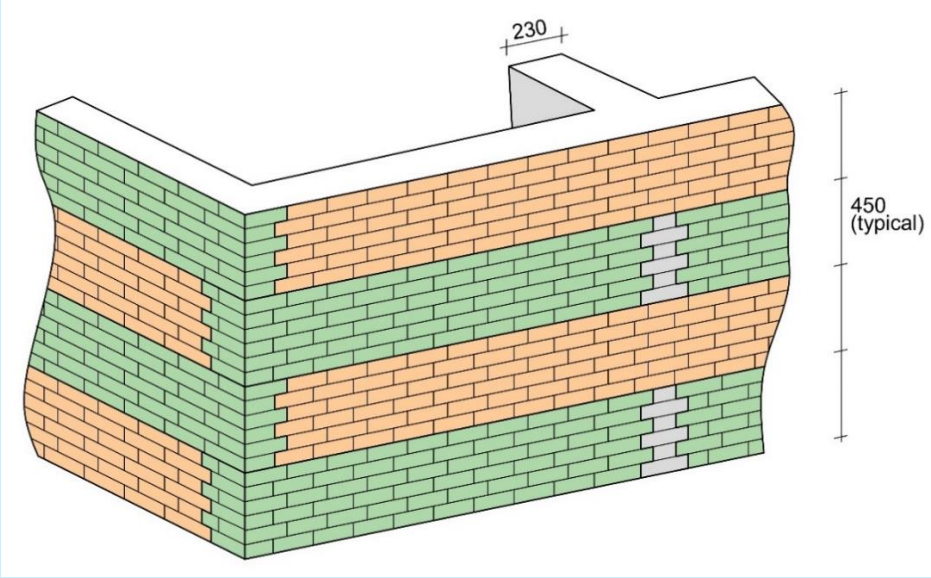
the wall in between them. Otherwise, the toothed joint should be made in both the walls alternatively in lifts of about 450 mm (Fig. 23).



All dimensions in millimetres.

FIG. 23 ALTERNATING TOOTHED JOINTS IN WALLS AT CORNER AND T-JUNCTION

8.5.2 *Openings in Bearing Walls*

8.5.2.1 Door and window openings reduce the lateral load resistance of walls, and hence, should preferably be small and more centrally located in the wall panels. The guidelines on the size and position of opening are given in Table 17 and Fig. 24.

8.5.2.2 Openings in any storey shall have their top at the same level to accommodate a continuous band over them, including the lintels throughout the building. In addition, openings in different storeys in a masonry wall shall be vertically aligned, and not staggered.

8.5.2.3 Where openings do not comply with the guidelines of Table 17, they shall be strengthened by providing reinforced concrete or reinforcing the brickwork (Fig. 25) with high strength deformed bars of 8 mm diameter but the quantity of steel shall be increased at the jambs to comply with **8.5.3.9**, if so required.

8.5.2.4 If a window or ventilator is to be projected out, the projection shall be in reinforced masonry or concrete and well anchored.

8.5.2.5 If an opening is tall (from bottom to almost top of a storey, thus dividing the wall into two portions), the adjoining portions shall be reinforced with horizontal reinforcement of 6 mm diameter bars at not more than 450 mm intervals, one on inner and one on outer face, properly tied to vertical steel at jambs, corners or junction of walls, where used.

8.5.2.6 The use of arches to span over the openings is a source of weakness and shall be avoided. When compelled to use, steel ties shall be provided.

8.5.3 *Earthquake Strengthening Arrangements*

8.5.3.1 All masonry buildings shall be strengthened by the methods specified for different types of buildings, as listed in Table 18, and detailed in subsequent clauses. Fig. 26 and Fig. 27 show, schematically, the overall strengthening arrangements to be adopted for types D and E buildings, which consist of horizontal bands of reinforcement at critical levels, vertical reinforcing bars at corners, junctions of walls and jambs of openings.

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In four-storey buildings of type B, the requirements of vertical steel may be checked through an earthquake analysis using a design horizontal acceleration coefficient equal to 4 times that given in **5** of Part C/Section 1 of this NBCS. If this analysis shows that vertical steel is not required, the designer may take the decision accordingly.

Table 17 Size and Position of Openings in Bearing Walls

(*Clauses* 8.5.2.1, 8.5.2.3 *and* Fig. 24)

Sl No.	Position of Opening	Details of Opening		
		B	C	D, E
(1)	(2)	(3)	(4)	(5)
i)	Minimum Distance b_5 (mm) from inside corner of outside wall	0	230	450
ii)	Total length of openings shall be such that the ratio $(b_1 + b_2 + b_3)/l_1$ or $(b_6 + b_7)/l_2$ does not exceed:			
	a) 1-storeyed building	0.60	0.55	0.50
	b) 2-storeyed building	0.50	0.46	0.42
	c) 3- or 4-storeyed building	0.42	0.37	0.33
iii)	Minimum width b_4 (mm) of pier between consecutive openings	340	450	560
iv)	Minimum vertical distance h_3 (mm) between two openings (one above the other)	600	600	600
v)	Maximum Width b_8 (mm) of opening of ventilator	900	900	900

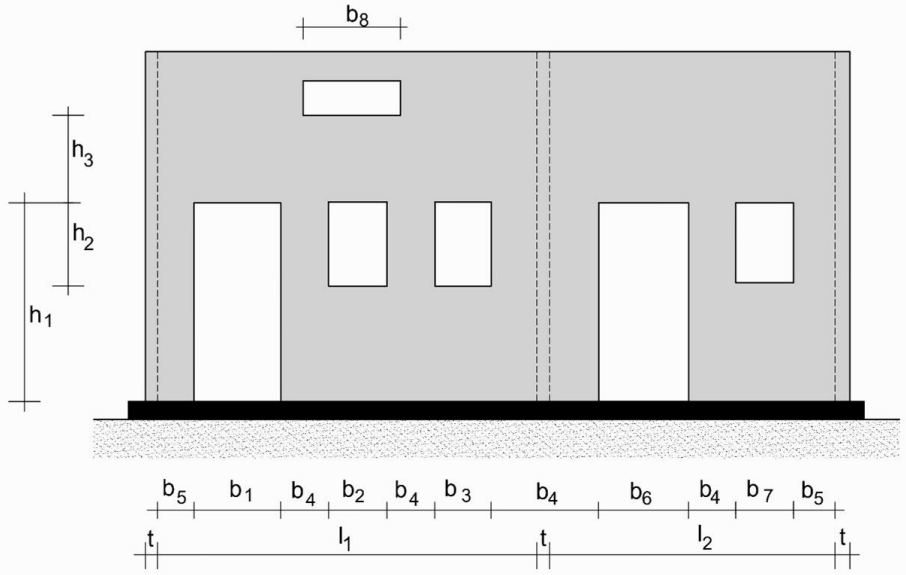

FIG. 24 DIMENSIONS OF OPENINGS AND PIERS FOR RECOMMENDATIONS IN TABLE 17

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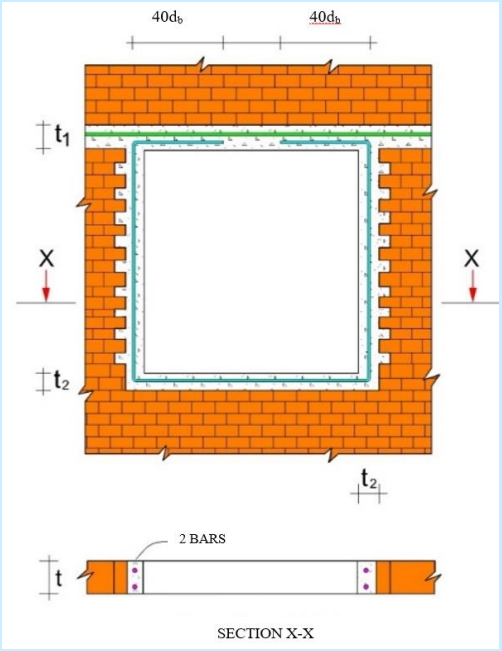

FIG. 25 STRENGTHENING MASONRY AROUND AN OPENING

Table 18 Measures Recommended in Masonry Buildings Made with Rectangular Masonry Units

(*Clauses* 8.3.7.1, 8.5.3.1 *and* 8.6.2.3)

Sl No.	Structure Type	Number of Storeys	Measures to be Provided in all Storeys
(1)	(2)	(3)	(4)
i)	B	1 to 3	a, b, c, f, g
		4	a, b, c, d, f, g
ii)	C	1 and 2	a, b, c, f, g
		3 and 4	a, b, c, d, e, f, g
iii)	D	1 and 2	a, b, c, d, e, f, g
		3 and 4	a, b, c, d, e, f, g, h
iv)	E	1 to 3	a, b, c, d, e, f, g, h

where

a = masonry mortar (as per **5.4.6**);

b = lintel band (as per **8.5.3.2**);

c = roof band and gable band where necessary (as per **8.5.3.3** and **8.5.3.4**, respectively);

d = vertical steel at corners and junctions of walls (as per **8.5.3.8**);

e = vertical steel at jambs of openings (as per **8.5.3.9**);

f = bracing in plan at tie level of roofs;

g = plinth band where necessary (as per **8.5.3.6**); and

h = dowel bars (as per **8.5.3.7**).

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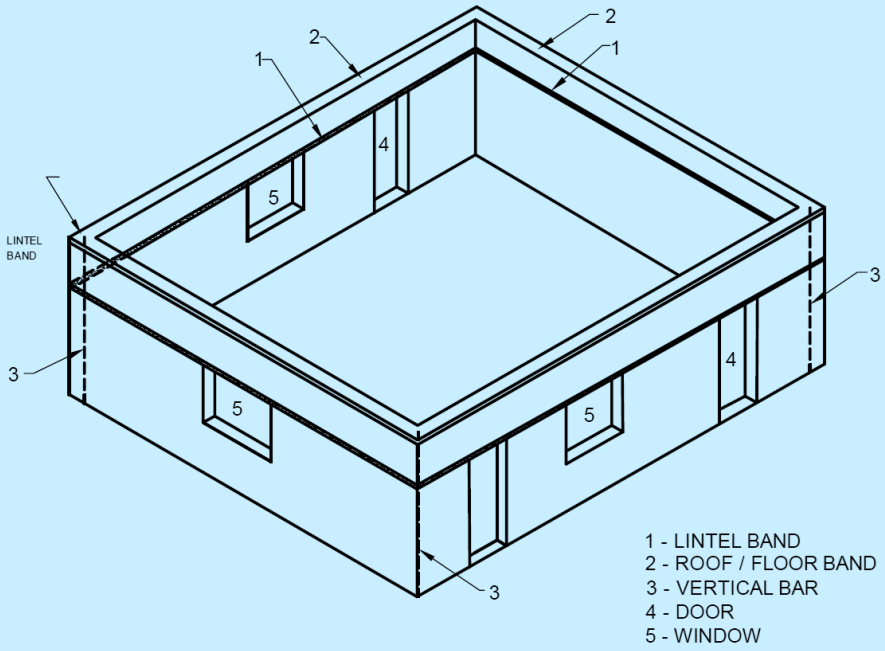
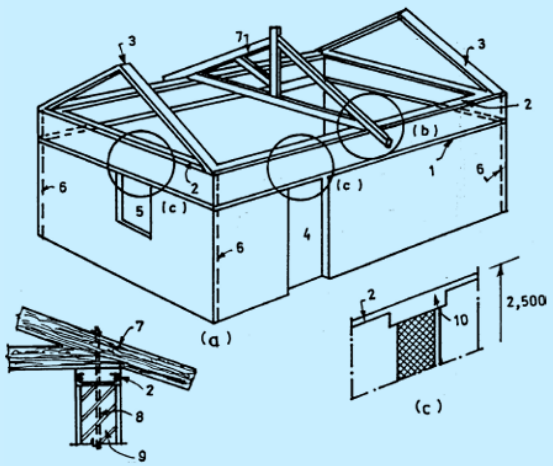


FIG. 26 OVERALL ARRANGEMENT OF REINFORCING MASONRY BUILDINGS



1 — Lintel band
2 — Eaves level (roof) band
3 — Gable band
4 — Door
5 — Window
6 — Vertical steel bar
7 — Rafter
8 — Holding down bolt
9 — brick/stone wall
10 — Door lintel integrated with roof band:
a) Perspective view
b) Details of truss connection with wall
c) Detail of integrating door lintel with roof band

FIG. 27 OVERALL ARRANGEMENT OF REINFORCING MASONRY BUILDING HAVING PITCHED ROOF

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8.5.3.2 Lintel band shall be provided at the lintel level on all load bearing internal, external longitudinal and cross walls, as per **8.5.3.5**. This band provided in panel or partition walls improves their stability during strong earthquake shaking.

8.5.3.3 Roof band shall be provided immediately below the roof or floors. The specifications of the band are given in **8.5.3.5**. Such a band need not be provided underneath reinforced concrete or brick-work slabs resting on bearing walls, provided that the slabs are continuous over the intermediate wall up to the crumple sections, if any, and cover the width of end walls, fully or at least three-fourths of the wall thickness.

8.5.3.4 Gable band is provided at the top of gable masonry below the purlins. The specifications of the band are given in **8.5.3.5**. This band shall be made continuous with the roof band at the eaves level.

8.5.3.5 *Section and reinforcement of band*

8.5.3.5.1 The band shall be as per **8.4.2**.

8.5.3.5.2 The bands shall be of the full width of the wall not less than 75 mm in depth and reinforced with longitudinal steel (Table 19). The longitudinal steel bars shall be held in position by steel links or stirrups 6 mm diameter spaced 150 mm apart. With respect to Table 19:

a) span of wall shall be the distance between centrelines of its cross walls or buttresses. For spans greater than 8 m intermediate pilasters or buttresses shall be provided or the adequacy of such long-span walls shall be ascertained through detailed analysis and full-scale test.

b) the numbers and diameter of bars given above pertain to high strength deformed bars.

c) the geometry of the bands shall be as per **8.4.2.2**.

d) concrete used in the bands shall be as per **8.3.4**.

e) the longitudinal steel bars shall be held in position by steel links or stirrups 6 mm diameter spaced at maximum 150 mm apart.

8.5.3.5.3 In reinforced brickwork, the thickness of joints containing steel bars shall be increased to have at least a minimum mortar cover of 10 mm around the bar. In bands of reinforced brickwork, the area of steel provided should be at least equal to that specified in Table 19.

8.5.3.5.4 For full integrity of walls at corners and junctions of walls and effective horizontal bending resistance of bands continuity of reinforcement is essential. The details as shown in Fig. 28 are recommended.

Table 19 Recommended Longitudinal Steel in Reinforced Concrete Bands

(*Clauses* 8.3.7.2, 8.5.3.5.2 *and* 8.5.3.5.3)

Sl No.	Span	Building Structure Type							
		B		C		D		E	
		Number of bars	Diameter	Number of bars	Diameter	Number of bars	Diameter	Number of bars	Diameter
	m		mm		mm		mm		mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
i)	5 or less	2	8	2	8	2	8	2	10
ii)	6	2	8	2	8	2	10	2	12
iii)	7	2	8	2	10	2	12	4	10
iv)	8	2	10	2	12	4	10	4	12

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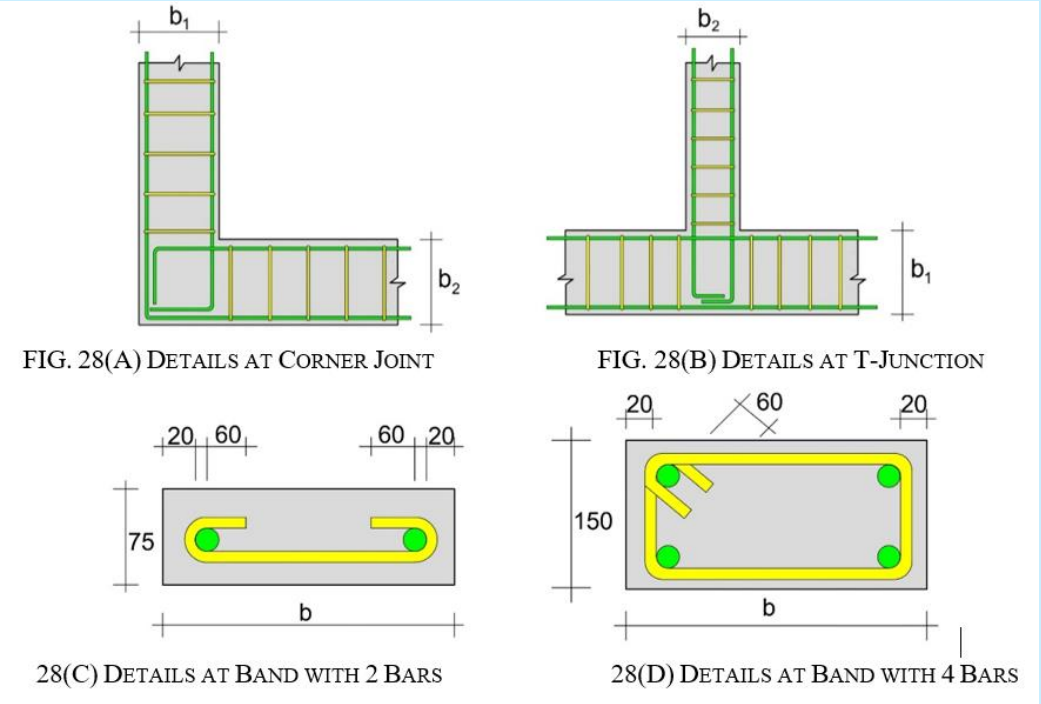

FIG. 28(A) DETAILS AT CORNER JOINT

FIG. 28(B) DETAILS AT T-JUNCTION

28(C) DETAILS AT BAND WITH 2 BARS

28(D) DETAILS AT BAND WITH 4 BARS

FIG. 28 REINFORCEMENT AND BENDING DETAIL IN RC BAND

8.5.3.6 Plinths band is provided at plinth level of walls on top of the foundation wall. This is to be provided where strip footings of masonry (other than reinforced concrete or reinforced masonry) are used and the soil is either soft or uneven in its properties. Where used, its section may be kept the same as in **8.5.3.5**. This band shall serve as damp proof course as well.

8.5.3.7 In Type D and E buildings, to further enhance the box action of walls, steel dowel bars may be used at corners and T-junctions of walls at the sill level of windows to length of 900 mm from the inside corner in each wall. Such dowel may be in the form of U stirrups 8 mm diameter. Where used, such bars must be laid in 1 : 3 cement sand mortar with a minimum clear cover of 10 mm on all sides to minimize corrosion.

8.5.3.8 *Vertical reinforcement*

Vertical steel at corners and junctions of walls, which are up to 340 mm (1½ brick) thick, shall be provided as specified in Table 20. For walls thicker than 340 mm, the area of the bars shall be proportionately increased. With respect to Table 20:

a) the diameters given above are for reinforcing steels of grade Fe415 or higher; and

b) typical details of providing vertical steel in brickwork masonry with rectangular solid units at corners and T-junctions are shown in Fig. 29. The vertical bars shall be covered with concrete of grade M20 or 1 : 3 mortar in suitably created pockets around the bars (Fig. 29). This shall ensure their safety from corrosion and good bond with masonry.

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Table 20 Diameter (mm) of Single Vertical Steel Reinforcement in Masonry Walls with Rectangular Masonry Units

(*Clause* 8.5.3.8, 8.5.3.9 *and* 8.6.2.3)

Sl No.	Number of Storeys	Storey	Buildings Structure Type			
			Type B	Type C	Type D	Type E
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	1	—	Nil	Nil	10	12
ii)	2	Top	Nil	Nil	10	12
		Bottom	Nil	Nil	12	16
iii)	3	Top	Nil	10	10	12
		Middle	Nil	10	12	16
		Bottom	Nil	12	12	16
iv)	4	Top	10	10	10	Not permitted
		Third	10	10	12	
		Second	10	12	16	
		Bottom	12	12	20	

8.5.3.8.1 The vertical reinforcement shall be properly embedded in the plinth masonry of foundations and roof slab or roof band to develop its tensile strength in bond. It shall pass through the lintel bands and floor slabs or floor level bands in all storeys.

8.5.3.8.2 Bars in different storeys shall be lapped as per the requirements of Part C/Section 5 of this NBCS.

8.5.3.9 Vertical reinforcement of jambs of window and door openings shall be provided as per Table 20. It may start from the foundation of floor and terminate in lintel band (Fig. 29).

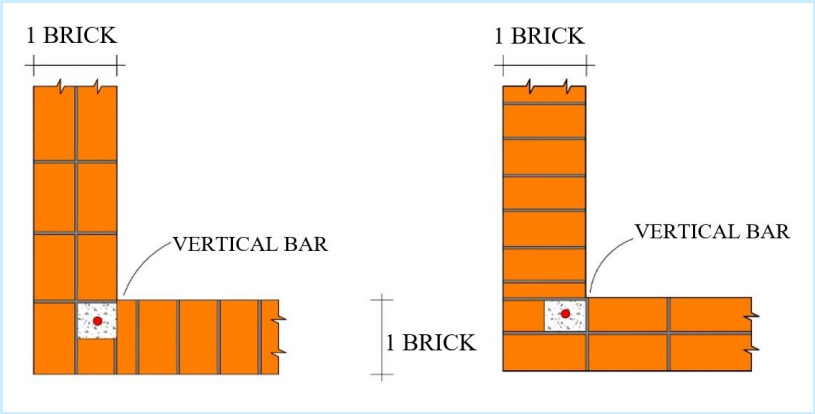

FIG. 29(A) SINGLE BRICK WALL CORNER JOINT DETAIL

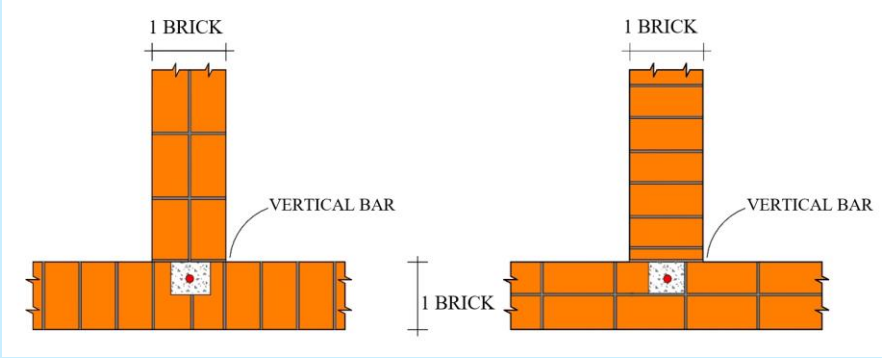

FIG. 29(B) SINGLE BRICK WALL T-JOINT DETAIL

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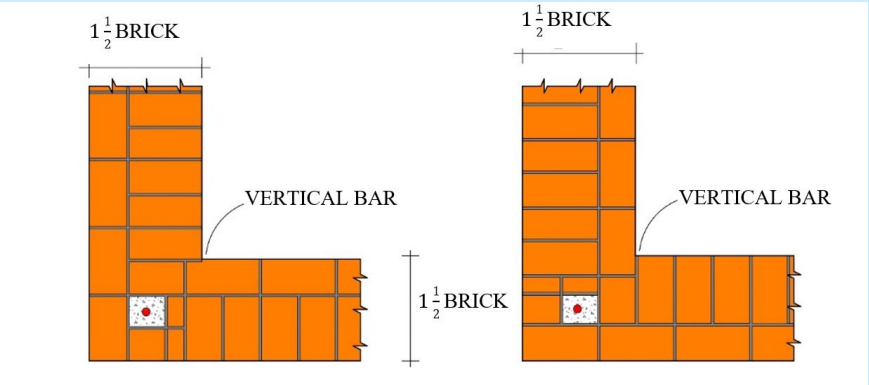

FIG. 29(C) 1½ BRICK WALL CORNER JOINT DETAIL

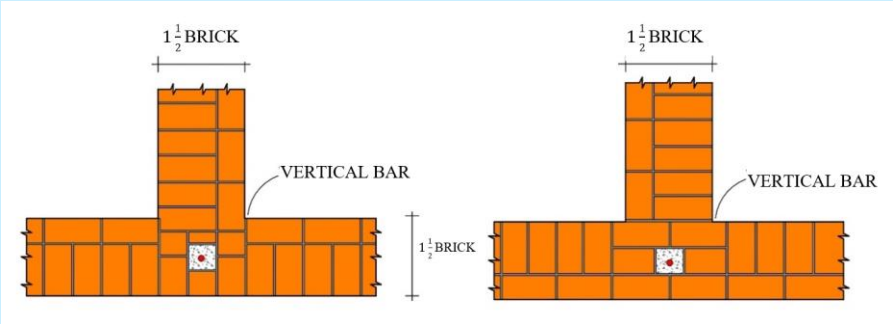

FIG. 29(D) 1½ BRICK WALL T-JOINT DETAIL

FIG. 29 TYPICAL DETAILS OF PROVIDING VERTICAL STEEL BARS IN BRICK MASONRY

8.6 Floors/Roofs with Small Precast Components

8.6.1 *Types of Precast Floors/Roofs*

Earthquake resistance measures for floors and roofs with small precast components, as covered in this Section, have been dealt with as typical examples.

8.6.1.1 *Precast reinforced concrete unit roof/floor*

The unit is a precast reinforced concrete component, channel (inverted trough) shaped in section (*see* Fig. 30). The nominal width of the unit varies from 300 mm to 600 mm, its height from 150 mm to 200 mm and a minimum flange thickness of 30 mm. Length of unit shall vary according to room dimensions, but the maximum length is restricted to 4.2 m from stiffness considerations. Horizontal corrugations are provided on the two longitudinal faces of the units so that the structural roof/floor acts monolithic after concrete grouted in the joints between the units attains strength (*see* Fig. 31).

8.6.1.2 *Precast reinforced concrete cored unit roof/floor*

The unit is a reinforced concrete component having a nominal width of 300 to 600 mm and thickness of 130 to 150 mm having two circular hollows 90 mm diameter, throughout the length of the unit (*see* Fig. 32). The minimum flange/web thickness of the unit shall be 20 mm. Length of unit varies according to room dimensions, but the maximum length shall be restricted to 4.2 m from stiffness considerations. Horizontal corrugations are provided on the two longitudinal faces of the units so that the structural roof/floor acts monolithic after concrete grouted in the joints between the units attains strength (*see* Fig. 33).

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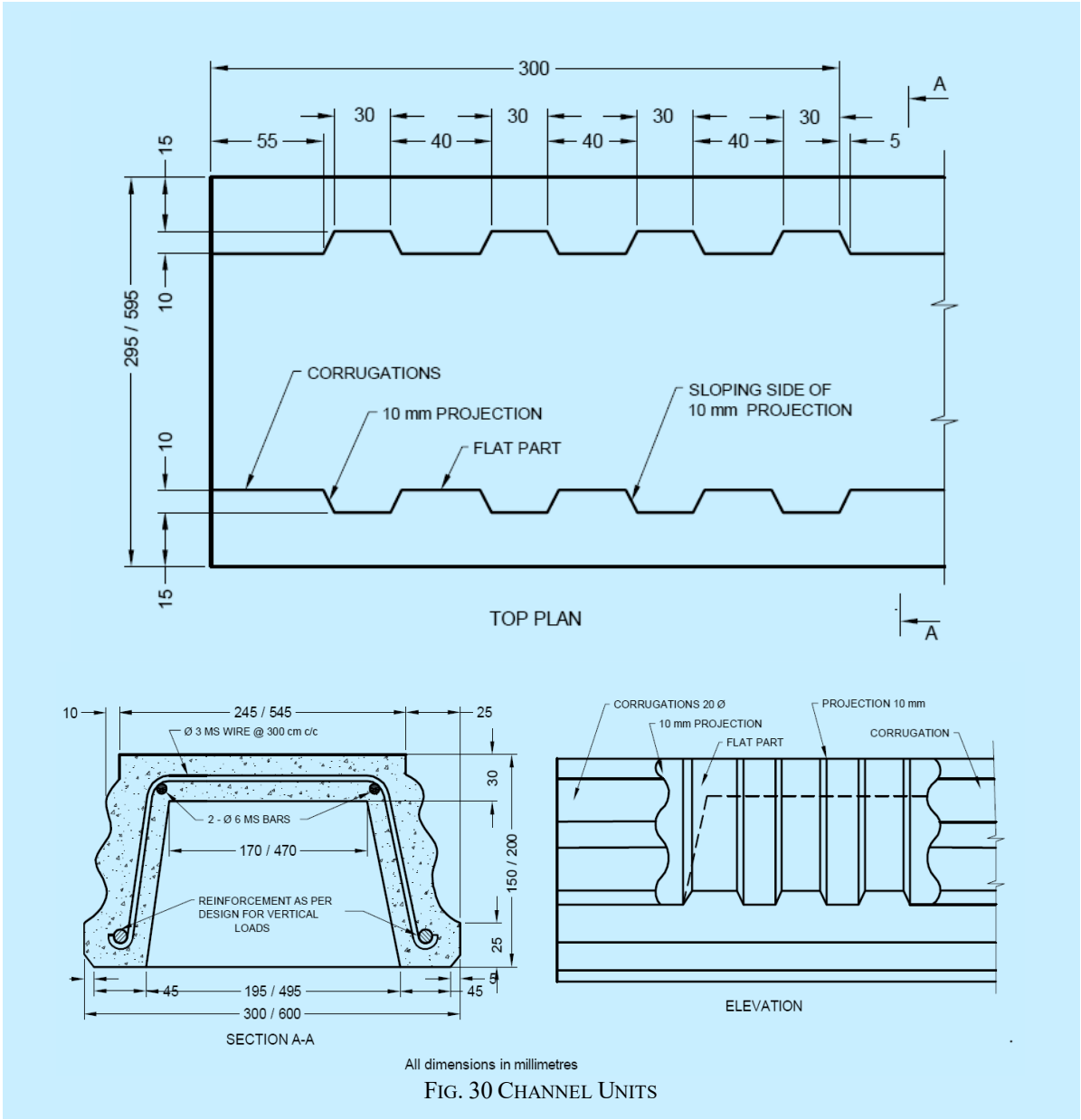

FIG. 30 CHANNEL UNITS

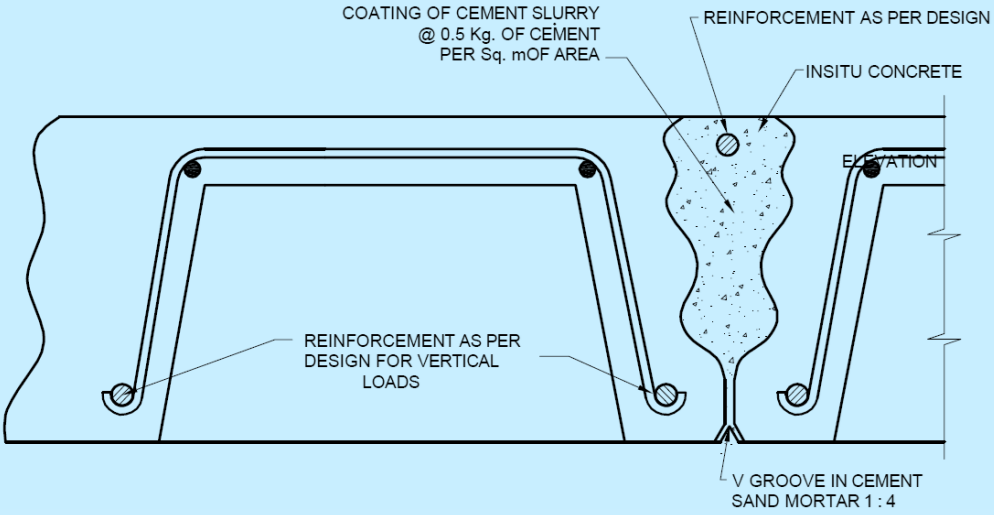

FIG. 31 CHANNEL UNITS

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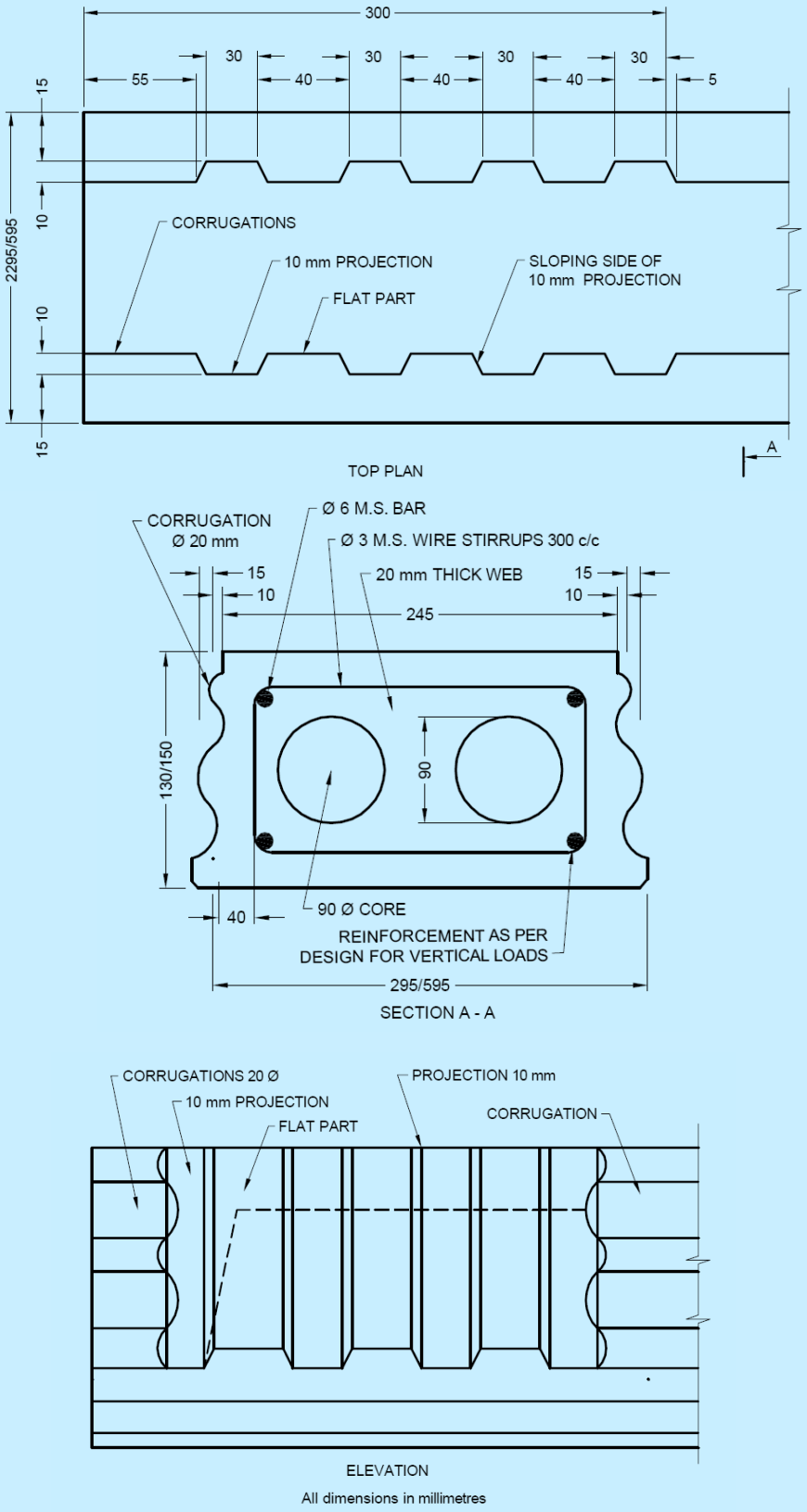

FIG. 32 CORE UNITS

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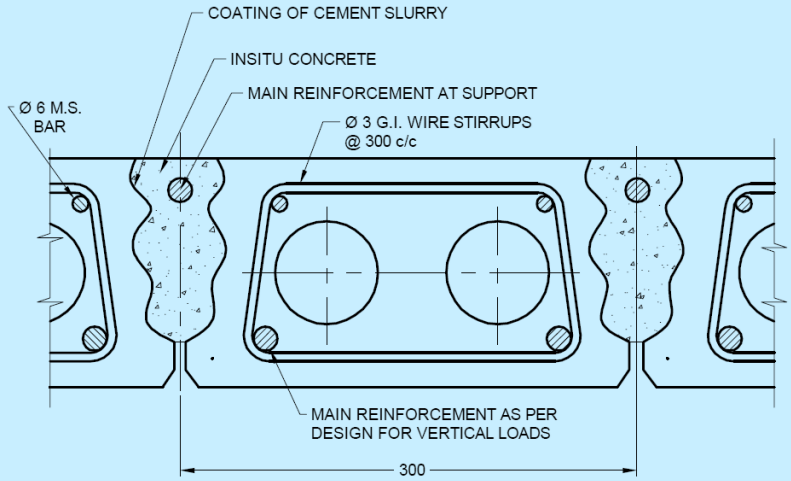


All dimensions in millimeteres.

FIG. 33 CORED, UNIT FLOOR

8.6.1.3 *Precast reinforced concrete plank and joist scheme for roof/floor*

The scheme consists of precast reinforced concrete planks supported on partially precast reinforced concrete joists. The reinforced concrete planks are 300 mm wide and the length varies according to the spacing of the joists, but it shall not exceed 1.5 m (*see* Fig. 34). To provide monolithic property to the roof/floor and to have T-beam effect with the joists, the planks shall be made partially 30 mm thick and the partially 60 mm thick and *in-situ* concrete shall be filled in the depressed portions to complete the roof/floor structurally (*see* Fig. 35).

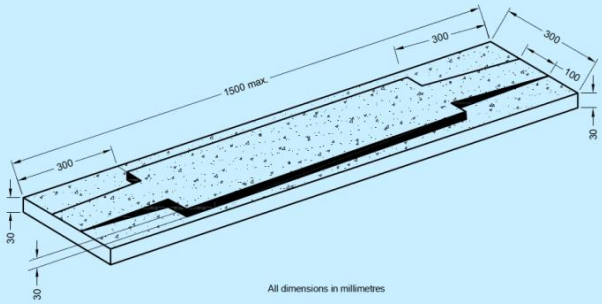
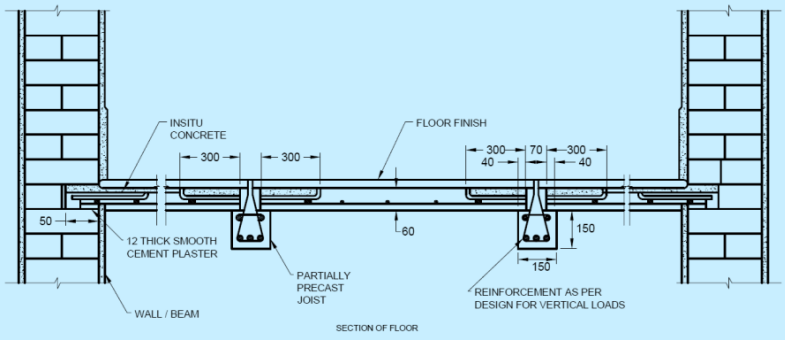


FIG. 34 PRECAST REINFORCED PLANK



All dimensions in millimeteres.

FIG. 35 PRECAST REINFORCED PLANK FLOOR

8.6.1.4 *Prefabricated brick panel system for roof/floor*

It consists of prefabricated reinforced brick panels (*see* Fig. 36) supported on Precast reinforced concrete joists with nominal reinforced 35 mm thick structural deck concrete over the brick panels and joists (*see* Fig. 37). The width of the brick panels shall be 530 mm for panels made of bricks of conventional size and 450 mm for panels made of bricks of modular size. The thickness of the panels shall be 75 mm or 90 mm, respectively depending upon whether conventional or modular bricks are used. The length of the panels shall vary depending upon the spacing of the joists, but the maximum length shall not exceed 1.2 m.

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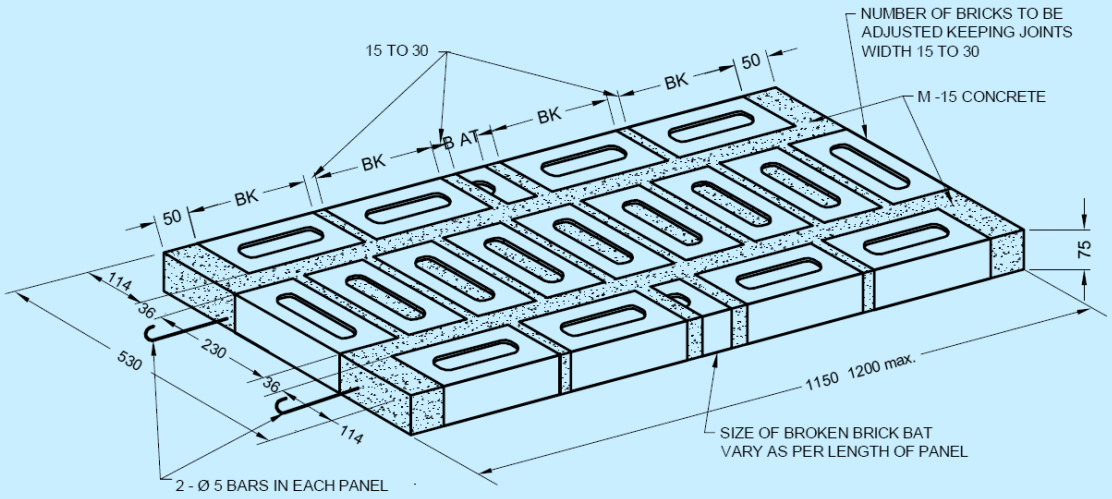

All dimensions in millimeteres.

FIG. 36 PREFAB BRICK PANEL

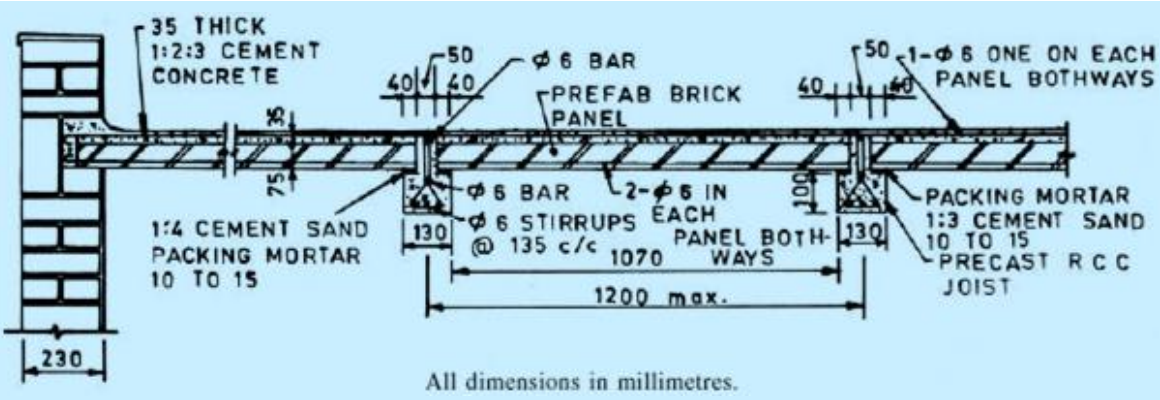

FIG. 37 BRICK PANEL FLOOR

8.6.1.5 *Precast reinforced concrete waffle unit roof/floor*

Waffle units are of the shape of inverted troughs, square or rectangular in plan, having lateral dimensions up to 1.2 m and depth depending upon the span of the roof/floor to be covered (*see* Fig. 38 and Fig. 39). The minimum thickness of flange/web shall be 35 mm. Horizontal projections may be provided on all the four external faces of the unit and the unit shall be so shaped that it shall act monolithic with *in-situ* concrete to ensure load transfer. Vertical castellation, called shear keys, shall be provided on all the four external faces of the precast units to enable them to transfer horizontal shear force from one unit to adjacent unit through *in-situ* concrete filled in the joints between the units. The waffle units shall be laid in a grid pattern with gaps between two adjacent units, and reinforcement, as per design, and structural concrete shall be provided in the gaps between the units in both the directions. The scheme is suitable for two way spanning roofs and floors of buildings having large spans.

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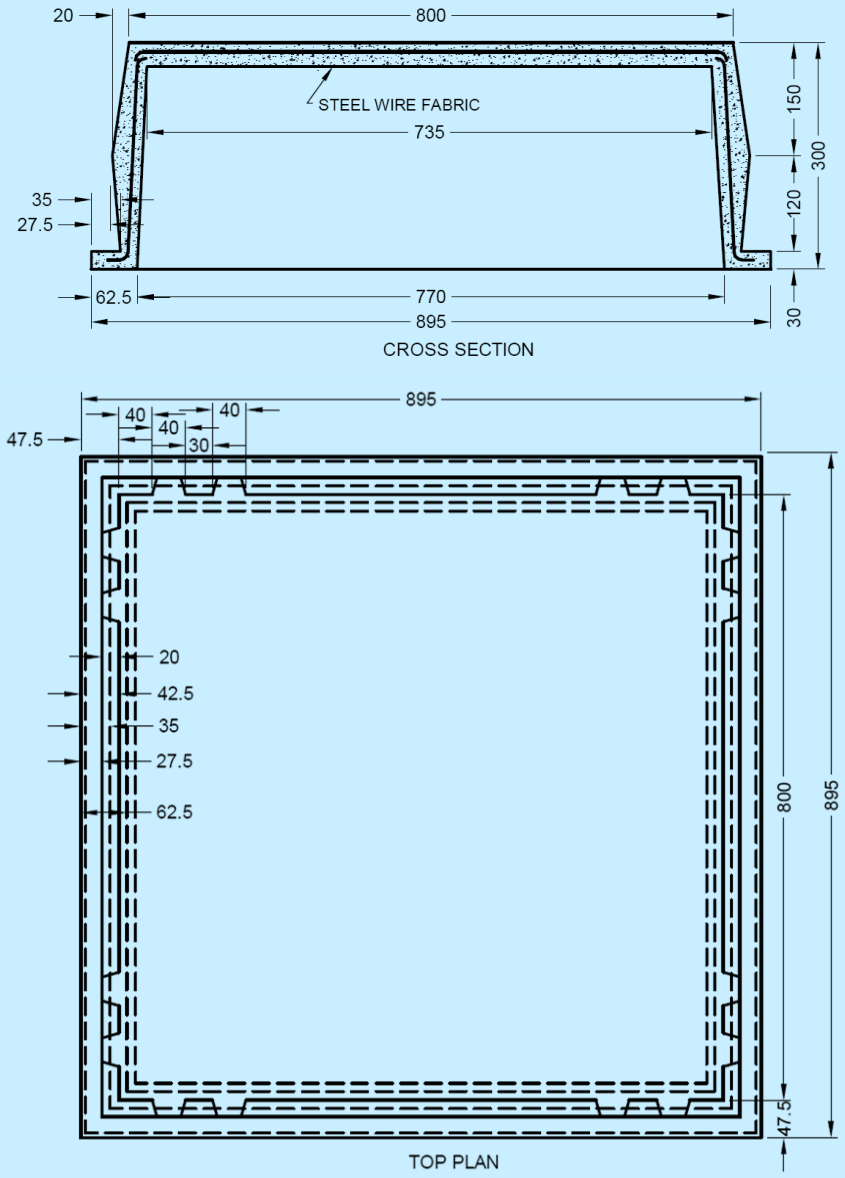
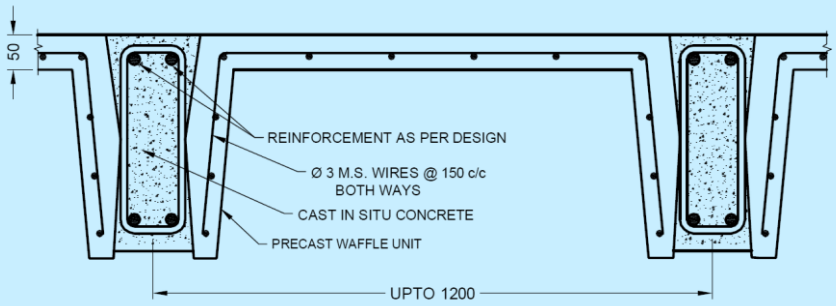


All dimensions in millimeteres.

FIG. 38 WAFFLE UNITS



All dimensions in millimeteres.

FIG. 39 WAFFLE UNIT FLOOR

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8.6.2 *Seismic Resistance Measures*

8.6.2.1 All floors and roofs to be constructed with small precast components shall be strengthened as specified for various categories of buildings in Table 21. The strengthening measures are detailed in **8.6.2.3** and **8.6.2.8**.

8.6.2.2 Vertical castellation, called shear keys, shall be provided on the longitudinal faces of the channel, cored and waffle units to enable them to transfer horizontal shear force from one unit to the adjacent unit through the *in-situ* concrete filled in the joints between the units. The minimum percentage of area of shear keys as calculated below, on each face of the unit, shall be 15.

Shear keys shall have a minimum width of 40 mm at its root with the body of the component and shall be to the full height of the component and preferably at uniform spacing. Percentage of area of shear keys shall be calculated as:

$$\frac{\text{No. of shear keys on one face of the component} \times 40}{\text{Length of the face of the component, in mm}} \times 100$$

Table 21 Strengthening Measures for Floors/Roofs with Small Precast Components

(*Clauses* 8.6.2.1, 8.6.2.4, 8.6.2.5, 8.6.2.6, 8.6.2.7 *and* 8.6.2.8)

Sl No.	Building Category	Number of Storeys	Strengthening to be Provided in Floor/Roof with			
			Channel/Cored Unit	R.C. Planks and Joists	Brick Panels and Joists	Waffle Units
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	B	1 to 34	aa, c	aa, c	aa, d	aa
ii)	C	1 and 2	a, ba, b, c	aa, c	aa, d	aa, e
		3 and 4				
iii)	D	1 to 4	a, b c	a, c	a, d	a, c, e
iv)	E	1 to 3	a, b, c	a, c	a, d	a, c, e

NOTES

1 Tie beam as per **8.6.2.3**;

2 Reinforcing bars of precast unit and tied to tie beam reinforcement as per **8.6.2.4**;

3 Reinforced deck concrete as per **8.6.2.5**;

4 Reinforced deck concrete as per **8.6.2.6**; and

5 Reinforced bars in joint between precast waffle units tied to tie beam reinforcement as per **8.6.2.7**.

8.6.2.3 Tie beam (*see* Table 20) is a beam provided all round the floor or roof to bind together all the precast components to make it a diaphragm. The beams shall be to the full width of the supporting wall or beam less the bearing of the precast components. The depth of the beam shall be equal to the depth of the precast components plus the thickness of structural deck concrete, where used over the components. The beam shall be made of cement concrete of grade not leaner than M15 and shall be reinforced as indicated in Table 18. If depth of tie is more than 75 mm, equivalent reinforcement shall be provided with one bar of minimum diameter 8 mm at each corner. Tie beams shall be provided on all longitudinal and cross walls. Typical details of the beams are shown in Fig. 40 to Fig. 44.

NOTE — Adequate edge support say 60 mm, shall be provided to precast element on the wall so as to avert its slippage during seismic ground motion.

8.6.2.4 Top reinforcement in the channel or cored units (*see* Table 21) shall be projected out at both the ends for anchorage length and tied to tie beam reinforcement.

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8.6.2.5 Structural deck concrete (*see* Table 21) of grade not leaner than M15 shall be provided over precast components or act monolithic. Wherever, deck concrete is to be provided, the top surface of the components shall be finished rough. Cement slurry with 0.5 kg of cement/m^2 of the surface area shall be applied over the components immediately before laying the deck concrete and the concrete shall be compacted using plate vibrators. The minimum thickness of deck concrete shall be 35 mm or 40 mm reinforced with 6 mm diameter bars and 150 mm apart both ways and anchored into the tie beam placed all round. The maximum size of coarse aggregate used in deck concrete shall not exceed 12 mm.

NOTE — Under conditions of economic constraints, the deck concrete itself could serve as floor finish. The concrete is laid in one operation (*see* Fig. 44) without joints.

8.6.2.6 The deck concrete normally used over the brick panel with joist floor shall be reinforced with 6 mm diameter bars spaced 150 mm apart both ways (*see* Table 21).

8.6.2.7 For floors/roofs with precast waffle units, two 16 mm diameter high strength deformed bars shall be provided as top reinforcement in the joints between waffle units, in addition to reinforcement required for taking bending moment for vertical loads. This reinforcement (*see* Table 21) shall be fixed to tie beam reinforcement.

8.6.2.8 In case of floors/roofs with precast components other than those indicated in Table 21, the buildings shall be analysed for maximum expected seismic forces and the floor/roof shall be designed to act as diaphragm and take care of the resulting forces.

8.7 Timber Construction

8.7.1 Timber has higher strength per unit weight and is, therefore, very suitable for earthquake resistant construction. Materials, design and construction in timber shall generally conform to good practice [C-4(9)].

8.7.2 Timber (with masonry in-fill) construction shall generally be restricted to two storeys with or without the attic floor.

8.7.3 In timber (with masonry in-fill) construction attention shall be paid to fire safety against electric short circuiting, kitchen fire, etc.

8.7.4 The superstructure of timber buildings shall be made rigid against deformations by adopting suitable construction details at the junctions of the framing members and in wall panels as given in **8.7.6** to **8.7.10** so that the construction as a whole behaves as one unit against earthquake forces.

8.7.5 *Foundations*

8.7.5.1 Timber construction shall preferably start above the plinth level, the portion below shall be in masonry or concrete.

8.7.5.2 The superstructure may be connected with the foundation in one of the two ways as given below:

a) The superstructure may simply rest on the plinth masonry, or in the case of small buildings of one storey having plan area less than about 50 m^2, it may rest on firm plane ground so that the building is free to slide laterally during ground motion; and

NOTES

1 Past experience has shown that superstructure of the buildings not fixed with the foundation escaped collapse even in a severe earthquake although they were shifted sideways.

2 Where fittings for water supply or water borne sanitation from the house are to be installed, proper attention should be given to permit movement so as to avoid fracture of damage to pipes.

b) The superstructure may be rigidly fixed into the plinth masonry or concrete foundation as given in Fig. 45 or in case of small building having plan area less than 50 m^2, it may be fixed to vertical poles embedded into the ground. In each case the building is likely to move along with its foundation. Therefore, the superstructure shall be designed to carry the resulting earthquake shears.

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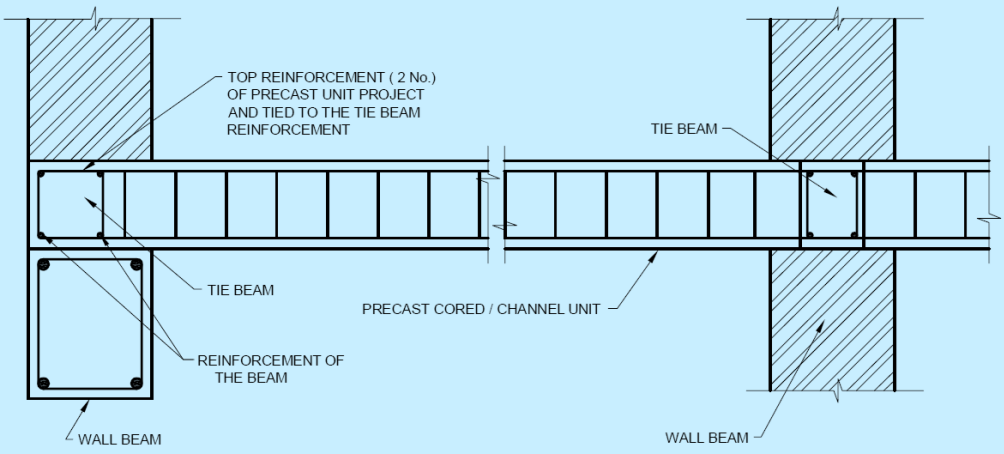

FIG. 40 CONNECTION OF PRECAST CORED/CHANNEL UNIT WITH TIE BEAM

8.7.6 *Types of Framing*

The types of construction usually adopted in timber building are as follows:

a) Stud wall construction; and

b) Brick nogged timber frame construction.

8.7.7 *Stud Wall Construction*

8.7.7.1 The stud wall construction consists of timber studs and corner posts framed into sills, top plates and wall plates. Horizontal struts and diagonal braces are used to stiffen the frame against lateral loads. The wall covering may consist of *EKRA*, timber or like. Typical details of stud walls are shown in Fig. 46. Minimum sizes and spacing of various members used are specified in **8.7.7.2** to **8.7.7.10**.

8.7.7.2 The timber studs for use in load bearing walls shall have a minimum finished size of 40 mm × 90 mm and their spacing shall not exceed those given in Table 22.

8.7.7.3 The timber studs in non-load bearing walls shall not be less than 40 mm × 70 mm in finished cross section. Their spacing shall not exceed 1 m.

8.7.7.4 There shall be at least one diagonal brace for every 1.6 m × 1 m area of load bearing walls. Their minimum finished sizes shall be in accordance with Table 23.

8.7.7.5 The horizontal struts shall be spaced not more than 1 m apart. They shall have a minimum size of 30 mm × 40 mm for all locations.

8.7.7.6 The finished sizes of the sill, the wall plate and top plate shall not be less than the size of the studs used in the wall.

8.7.7.7 The corner posts shall consist of three timbers, two being equal in size to the studs used in the walls meeting at the corner and the third timber being of a size to fit so as to make a rectangular section (*see* Fig. 46).

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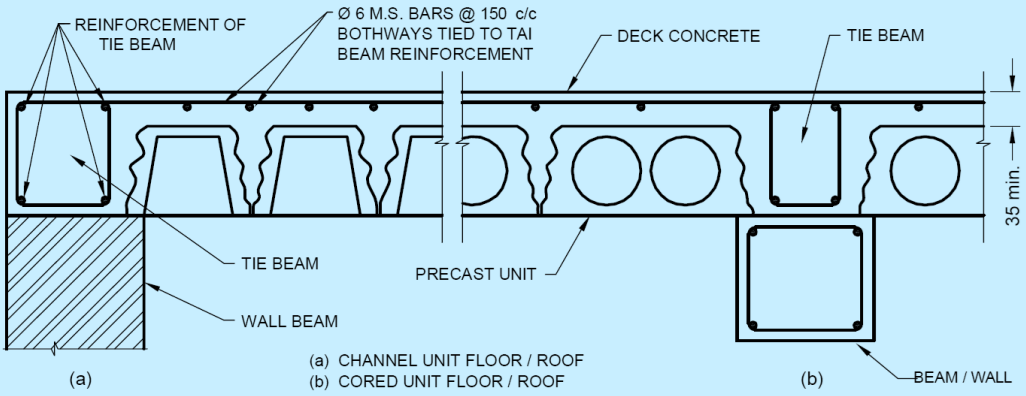
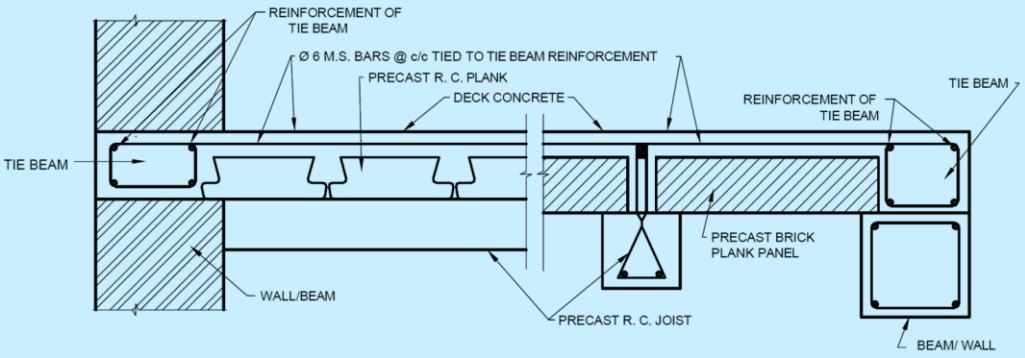


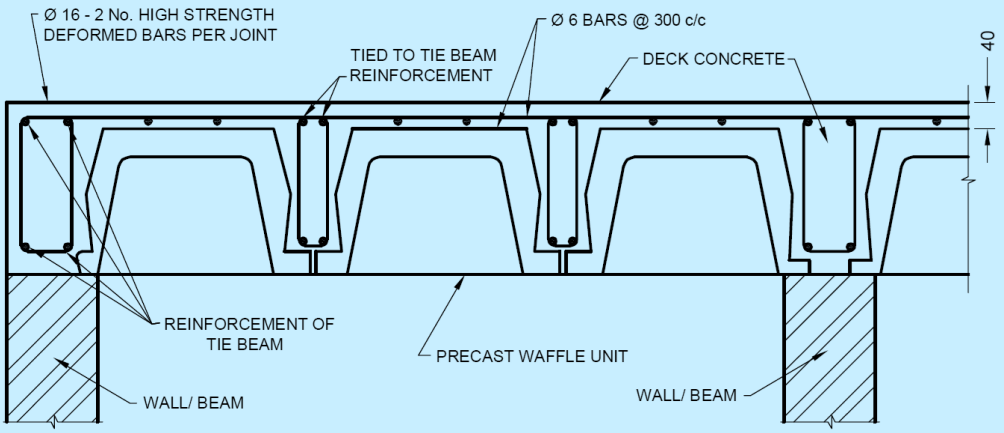
All dimensions in millimetres.

FIG. 41 CONNECTION OF CHANNEL/CORED UNIT FLOOR / ROOF "(WITH DECK CONCRETE) WITH THE BEAM



All dimensions in millimetres.

FIG. 42 CONNECTION OF PRECAST REINFORCED CONCRETE PLANK AND PRECAST BRICK PANEL FLOOR/ROOF (WITH DECK CONCRETE) WITH TIE BEAM



All dimensions in millimetres.

FIG. 43 CONNECTION OF PRECAST WAFFLE UNIT FLOOR/ROOF (WITH DECK CONCRETE) WITH TIE BEAM

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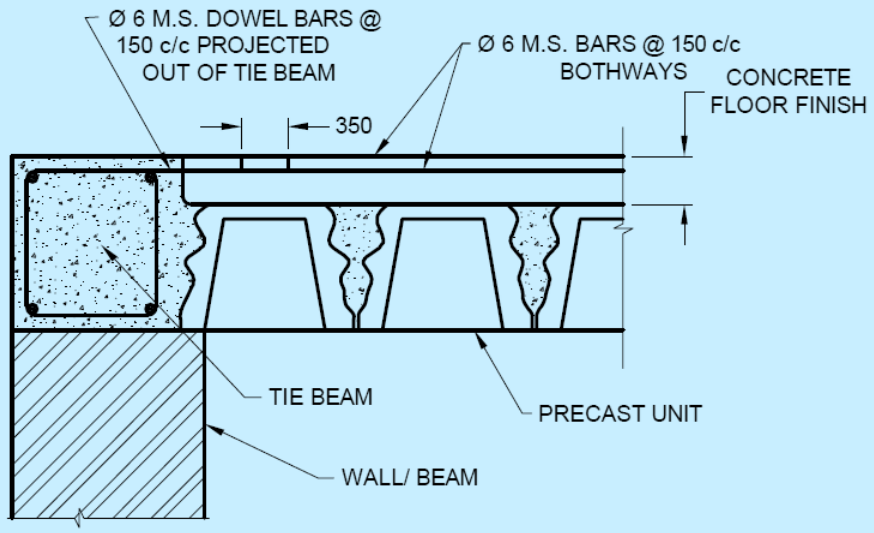
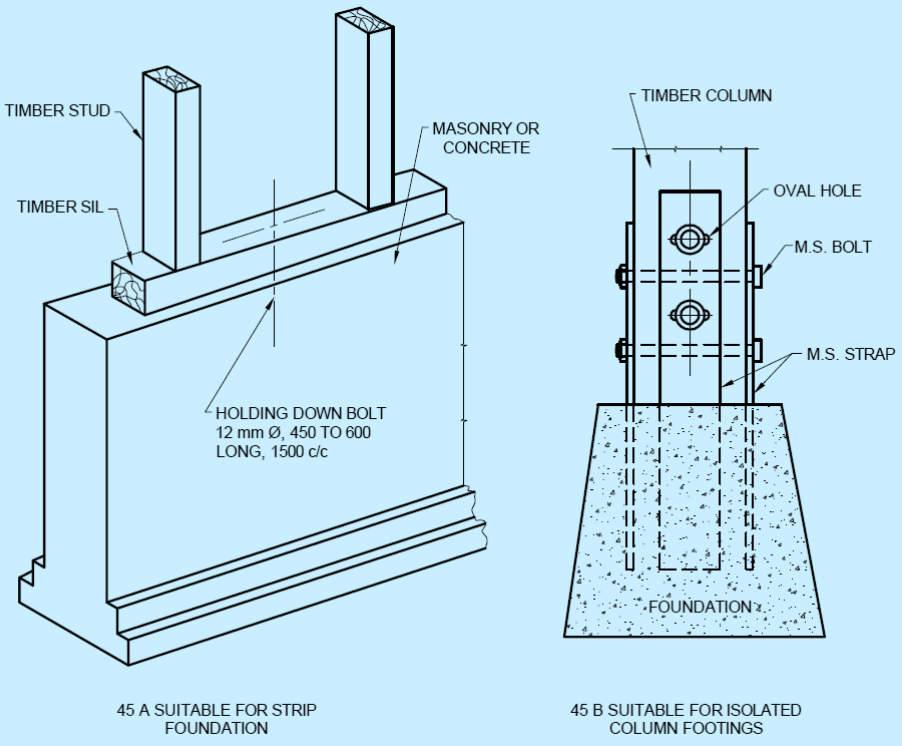


All dimensions in millimetres.

FIG. 44 PROVISION OF REINFORCEMENT IN CONCRETE FLOOR FINISH TIE BEAM



All dimensions in millimetres.

FIG. 45 DETAILS OF CONNECTION OF COLUMN WITH FOUNDATION

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Table 22 Maximum Spacing of 40 mm × 90 mm Finished Size Studs in Stud Wall Construction

(*Clauses* 8.7.7.2)

Sl No.	Group of Timber (Grade I[1])	Single Storeyed or First Floor of the Double Storeyed Buildings		Ground Floor of Double Storeyed Buildings	
		Exterior Wall mm	Interior Wall mm	Exterior Wall mm	Interior Wall mm
(1)	(2)	(3)	(4)	(5)	(6)
i)	Group A, B	1000	800	500	400
ii)	Group C	1000	1000	500	500

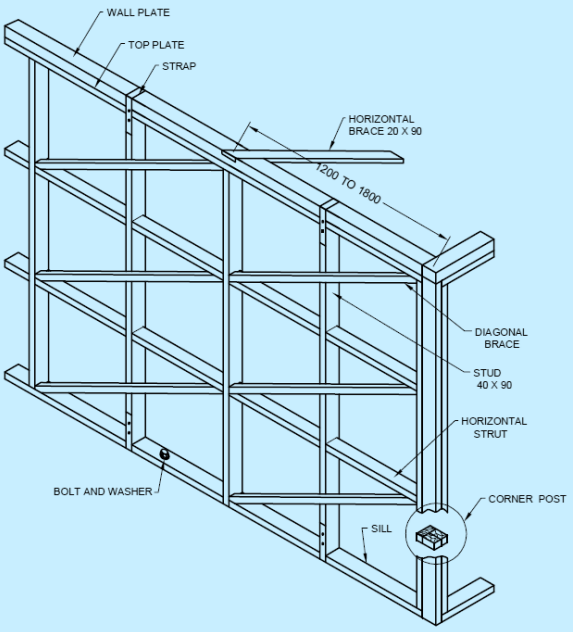
[1] Grade I timbers as defined in Table 5 of [C-4(9)].

Table 23 Minimum Finished Sizes of Diagonal Braces

(*Clauses* 8.7.7.4, 8.7.8.2 *and* 8.7.8.4)

Sl No.	Building Category	Group of Timber (Grade I[1])	Single Storeyed or First Floor of Double Storeyed Buildings		Ground Floor of Double Storeyed Building	
			Exterior Wall mm × mm	Interior Wall mm × mm	Exterior Wall mm × mm	Interior Wall mm × mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	B, C	All	20 × 40	20 × 40	20 × 40	20 × 40
ii)	D and E	Group A and Group B	20 × 40	20 × 40	20 × 40	30 × 40
		Group C	20 × 40	30 × 40	30 × 40	30 × 40

[1] Grade I timber as defined in Table 5 of [C-4(9)].



46 A TIMBER FRAMING IN STUD WALL CONSTRUCTION WITHOUT OPENING IN WALL

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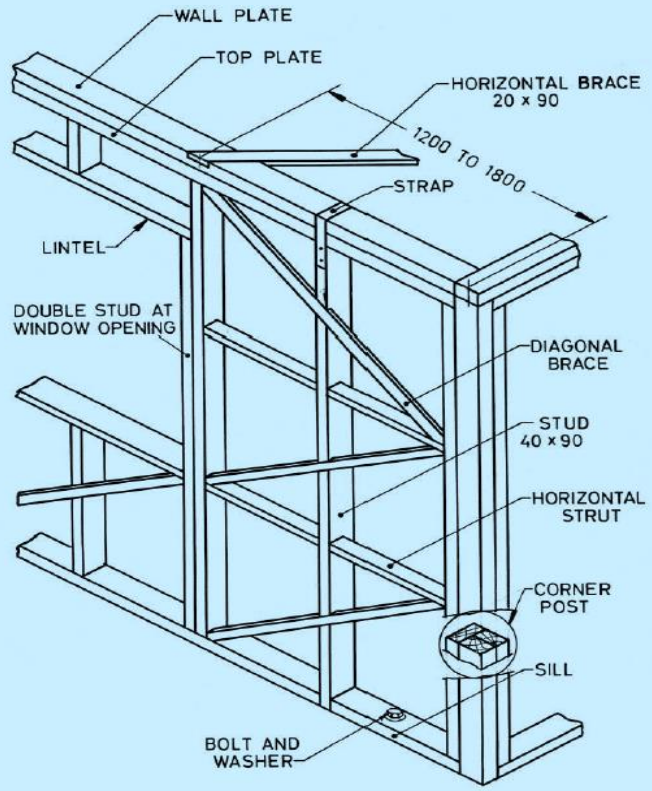

46 B TIMBER FRAMING IN STUD WALL CONSTRUCTION WITH OPENING IN WALL

All dimensions in millimetres.

FIG. 46 STUD WALL CONSTRUCTION

8.7.7.8 The diagonal braces shall be connected at their ends with the stud wall members by means of wire nails having 6 gauge (4.88 mm diameter) and 100 mm length. Their minimum number shall be 4 nails for 20 mm × 40 mm braces and 6 nails for 30 mm × 40 mm braces. The far end of nails may be clutched as far as possible.

8.7.7.9 Horizontal bracing shall be provided at corners of T-junctions of walls at sill, first floor and eaves levels. The bracing members shall have a minimum finished size of 20 mm × 90 mm and shall be connected by means of wire nails to the wall plates at a distance between 1.2 m and 1.8 m measured from the junction of the walls. There shall be a minimum number of six nails of 6 gauge (4.88 mm diameter) and 10 cm length with clutching as far ends.

8.7.7.10 Unsheathed studding shall not be used adjacent to the wall of another building. The studding shall be sheathed with close jointed 20 mm or thicker boards.

8.7.8 *Brick Nogged Timber Frame Construction*

8.7.8.1 The brick nogged timber frame consists of intermediate verticals, columns, sills, wall plates, horizontal nogging members and diagonal braces framed into each other and the space between framing members filled with tight-fitting brick masonry in stretcher bond. Typical details of brick nogged timber frame construction are shown in Fig. 47. Minimum sizes and spacing of various elements used are specified in **8.7.8.2** to **8.7.8.9**.

8.7.8.2 The vertical framing members in brick nogged load bearing walls shall have minimum finished sizes as specified in Table 23.

8.7.8.3 The minimum finished size of the vertical members in non-load bearing walls shall be 40 mm × 100 mm spaced not more than 1.5 m apart.

8.7.8.4 The sizes of diagonal bracing members shall be the same as in Table 23.

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8.7.8.5 The horizontal framing members in brick-nogged construction shall be spaced not more than 1 m apart. Their minimum finished sizes shall be in accordance with Table 24 and Table 25.

8.7.8.6 The finished sizes of the sill, wall plate and top plate shall be not less than the size of the vertical members used in the wall.

8.7.8.7 Corner posts shall consist of three vertical timbers as described in **8.7.7.7**.

8.7.8.8 The diagonal braces shall be connected of their ends with the other members of the wall by means of wire nails as specified in **8.7.7.8**.

8.7.8.9 Horizontal bracing members of corners of T-junctions of wall shall be as specified in **8.7.7.9**.

Table 24 Minimum Finished Sizes of Vertical in Brick Nogged Timber Frame Construction

(*Clause* 8.7.8.5)

Sl No.	Spacing	Group of Timber (Grade I[1)])	Single Storeyed or First Floor of Double Storeyed Buildings		Ground Floor of Double Storeyed Buildings	
			Exterior Wall	Interior Wall	Exterior Wall	Interior Wall
	m		mm × mm	mm × mm	mm × mm	mm × mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	1	Group A, B	50 × 100	50 × 100	50 × 100	50 × 100
		Group C	50 × 100	70 × 100	70 × 100	90 × 100
ii)	1.5	Group A, B	50 × 100	70 × 100	70 × 100	80 × 100
		Group C	70 × 100	80 × 100	80 × 100	100 × 100

[1)] Grade I timber as defined in Table 5 of [C-4(9)].

Table 25 Minimum Finished Size of Horizontal Nogging Members

(*Clause* 8.7.8.5)

Sl No.	Spacing of Verticals	Size
(1)	(2)	(3)
i)	1.5	70 × 100
ii)	1	50 × 100

8.7.9 *Notching and Cutting*

8.7.9.1 Timber framing frequently requires notching and cutting of the vertical members. The notching or cutting should in general be limited to 20 mm in depth unless steel strips are provided to strengthen the notched face of the members. Such steel strips, where necessary shall be at least 1.5 mm thick and 35 mm wide extending at least 150 mm beyond each side of the notch or cut and attached to the vertical member by means of bolts or screws at each end.

8.7.9.2 The top plate, the wall plate or the sill of a wall may be notched or cut, if reinforcing strip of iron is provided as specified in **8.7.9.1**. In case the member is notched or cut not to exceed 40 mm in depth, such reinforcing strip may be placed along the notched edge only. Where the notch or cut is more than 40 mm in depth or the member is completely cut through, such reinforcing strips shall be placed on both edges of the member. The details of notching and cutting are shown in Fig. 48.

8.7.9.3 Joints in timber shall preferably be bound by metallic fasteners.

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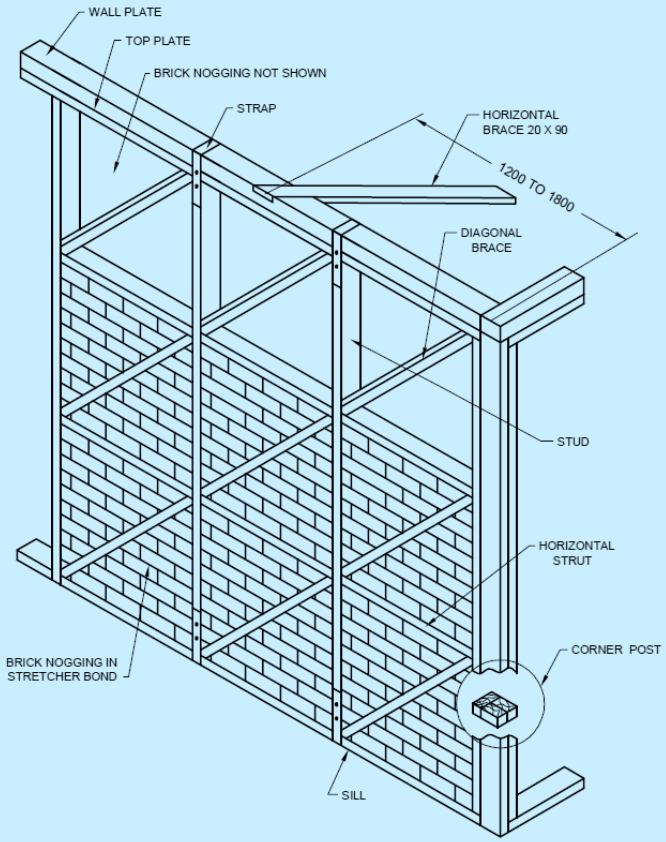
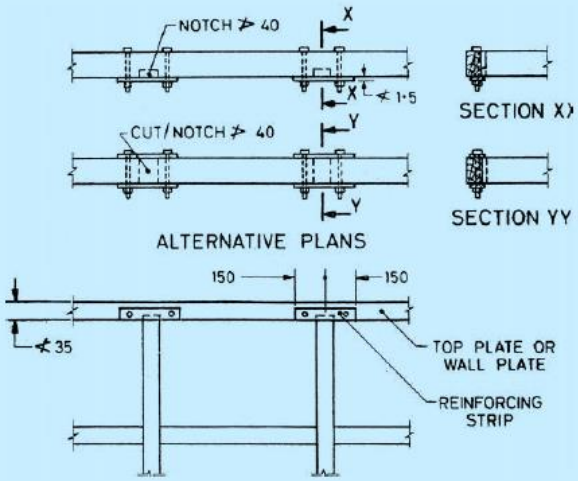


FIG. 47 BRICK NOGGED TIMER FRAME CONSTRUCTION



All dimensions in millimetres.

FIG. 48 NOTCHING AND CUTTING

8.7.10 *Bridging and Blocking*

8.7.10.1 All wooden joists shall have at least one row of cross bridging for every 3.5 m length of span. The cross-section of the bridging member shall be a minimum of 40 mm × 70 mm and the member shall be screwed or nailed to the joists.

8.7.10.2 All spaces between joists shall be blocked at all bearing with solid blocks not less than 40 mm thick and the full depth of the joists. The block shall be screwed or nailed to the joists as well as to the bearings.

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9 CONFINED MASONRY WALLS (CMW)

9.1 Design Considerations

9.1.1 *General*

The provisions given in 9.1.2 to 9.1.6 apply in addition to those specified in 4.

9.1.2 *Structural Integrity*

Intersecting walls shall be joined together to resist the effect of gravity and lateral loads. The walls shall be adequately connected to elements which provide lateral support, such as floors and roofs.

9.1.3 *Building Configuration*

A regular building configuration is one of the key requirements for satisfactory earthquake performance. The following recommendations related to building plan shape shall be followed:

a) The building's length-to-width ratio in plan shall not exceed 4 and storey height shall be less than 4 m;

b) The walls shall be built in a symmetrical manner with regard to the horizontal axes through the centre of the building plan. Some walls shall be placed on the perimeter also to increase resistance of the building to twisting (torsion) during an earthquake;

c) There shall be at least two lines of walls in each orthogonal direction of the building plan, and the walls along each line shall extend over at least 50 percent of the building dimension in the direction considered at each storey level (Fig. 49);

d) The walls shall be continuous up to the full building height. There shall be no vertical offsets in the walls along the building height;

e) The openings (doors and windows) shall be placed in the same position on each floor;

f) The total cross-sectional area of all walls in two adjacent floors shall not be different by more than 30 percent; and

g) The structural system in a building shall be the same in all storeys and in both principal plan directions.

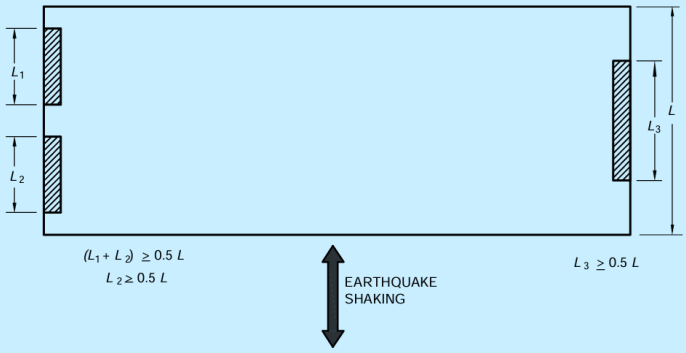

FIG. 49 AT LEAST TWO PARALLEL LINES OF WALLS ARE REQUIRED IN EACH PLAN DIRECTION

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9.1.4 *Minimum Design Dimensions and Placements of Confining Elements*

Requirements regarding spacing of tie-columns and tie-beams are presented in Fig. 50. Minimum dimension of tie-column and tie-beam shall be 150 mm along the wall direction and as much as the thickness of the wall in the direction perpendicular to it.

9.1.4.1 *Placement of reinforced concrete tie-columns and tie-beams*

Tie-columns should be provided at:

a) intersections of walls;

b) intermediate locations in longer walls, where spacing shall not exceed lesser than 1.5 *h* (where *h* is the clear wall height between floors) or 4 m; and

c) free ends of wall panels that provide lateral load resistance to the building (Fig. 51).

Also, reinforced concrete tie-beams must be provided at the top of each wall and spacing between tie-beams should be preferably less than 4 m.

9.1.4.2 *Horizontal lintel bands*

When the unsupported height of the wall *h* is greater than 2.5 m, a continuous horizontal lintel band shall be provided over the openings. A continuous horizontal band is not essential for $h < 2.5$ m, but a lintel beam shall be provided above the openings.

9.1.5 *Minimum Dimensions of Masonry Walls*

The following shall be applicable:

a) the wall thickness (t) shall not be less than 110 mm; and

b) the maximum wall height-to-thickness (h/t) ratio shall not exceed 25, where *h* is the height between floors.

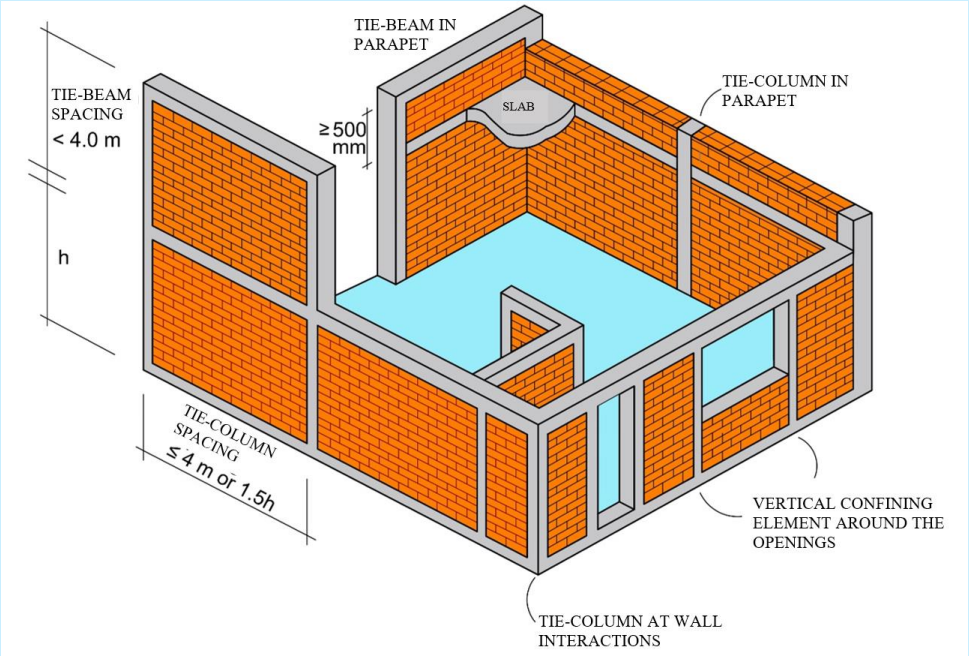

FIG. 50 REQUIREMENTS REGARDING CONFINING ELEMENTS

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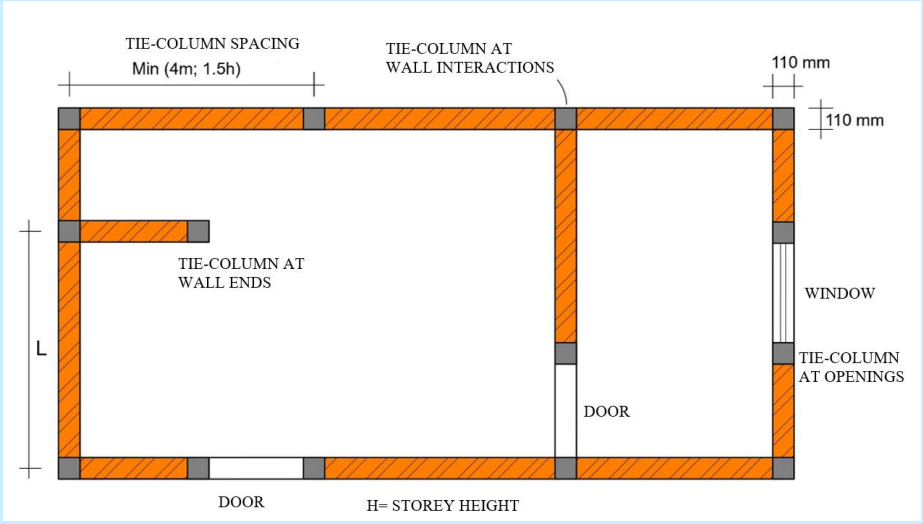

FIG. 51 TYPICAL PLAN ILLUSTRATING THE PLACEMENT OF RC TIE COLUMNS

9.1.5.1 *Aspect Ratio of Confined Masonry Walls*

The height-to-width ratio of a wall that is a part of the lateral force-resisting system shall be less than or equal to 2.

9.1.5.2 *Parapets*

RC tie-columns and tie-beams shall extend to the top of the parapet Fig. 51. When a parapet is not confined by tie-beams, its height shall not be more than 3 times its thickness. For parapet walls with top tie-beams, the height shall not be more than 1.2 m.

9.1.5.3 *Gable Walls*

The top of gable shall be confined with reinforced concrete tie-beams, and tie-columns located at the middle of the gable wall shall be extended from the lower floor to the top of gable wall (whenever applicable), as shown in Fig. 52(A) Alternatively, a gable portion of the wall can be made of timber or other light-weight material that is continuous Fig. 52(B).

9.1.6 *Walls with Openings*

9.1.6.1 *Size of opening*

The presence of large openings may have a negative effect on earthquake performance of confined masonry buildings, especially if openings are not confined. A large opening has a total area greater than 10 percent of the wall panel area, while a small opening has a total area less than or equal to 10 percent of the wall panel area.

9.1.6.2 *Walls with large openings*

The following three approaches shall be followed in walls with large openings:

a) when reinforced concrete tie-columns are not provided at the ends of an opening, the panel shall not be considered as confined and its contribution to earthquake resistance of the building shall be disregarded but shall be strengthened [*see* Fig. 53(A)];

b) when reinforced concrete tie-columns are provided at the opening, and the confined masonry panels are considered to contribute to earthquake resistance of the building, the aspect ratio H/L of these panels shall be less than 2.0 [Fig. 53(B)]. Better performance can be achieved by providing both sill and lintel bands below and above the openings, respectively; and

c) if total area of openings is greater than 25 percent, both openings and masonry piers shall be confined with horizontal and vertical confining elements [Fig. 53(C)].

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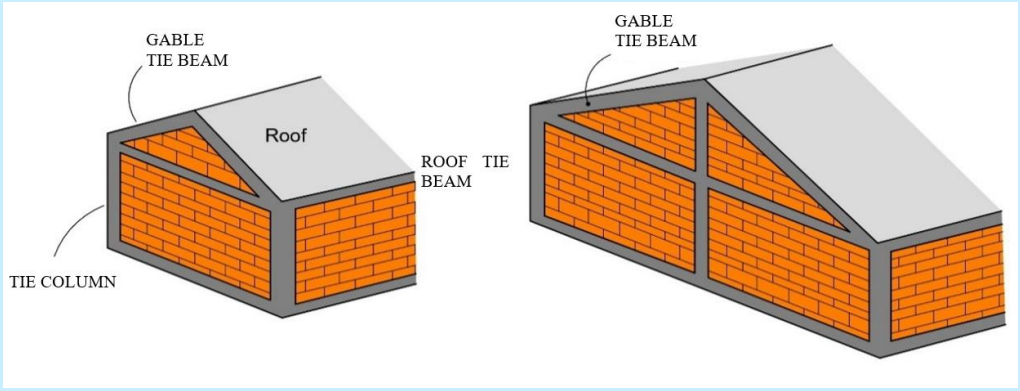

FIG. 52(A) CONFINING ELEMENTS

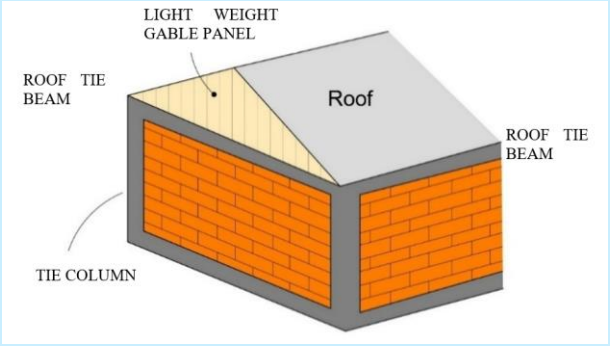

FIG. 52(B) LIGHT-WEIGHT GABLE PANELS

FIG. 52 TYPES OF GABLE WALLS

9.1.6.3 *Walls with small and single openings*

The following three approaches shall be followed in walls with small openings:

a) when the opening is located outside the diagonals Fig. 54(a), it shall be ignored, and the entire wall cross-sectional area considered for earthquake resistance;

b) when an opening is located at the intersection of the panel diagonals [Fig. 54(b)], the panel cross-sectional area (A_T) considered for earthquake resistance should exclude the opening length; and

c) when an opening is located close to one end of the panel, the panel cross-sectional area (A_T) considered in earthquake resistance shall use the larger pier length [Fig. 54(c)].

9.2 Structural Design

9.2.1 *Design Criteria*

Design of structural elements shall be performed as per the working stress method.

9.2.2 *Structural Design as per Working Stress Method*

9.2.2.1 *Load combinations*

The working stress method shall be employed for the structural design of SEs of confined masonry buildings for the load combinations specified in **5** of Part C/Section 1 of this NBCS.

9.2.2.2 *Transformed section properties*

Confined masonry wall panel shall be treated as a composite structural element which consists of a masonry wall and reinforced concrete tie-columns, and designed as a transformed section. Modular ratio (m) shall be taken as the ratio of elastic modulus for concrete and masonry. The modulus of elasticity of concrete can be taken as per

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Part C/Section 5 of this NBCS and modulus of elasticity of masonry as per **5.4.6**. Transformed section properties of a confined masonry panel used in design are illustrated in Fig. 55.

9.2.2.3 *Permissible compressive force*

Compressive force in confined masonry due to axial load shall not exceed that given by:

$$P_o = (0.25\, f_m\, A_m + f_{cc}\, A_c + 0.65\, A_s f_s)\, k_s,$$

where

A_c = cross-sectional area of concrete excluding reinforcing steel;

A_m = net area of masonry;

A_s = area of steel;

f_{cc} = permissible stress in concrete in direct compression as per Part C/Section 5 of this NBCS;

f_m = compressive strength of masonry as per **5.4.5**;

f_s = permissible steel tensile stress as per Part C/Section 5 of this NBCS; and

k_s = stress reduction factor as per Table 10.

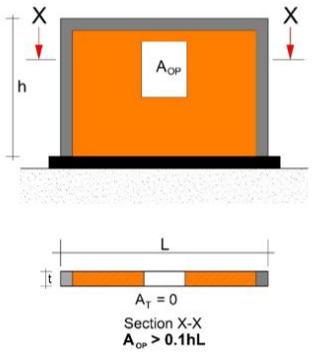

FIG. 53(A) UNCONFINED PANEL

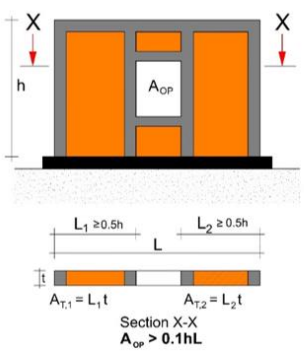

FIG. 53(B) PANEL WITH TIE COLUMNS

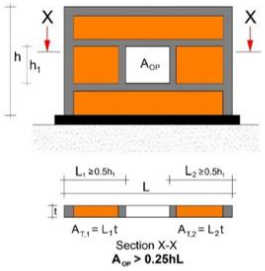

FIG. 53(C) PANEL WITH HORIZONTAL BANDS

FIG. 53 MASONRY WALLS WITH LARGE OPENING

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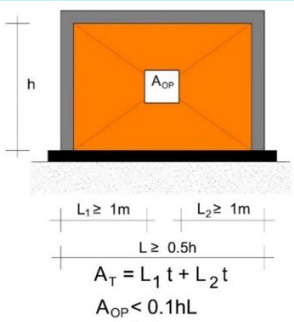


FIG. 54(A) OPENING ALONG DIAGONAL

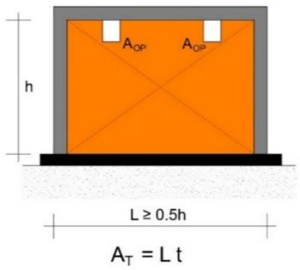


FIG. 54(B) OPENING OFF THE DIAGONAL

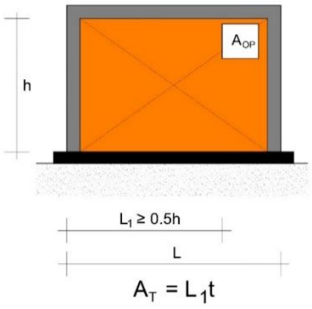
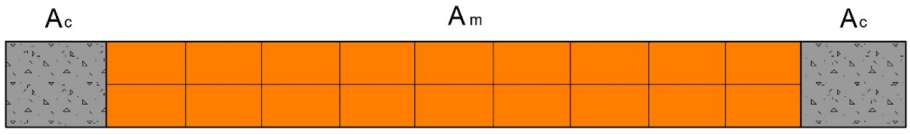
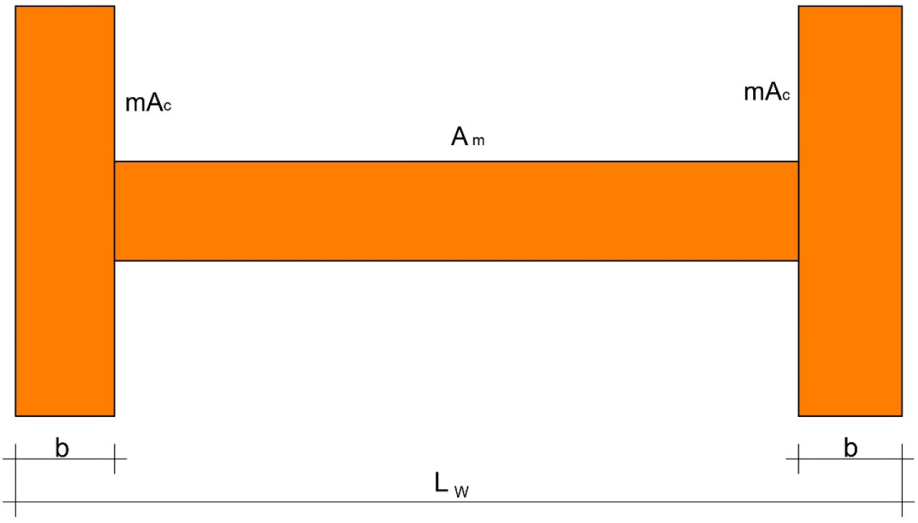


FIG. 54(C) OPENING ALONG THE DIAGONAL

FIG. 54 CONFINED MASONRY WALL PANEL WITH SMALL OPENING



55A) MASONRY WALL



55B) WALL TO BE CONSIDERED IN DESIGN

FIG. 55 TRANSFORMED SECTION PROPERTIES OF A CONFINED MASONRY PANEL

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Table 26 Maximum Allowable Shear Stress τ_{cm} of Confined Masonry

(*Clause* 9.2.2.6)

Sl No.	$\left(\frac{M}{Vd}\right)$	f_t (MPa)	τ_{cm} (MPa)
(1)	(2)	(3)	(4)
i)	< 1	$\frac{1}{36}\left(4-\frac{M}{Vl_w}\right)$	$0.4\text{-}0.2\left(\frac{M}{Vl_w}\right)$
ii)	> 1	$0.083\sqrt{f_m}$	0.4

9.2.2.4 *Combined permissible axial and flexural compressive stress*

For walls subjected to combined axial load and flexure, the compressive stress in masonry due to combined action of axial load and bending shall not exceed 1.25 f_a and compressive stress in masonry due to axial load only shall not exceed f_a.

9.2.2.5 *Permissible tensile stress*

Provisions of **5.4.2** shall apply.

9.2.2.6 *Permissible shear stress*

The permissible shear stress τ_{cm} for confined masonry walls shall be as per Table 26. If there is tension in any part of a section of masonry, the area under tension shall be ignored while estimating the shear stress demand on the cross-section.

When designing buildings as per the working stress method, the load combinations given in Table 27 shall be used. These combinations are provided considering short-term effects. When assessing the long-term effects due to creep, the dead load, and that part of imposed load likely to be permanent alone shall be used.

Table 27 Load Combinations to be Considered in Design of Buildings as per Working Stress Method

(*Clause* 9.2.2.6)

Sl No.	Combination Case	Serviceability Load Combination
(1)	(2)	(3)
i)	1	DL + IL
ii)	2	DL + EL
iii)	3	DL + 0.8 IL + 0.8 EL

9.2.2.7 *Design assumptions*

The following assumptions shall be made in design of confined masonry wall panels subjected to axial and/or flexural loads:

a) masonry behaves like a homogeneous material;

b) strain distribution across the cross-section of the wall varies linearly;

c) tensile stresses are resisted only by reinforcing steel (masonry and concrete tensile resistance is ignored);

d) bond is perfect between steel reinforcement and adjacent concrete;

e) maximum compressive strains in masonry (ε_m) and concrete (ε_c) shall be taken equal to 0.004 0 and 0.003 0, respectively; and

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the stress-strain curve of masonry is linear, unless a more accurate relation is determined by experimental study of masonry prisms.

9.2.2.8 *Stress-strain curves for concrete, masonry and steel*

The relationship between the compressive stress distribution in concrete and the strain in concrete may be assumed to be rectangle, trapezoid, parabola, or any other shape which results in prediction of strength in substantial agreement with the results of test. An acceptable stress-strain curve is given in Part C/Section 5 of this NBCS. But the ultimate strain in concrete shall be taken as 0.003 0 in place of 0.003 5. For design purposes, the compressive strength of concrete in the structure shall be assumed to be 0.67 times the characteristic strength. The partial safety factor γ_m of 1.5 shall be applied in addition to this.

The stresses in the reinforcement are derived from representative stress-strain curve for the type of steel used. Typical curves are given in Part C/Section 5 of this NBCS. For design purposes, the partial safety factor γ_m of 1.15 shall be applied.

The relation between the compressive stress-strain in masonry may be assumed to be rectangle, trapezoid, parabola, or any other shape which results in prediction of strength in substantial agreement with the results of test. An acceptable stress-strain curve is given in Fig. 56. For design purposes the partial safety factor γ_m of 2.0 shall be applied.

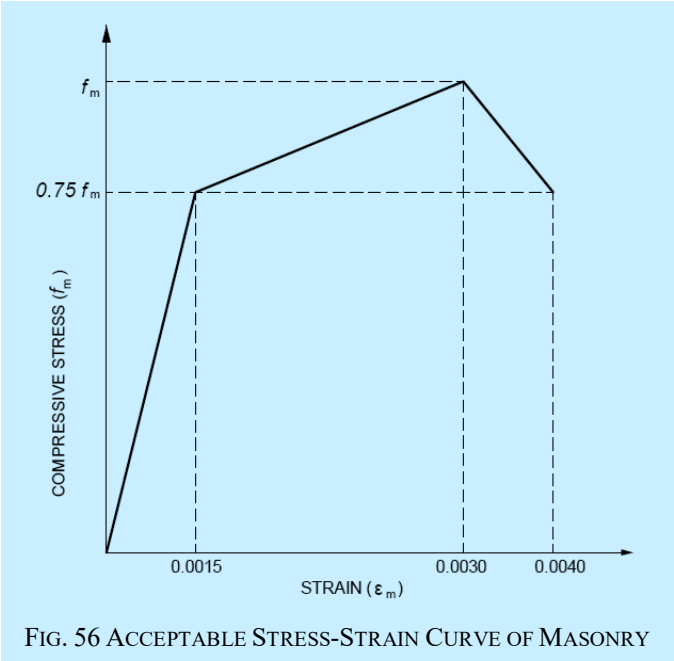

FIG. 56 ACCEPTABLE STRESS-STRAIN CURVE OF MASONRY

9.2.2.9 *Axial load resistance*

Axial load resistance of a confined masonry wall (Pu) shall be determined considering the contribution of masonry and longitudinal steel reinforcement in tie-columns assuming that the steel has yielded in compression, that is:

$$P_u = (0.4\, f_m\, A_m + 0.45\, f_{ck}\, A_c + 0.75\, f_y\, A_s)\, k_s,$$

where

f_m = compressive strength of masonry;

A_m = net area of masonry;

f_{ck} = characteristic cube compressive strength of concrete;

A_c = cross-sectional area of concrete excluding reinforcing steel;

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f_y = characteristic yield strength of the reinforcing steel;

A_s = area of steel; and

k_s = stress reduction factor as in Table 10.

9.2.2.10 *Design of confined masonry walls for combined axial load and out-of-plane bending*

Confined masonry walls need to be designed for combined effects of axial load and out-of-plane bending (bending perpendicular to the wall surface). Bending moments may be due to eccentric gravity loads or lateral loads acting perpendicular to wall plane (wind and earthquake). Generally, confined masonry walls have sufficient resistance against earthquake force acting in out-of-plane direction if slenderness limits in **9.4.1** are satisfied. For out-of-plane forces, only one-way bending may be considered, that is, bending in the vertical direction as shown in Fig. 57.

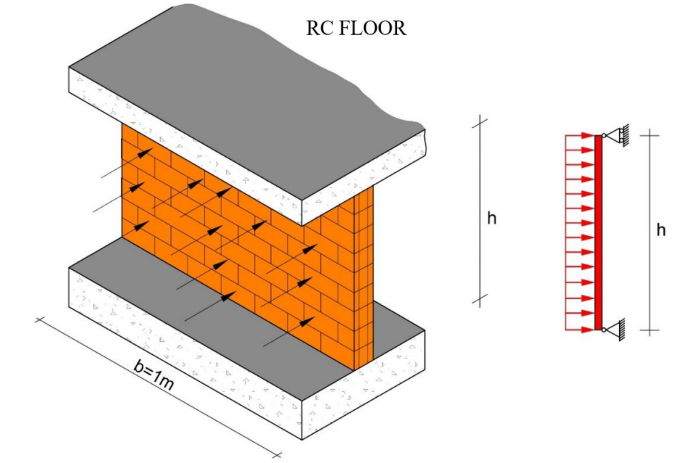

FIG. 57 UNIT LENGTH B OF MASONRY WALL CONSIDERED FOR OUT-OF-PLANE BENDING IN VERTICAL DIRECTION

The interaction of compression due to axial stresses and bending stresses should be within the following limits:

$$\frac{P}{P_u} + \frac{M}{M_u} \leq 1.0$$

Design bending moment demand M due to design wind pressure p_d [as per Part C/Section 1 of this NBCS] acting out-of-plane over unit length b can be calculated as follows:

$$M = \frac{p_d h^2}{8}$$

where h is floor-to-floor height. The axial resistance P_u shall be calculated as per **9.2.2.9** by ignoring the contribution of concrete and reinforcement in tie-column and considering the net area of masonry per unit length A'_m. The moment of resistance M_u per unit length shall be estimated for bending about horizontal axis normal to the length of wall as follows:

$$M_u = \left(\frac{f_t}{\gamma_m} + \frac{P}{A'_m} \right) S'_m$$

where axial load P should not exceed self-weight of the unit length of panel plus $0.15A'_m$ (in MPa),

where

f_t = flexural tensile strength normal to bed joints;

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$$= \begin{cases} 0.3 \text{ MPa} & \text{Grade M1 or better Mortar} \\ 0.2 \text{ MPa} & \text{Grade M2 Mortar} \end{cases}$$

A'_m = net area of masonry per unit length (mm^2); and

S'_m = section modulus per unit length (mm^3).

9.2.2.11 *Moment resistance of confined masonry walls due to combined axial load and in-plane bending*

a) *General approach* — Moment resistance M_u for a section of a confined masonry wall subjected to the design axial load P shall be determined from equilibrium of internal forces acting on the section, by following the assumptions stated in **9.2.2.7**; and

b) *Simplified axial load and bending moment interaction diagram* — For walls with longitudinal bars placed symmetrically in tie-columns, the following equations define a simplified axial load-bending moment interaction diagram (Fig. 58):

$$M_u = \begin{cases} 0.3Pd + M_{uf} & \text{Points A-B :: } 0 \le P < (P_u/3) \\ 0.10Pd + M_{uf} & \text{Point B :: } P = (P_u/3) \\ \frac{2}{3}(0.05Pd + 1.5M_{uf}) & \text{Point C :: } P = (P_u/3) \\ \left(1 - \frac{P}{P_u}\right)(0.15Pd + 1.5M_{uf}) & \text{Points A-B :: } P > (P_u/3) \end{cases}$$

where

M_f = $0.87 f_y A_s (l_w - b)$ is the moment resistance corresponding to pure bending load condition, and d the effective depth of the wall section.

c) *Alternate method* — The P-M interaction curve for the confined masonry section can also be determined by sectional analysis method employed in reinforced concrete members.

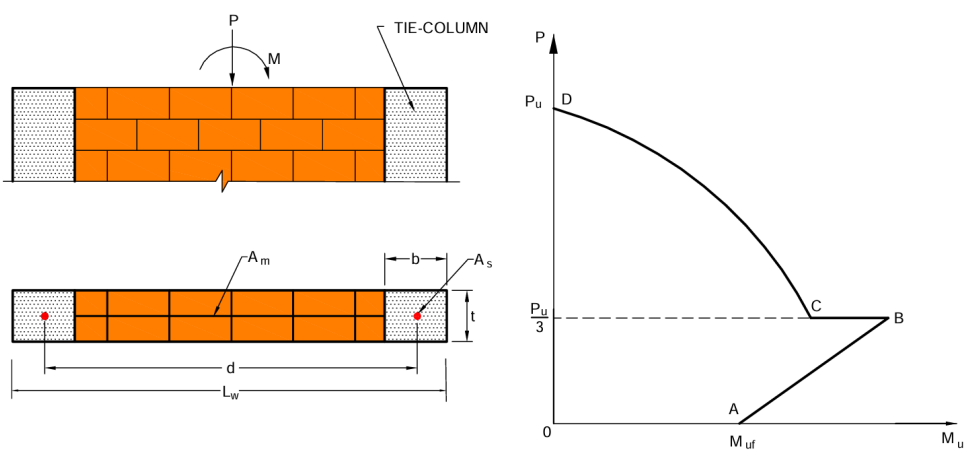

FIG. 58 SIMPLIFIED INTERACTION DIAGRAM FOR DESIGN OF CONFINED MASONRY WALLS

9.2.2.12 *Design for shear*

Shear resistance of a confined masonry wall is due to combined contribution of masonry and horizontal wall reinforcement. Although reinforced concrete tie-columns contribute to shear resistance of a confined masonry wall, their contribution shall be ignored in the design of confined masonry structure.

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a) *Masonry shear resistance*

1) *Empirical expression method* — Shear strength of masonry wall may be estimated by any of the following methods:

Shear resistance V_u provided by masonry wall shall be determined as:

$$V_u = 0.8(0.5v_m A_T + 0.4P_d)f \leq 1.5v_m A_T$$

where

A_T = area of cross-section of confined masonry wall including tie-columns;

f = corrected factor for the aspect ratio H/L of the wall;

$$= \begin{cases} 1.55 & \left(\frac{H}{L}\right) \leq 0.2 \\ 1.70\text{-}0.7\left(\frac{H}{L}\right) & 0.2 < \left(\frac{H}{L}\right) \leq 1.0 \\ 1.00 & 1.0 < \left(\frac{H}{L}\right) \end{cases};$$

P_d = design compressive axial load which shall include permanent loads only and with the partial safety factor of 1.0;

v_m = masonry shear strength; and

$= 0.16\sqrt{f_m} \leq 0.6$ MPa.

2) *Test method* — Shear resistance V_u of masonry wall shall be determined as:

$$V_u = v_m A_T$$

where, v_m can be obtained from the standard test method for diagonal tension (shear) in masonry assemblages as given hereunder:

i) When shear strength v_m of masonry is to be established by tests, it shall be done in advance of the construction, using masonry specimens built of similar materials under the same conditions with the same bonding arrangement as for the structure. In making the specimens, moisture content of the units at the time of laying, the consistency of the mortar, the thickness of mortar joints and workmanship shall be the same as will be used in the structure.

The walls specimens (Fig. 59) shall have a length at least two and a half times the length of the masonry unit in one direction and adequate number of courses in perpendicular direction so that the specimen has approximately square shape. The sides of the specimen may not be less than 600 mm. The walls specimens shall be tested by loading them in compression along one diagonal. Tests shall be conducted on at least three specimens constructed with the same size and type of masonry units, mortar, and workmanship.

Specimens shall be tested after 28 days and two steel loading shoes (Fig. 59) shall be used to apply the machine load to the specimen. For the distribution of the applied load P_s, the ratio of the length of bearing, l_b and the width or height of specimen must be equal to or greater than 0.2. The load shall be applied at the rate of 50 kN/min to 100 kN/min and the load at failure shall be recorded. Load shall be applied at a uniform rate so that the maximum load is reached in not less than 1 min and not more than 2 min. Minimum thickness of steel plate used for loading shoe shall be 12 mm.

ii) The shear stress strength v_m of the specimen as:

$$v_m = \frac{P_s}{A_{sn}\sqrt{2}}$$

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where

A_{sn} = net area (mm^2) of the wall specimen;

= $\left(\frac{w_s + n_s}{2}\right) t_s$;

P_s = applied design axial load (N);

h_s = height (mm);

t_s = total thickness (mm) of the wall specimen; and

w_s = width (mm).

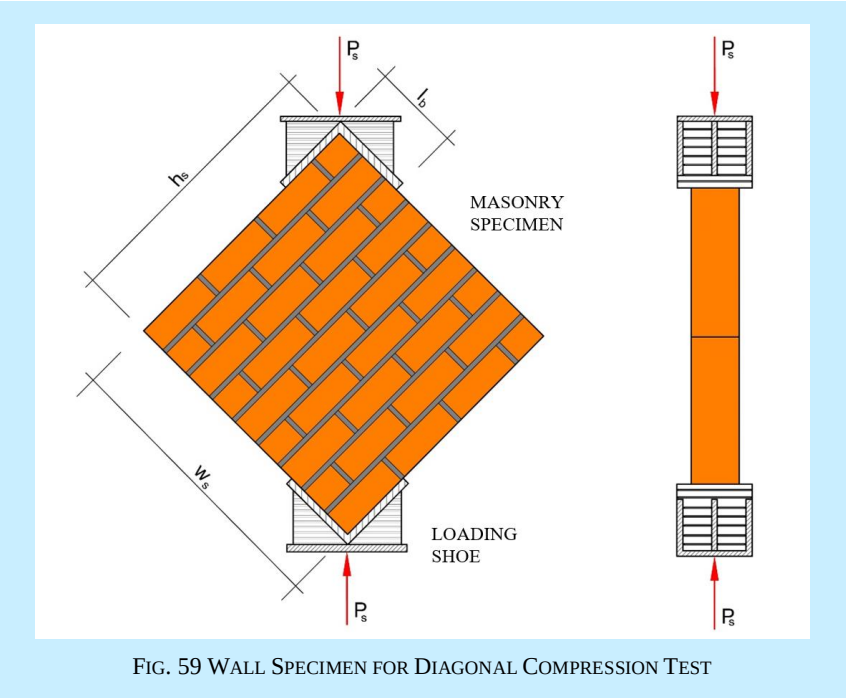

FIG. 59 WALL SPECIMEN FOR DIAGONAL COMPRESSION TEST

b) *Alternate method* (*Strut-and-Tie method*)

The Strut-and-Tie method can be used for determining the shear capacity of confined masonry walls. This method provides a rational and consistent design approach by idealizing complex structural members with an appropriate simplified truss model. According to this procedure, based on the knowledge of direction of principal stresses, load paths are drawn through the wall in form of a truss which is analyzed for the design loads. A possible strut-and-tie model for the confined masonry wall based on its load resisting mechanism is illustrated in Fig. 60; the broken and solid lines represent the strut and tie, respectively. If the opening distribution is irregular or complex in elevation, strut and tie method can be used to estimate the in-plane resistance of the confined masonry wall.

The basic prerequisites for the strut-and-tie model are as follows:

a) The equilibrium of forces at nodes should be maintained for a given set of loads;

b) Tension in concrete and masonry is neglected and adequate detailing of anchorage shall be provided for tie reinforcement;

c) The member forces in the struts and ties are uniaxial which shall not exceed the corresponding member strength at every section; and

d) Sufficient ductility shall be available to make the transition from elastic to plastic behaviour which will enable redistribution of internal forces in the members.

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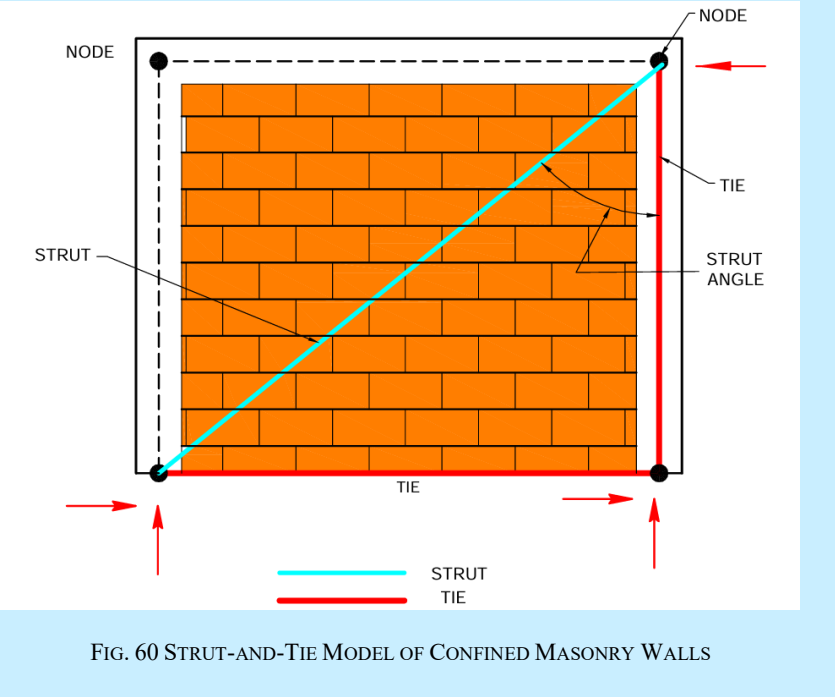

FIG. 60 STRUT-AND-TIE MODEL OF CONFINED MASONRY WALLS

9.2.2.13 *Design of tie-columns and tie-beams*

a) *Minimum amount of longitudinal reinforcement*

Longitudinal reinforcement in tie-columns and tie-beams shall be proportioned to resist the corresponding vertical and horizontal components of the compression strut that develops in masonry when resisting combined gravity and lateral loads. The total area of reinforcement shall be not less than 0.8 percent of the gross cross-section area of the column. For a 2-storey height, the minimum area of reinforcement can be 0.6 percent;

b) *Minimum amount of transverse reinforcement* (*Ties*)

Tie-columns and tie-beams shall have transverse reinforcement in the form of closed stirrups (ties) with the minimum total cross-sectional area A_{sc} (mm^2) of stirrup legs effective in shear equal to:

$$A_{sc} = 0.001\ 2\ sh_{c,}$$

where

s = tie spacing, in mm; and

h_c = Dimension, in mm of tie-column or tie-beam in the wall plane.

c) *Spacing of transverse reinforcement*

Tie spacing shall not exceed the lesser of 200 mm and 1.5t.

9.2.3 *Foundations and Plinth Construction*

The foundation shall be constructed in a similar manner as for traditional masonry construction. Either a brick masonry footing or an RC strip footing can be used. An RC plinth band shall be constructed on top of the foundation. A plinth band shall be provided to fully confine wall panels along their bases and prevent excessive wall damage due to building settlement in soft soil areas. Note that the longitudinal reinforcement shall be extended from an RC tie-column into the plinth band, and whenever possible, into the foundation.

9.3 Earthquake Resistant Design

9.3.1 *Basis*

Building shall be designed for earthquake forces determined as per **5** of Part C/Section 1 of this NBCS. Also, the load combinations shall be as per Part C/Section 1 of this NBCS.

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9.3.2 *Methods of Earthquake Analysis*

9.3.2.1 *General considerations*

9.3.2.1.1 For determining internal forces and moments acting on confined masonry walls, the building may be analyzed for estimating the effects of earthquake shaking using either the simplified method or the earthquake analysis methods specified in **5** of Part C/Section 1 of this NBCS. The simplified method can be used for buildings up to three storeys high, provided that the requirements stipulated in **9.3.2.2** have been met.

The effect of openings shall be considered in lateral stiffness and strength calculations. The structure shall be modelled to adequately simulate the behaviour of its critical structural elements. Modulus of elasticity E_m and shear modulus G_m of masonry shall be used to reflect the axial and shear stiffness expected in the structure.

9.3.2.1.2 Design forces and moments shall be obtained from the analysis using design loads and the corresponding load combinations specified in **5** of Part C/Section 1 of this NBCS. A confined masonry wall shall be designed for the effects of gravity loads, and the effects of lateral loads, including shear force, in-plane bending, and out-of-plane bending moments. When the simplified method is used, design for lateral loads may be limited to the effects of shear force.

9.3.2.2 *Applicability of the simplified method*

9.3.2.2.1 The simplified method is based on an idealized distribution of lateral earthquake forces in regular load bearing wall buildings with rigid diaphragms. The method compares the shear capacity of all walls at the base of the building and the earthquake base shear force determined from **5** of Part C/Section 1 of this NBCS. It is suitable for verifying adequacy of regular buildings with symmetrical wall layout and predominant shear behaviour. The method does not consider torsional effects, arising from eccentricity between centre of mass and centre of resistance. Due to its assumptions, application of this method is restricted to buildings with a height not exceeding three storeys.

9.3.2.2.2 The design can be simplified further for small buildings by applying the concept of wall index, as per procedure in **9.3.2.3**, which can be used for the design of buildings with the total ground plan area not exceeding 200 m^2 and the height not exceeding two storeys or 7 m.

9.3.2.3 *Wall index method*

9.3.2.3.1 Wall Index (*WI*), also known as wall density, shall be used as an indicator of earthquake load-resisting capacity of earthquake resistant design of low-rise buildings with regular plan shape and elevation. The design according to the simplified method shall be deemed satisfactory provided that the actual wall index (WI_{floor}) per floor in each direction of the building plan is greater than or equal to the required Wall Index (WI_{req}) for specific building.

Wall index WI_{floor} at the ground floor shall be taken as ratio of the sum of cross-sectional areas of all confined masonry walls at that level in the considered direction of earthquake shaking, given by:

$$WI_{Floor} = \frac{A_W}{A_p}$$

where

A_w = cross-sectional area of all confined walls in one direction at the ground floor level; the cross-sectional area of a confined masonry wall is the product of its length (including the reinforced concrete tie-columns and the masonry wall) and thickness; and

A_p = plan area of the floor diaphragm (floor slab) at the ground floor level; the area of any cutouts (openings) in the floor slab shall be deducted in the calculation of A_p.

Parameters for wall index calculations are illustrated in Fig. 61. The wall index shall be estimated along each principal plan direction (X and Y) of the building. Only confined masonry walls aligned in the direction under consideration (shown shaded in Fig. 61) shall be considered in the A_w calculations for the specific direction.

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9.3.2.3.2 The required wall index WI_{req} in buildings in different earthquake zones shall be taken as per Table 28 or estimated as follows:

$$WI_{req} \geq \left(\frac{1.5 A_h w}{v_m / \gamma_m}\right),$$

where

A_h = design horizontal acceleration coefficient determined as per **5** of Part C/Section 1 of this NBCS for different zones;

n = number of floor levels in a building;

v_m = masonry shear strength as per **9.2.2.12**;

γ_m = partial safety factor for masonry as per **9.2.2.8**; and

w = average floor weight, including the self-weight of floor slab, floor finish, walls and 25 percent of the imposed load.

9.3.2.4 *Earthquake analysis methods*

Earthquake analysis of confined masonry buildings of four-storey or higher and buildings up to three-storey high that do not meet requirements for the application of simplified method shall be analysed by the method specified in **5** of Part C/Section 1 of this NBCS. Lateral load effects induced by earthquake shall be determined based on the relative stiffness of walls and wall segments, by considering the effect of both shear and flexure in stiffness calculations. The analysis model shall consider stiffness of floor and roof systems, and any other restraints that may influence wall rotations.

9.3.3 *Other Prescriptive Requirements*

9.3.3.1 *Wall spacing*

The maximum spacing of transverse walls in buildings with flexible diaphragms shall not exceed 4 m. For spacing more than 4 m, the tie-beam at roof level should have a width of $L/20$.

9.3.3.2 *Spacing of transverse reinforcement* (*ties*) *in tie-columns*

In earthquake zones IV and above reduced tie spacing ($s/2$) shall be provided of the transverse reinforcement at the ends of tie-columns, where s is the spacing of transverse reinforcement specified *in* 9.2.2.13(c) as shown in Fig. 62. The length over which the reduced tie spacing is required shall not exceed the largest of ($2h_c$), ($2t$) and ($h_o/6$), where h_c is the depth of the tie-column, h_o the clear height of tie-column between floors, and t the width of the tie-column.

9.4 General Requirements

9.4.1 *Reinforcement Detailing*

9.4.1.1 *Concrete cover*

a) The minimum concrete cover to ties (transverse reinforcement) in tie-columns and tie-beams shall not be less than 20 mm; and

b) When horizontal wall reinforcement is provided, the minimum clear distance between a horizontal bar and the exterior wall surface shall not be less than 20 mm.

9.4.1.2 *Bar size and number in tie-columns and tie-beams*

Longitudinal reinforcement in tie-columns and tie-beams shall consist of a minimum of 4 reinforcing bars with the minimum 10 mm diameter.

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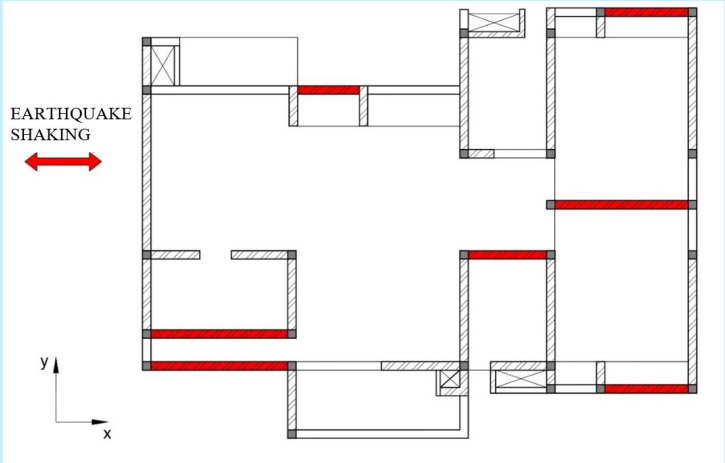

FIG. 61(A) X-DIRECTION

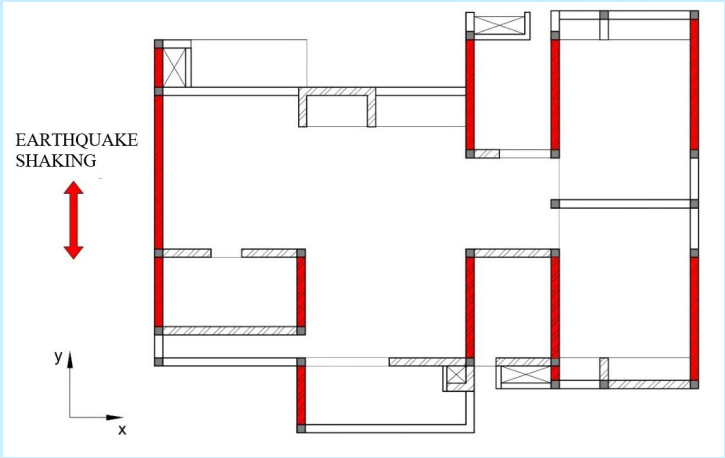

FIG. 61(B) Y-DIRECTION

FIG. 61 WALLS TO BE CONSIDERED IN ESTIMATION OF WALL INDEX ALONG PRINCIPAL PLAN DIRECTIONS OF A BUILDING

Table 28 WI_{req} for Different Earthquake Zones

(*Clause* 9.3.2.3.2)

Sl No.	Earthquake Zone	II	III	IV	V
(1)	(2)	(3)	(4)	(5)	(6)
i)	WI_{req} (percent)	0.7	1.1	1.6	2.4

9.4.1.3 *Ties in tie-columns and tie-beams*

Minimum 6 mm diameter bars shall be used for ties in tie-columns and tie-beams (Fig. 63). Ties shall have 135° hooked ends. Hooks shall be staggered as shown in Fig. 63.

9.4.1.4 *Anchorage of longitudinal bars*

a) Longitudinal reinforcement in tie-columns and tie-beams shall be anchored to develop the full specified steel yield stress. Longitudinal bars in tie-columns and tie-beams shall have a 90° hooked anchorage at intersections. The lap length of the hook tails shall be the largest of 20 times the bar diameter or 500 mm (Fig. 64); and

b) Tie-column longitudinal bars at the roof level shall be bent by 90° and lapped with the tie-beam longitudinal reinforcement (Fig. 65).

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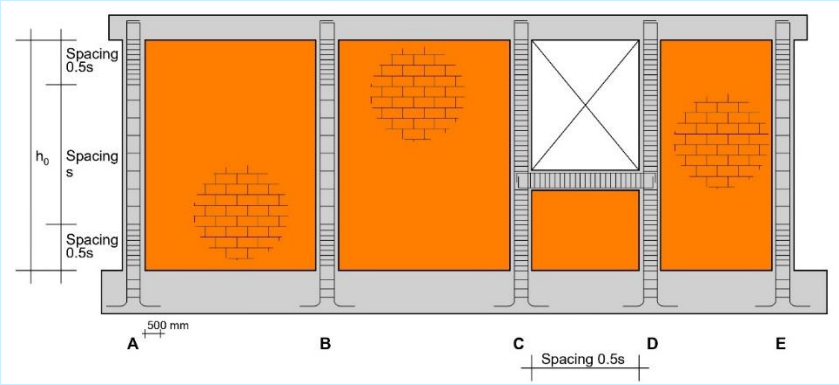

FIG. 62 TIE-COLUMN TRANSVERSE REINFORCEMENT REQUIREMENTS IN EARTHQUAKE ZONES IV AND ABOVE

9.4.1.5 *Lap Splices for longitudinal reinforcement*

Lap splices for longitudinal reinforcement should be at least 40 times the bar diameter. Longitudinal reinforcing bars shall be spliced within the middle third of the column height or beam span. The splices shall be staggered so that not more than 2 bars are spliced at any one location.

9.4.1.6 *Tie-column-to-tie-beam joints*

Continuity of longitudinal tie-beam reinforcement through the joint must be ensured. An example of a continuous longitudinal reinforcement is shown in Fig. 66. First tie at the ends of tie-columns (top and bottom) shall be placed as close to the joint as possible. When tie-beam depth exceeds 300 mm, vertical reinforcement in an RC tie-column must be confined by the ties, below and above the joint. An additional U-shaped stirrup shall be placed in the tie-beam at mid-height (Fig. 67).

Reinforced concrete tie-columns shall be provided at the openings as shown in Fig. 54(B) and Fig. 54(C). A nominal reinforcement around openings as detailed in Fig. 53 shall be provided. When reinforced concrete tie-columns are not provided at the ends of an opening then they should be strengthened as per good practice of [C-4(10)] or minimum two vertical bars of 10 mm diameter with 6 mm ties at 150 mm spacing should be provided.

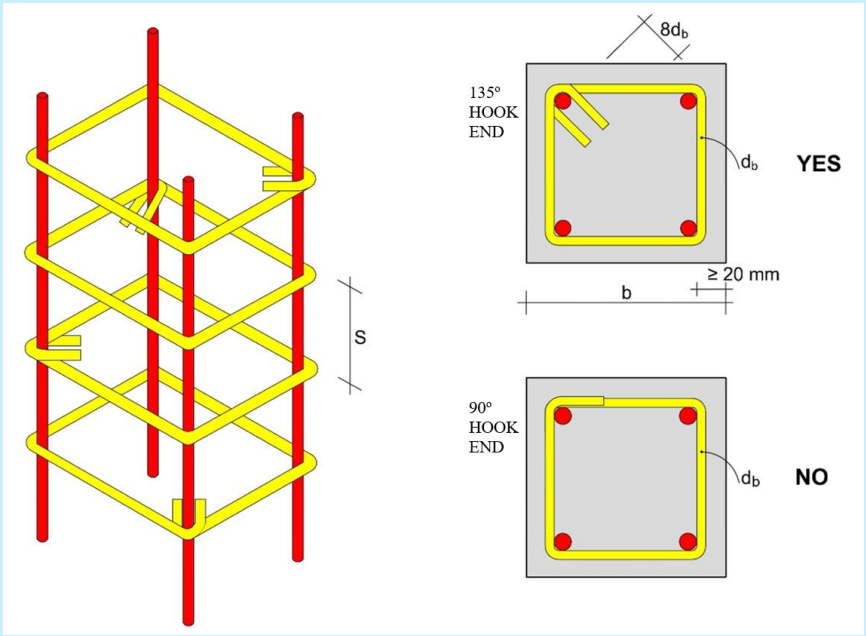

FIG. 63 LAYOUT AND DETAILING OF TIE-COLUMNS AND TIE-BEAMS

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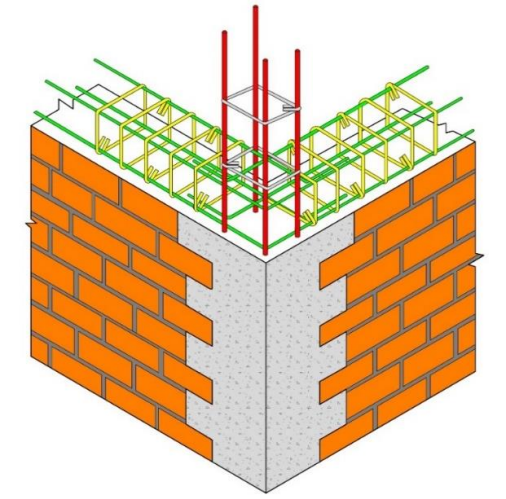

FIG. 64(A) WALL INTERSECTION

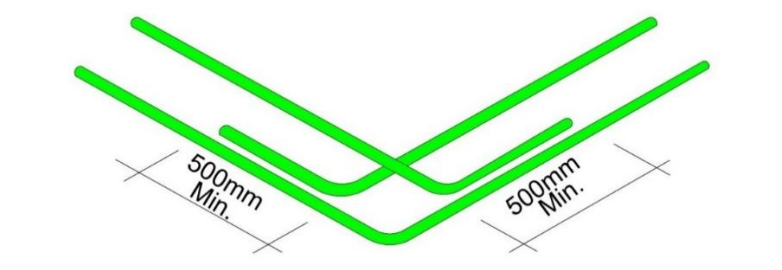

FIG. 64(B) HOOKED ANCHORAGE OF LONGITUDINAL REINFORCEMENT AT WALL INTERSECTIONS

FIG. 64 TIE-BEAM CONSTRUCTION

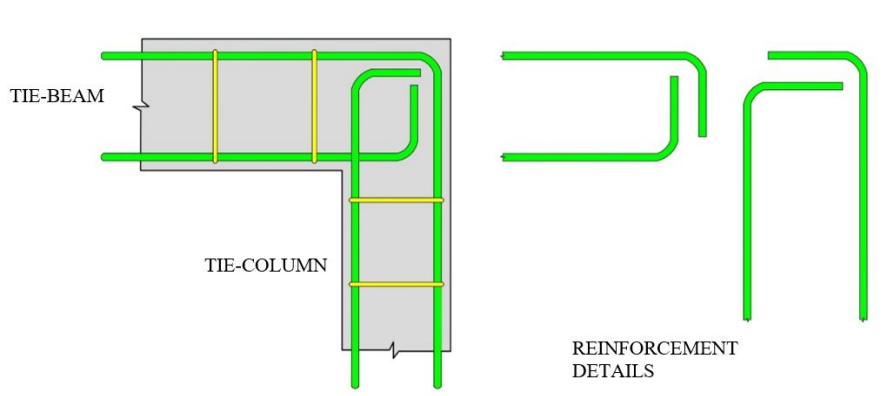

FIG. 65 ANCHORAGE OF LONGITUDINAL REINFORCEMENT OF TIE-COLUMN AND TIE-BEAM

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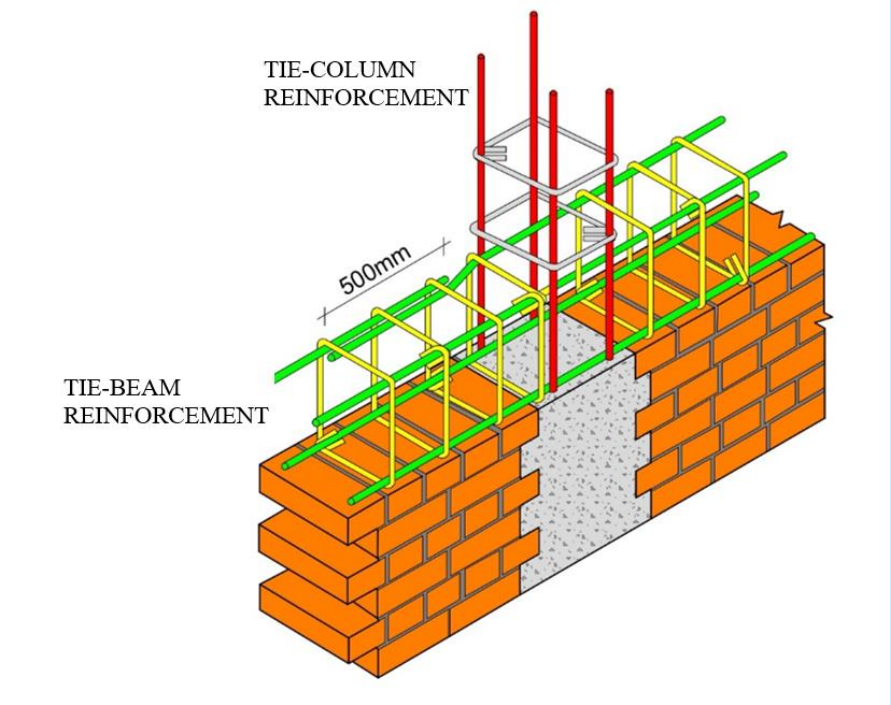

FIG. 66 CONTINUITY OF TIE-BEAM REINFORCEMENT THROUGH JOINT

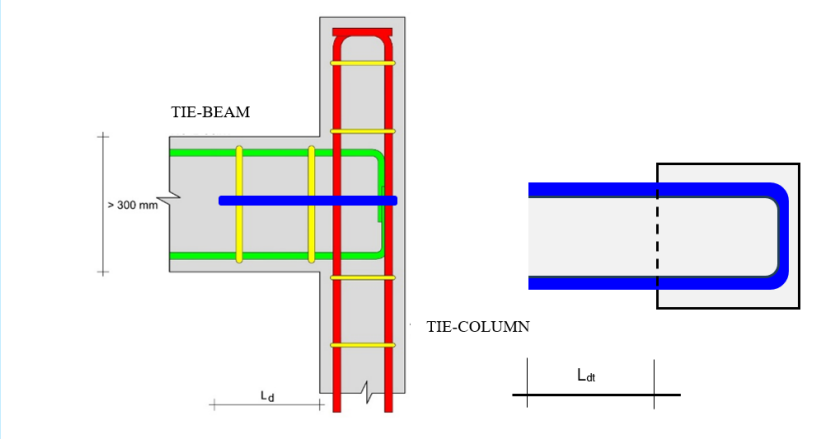

FIG. 67 ADDITIONAL CONFINEMENT OF VERTICAL REINFORCEMENT IN TIE-BEAMS AND TIE-COLUMNS IN EXTERIOR JOINT

9.4.2 *Construction*

Construction of confined masonry buildings shall satisfy the conditions specified hereunder.

9.4.2.1 *Masonry wall construction*

a) *General* — During construction, in addition to the requirements of the sections above, the following shall be complied:

1) all intersecting walls shall be connected, unless measures to assure stability and good performance are taken;

2) surfaces of construction joints shall be clean and rough. The joint shall be moistened before construction when clay units are used;

3) fulfill the requirements of excellent masonry work, Flemish bond or English bond in brickwork shall be adopted; and

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4) dring construction, necessary caution to assure wall stability at the job, shall be taken, possible horizontal pressure and loads, including wind and earthquake shall be considered.

b) *Masonry units*

The shape and dimension of masonry units, construction practices, including methods of positioning of reinforcement, placing and compacting of grout, as well as design and detailing shall be, such as to promote homogeneity of structural members, development of the bond between the grout to both reinforcement and masonry units and avoidance of corrosion of reinforcement.

c) *Mortar joint thickness*

Joint mortar shall totally cover the horizontal and vertical sides of the masonry unit. Joint thickness shall be the minimum to get a uniform layer of mortar and good alignment of the units. If industrialized units are used, the thickness of the horizontal joints shall not exceed 12 mm if horizontal reinforcement is placed in the joints, and 10 mm without horizontal reinforcement. Minimum thickness shall be 6 mm.

d) *Toothing*

The vertical interface of masonry with adjacent tie-columns shall be detailed to transfer shear forces. It shall be accepted to indent the masonry (referred to as toothing) or else, to place steel dowels or horizontal reinforcement as shown in Fig. 68.

Toothed edges shall be left on each side of the wall at the interface with the tie-columns. Toothing length shall be equal to one-quarter of the masonry unit length, but not less than 50 mm, as shown in Fig. 68(A). When hand-made bricks are used, it is desirable to cut the brick edges, as shown in Fig. 68(B). It is important to clean the surfaces of 'toothed' masonry units before the concrete has been poured. Horizontal reinforcement anchored into RC tie-columns, also known as dowels, can be used as an alternative no toothing, as shown in Fig. 68(C). The dowels are not necessary when toothed edges are used. 6 mm diameter bars shall be preferably used as dowels.

9.4.2.2 *Reinforcement*

Reinforcement shall be tightly secured before casting.

9.4.2.3 *Concrete*

Casting of tie-columns shall be cast after the masonry wall or the corresponding part has been constructed. The masonry around the tie-columns shall be saturated before the concrete has been poured.

9.4.2.4 *Piping and ducts*

Piping and ducts shall be installed without damaging the masonry. If solid or grouted hollow units are used, grooves in the wall shall be permitted to embed piping and ducts, but the following shall be satisfied:

a) Groove depth shall not exceed one fourth the thickness of the masonry wall (t/4);

b) Groove shall be vertical; and

c) Groove shall not be longer than one half the free height of the wall (H/2). If hollow units are used, pipes or ducts shall not be placed in cells with reinforcement. Cells with pipes and ducts shall be grouted.

It shall not be permitted to place piping and ducts in tie-columns with a structural function, whether external or internal.

10 REINFORCED MASONRY WALLS (RMW)

10.1 General

The requirements for structural design aspect of reinforced load bearing/walls, constructed with solid or perforated burnt clay bricks, sand-lime bricks, stones, concrete blocks, lime-based blocks or burnt clay hollow blocks shall be specified hereunder related to the materials to be used, the permissible stresses and methods of design.

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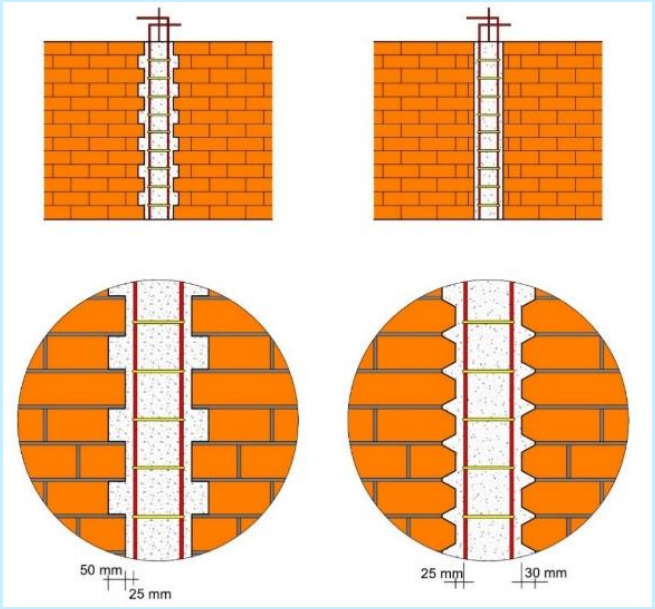

FIG. 68(A) FULL TOOTHING FIG. 68(B) PARTIAL TOOTHING

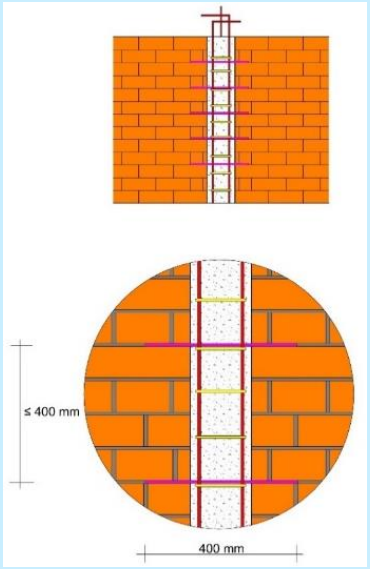

FIG. 68(C) NO TOOTHING

FIG. 68 REINFORCEMENT DETAILS OF TOOTHING IN CONFINED MASONRY WALLS

10.2 Design Considerations

10.2.1 *General*

Design considerations shall be as per geometrical requirements of 4 along with the following provisions.

10.2.2 *Structural Continuity*

Intersecting structural elements intended to act as a unit shall be joined together to resist the design forces. Walls shall be joined together to all floors, roofs or other elements which provide lateral support for the wall. Where floors or roofs are designed to transmit horizontal forces to walls, the anchorages to the walls shall be designed to resist the horizontal forces.

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10.2.3 *Effective Span*

10.2.3.1 Effective span of simply supported or continuous members shall be taken as the smaller of the following:

a) Distance between centers of supports; and

b) Clear distance between supports plus an effective depth *d*.

10.2.3.2 Effective span of a cantilever shall be taken as greater of:

a) distance between the end of cantilever and;

b) centre of its support; and

c) face of support plus half its effective depth.

10.2.4 *Slenderness Ratio*

10.2.4.1 *Wall*

The slenderness ratio (ratio of effective height h_{eff} to effective thickness t_{ef}) shall not exceed 27 for vertically loaded reinforced masonry walls in their plane. For reinforced masonry members, such as walls subjected to out-of-plane bending and for spandrel masonry beams that are a part of wall subjected to bending in the plane of the wall, the maximum effective span to effective depth ratio shall be as given in Table 29.

10.2.4.2 *Columns*

For a column, slenderness ratio shall be taken to be the greater of the ratios of effective heights to the respective effective thickness in the two principal directions. Slenderness ratio for a load bearing unreinforced column shall not exceed 15 whereas for reinforced column the slenderness ratio should be limited to 20.

10.2.5 *Minimum Design Dimensions*

10.2.5.1 *Minimum thickness of load bearing walls columns*

The nominal thickness of masonry bearing walls in building shall not be less than 230 mm.

10.2.5.2 *Parapet wall*

Parapet walls shall be at least 200 mm thick and height shall not exceed 3 times the thickness. The parapet wall shall not be thinner than the wall below. The tie-columns shall be extended into the parapet wall and terminated at the top of the parapet in a tie-beam.

10.2.6 *Eccentricity in Columns*

Columns shall be designed for a minimum eccentricity of 10 percent of side dimension for each axis in addition to applied loads.

10.3 Requirements Governing Reinforcement and Detailing

10.3.1 *General*

This section provides requirements for the working (allowable) stress design of masonry structure neglecting the contribution of tensile strength of masonry.

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Table 29 Maximum Ratio Of Effective Span to Effective Depth For Walls Subjected to Out-Of-Plane Bending

(*Clause* 10.2.4.1)

Sl No.	Boundary Conditions	Maximum Permissible Ratios	
		Effective Span to Effective Thickness (l_{ef}/t_{ef}) of Wall Subjected to Out-of-Plane Bending	Effective Span to Effective Depth (l_{ef}/d) of Beam Part of Wall Subjected to In-Plane Bending
(1)	(2)	(3)	(4)
i)	Simply supported	35	20
ii)	Continuous	45	26
iii)	Spanning in 2 directions	45	—
iv)	Cantilevered	18	7

10.3.1.1 Members are designed for composite action. Stresses shall be computed using transformed area concept of linear elastic analysis as follows:

$$A_t = A_b + m\,A_s$$

where

A_t = total transformed cross-sectional area of the member;

A_b = cross sectional area of brick;

m = modular ratio of steel reinforcement and brick; and

A_s = cross sectional area of reinforcement.

10.3.1.2 Stiffness calculation shall be based on un-cracked section properties.

10.3.2 *Steel reinforcement-allowable stresses*

10.3.2.1 *Tension*

Tensile stress in reinforcement shall not exceed the following:

a) 140 MPa in case of Mild Steel (MS) bars of diameter ≤ 20 mm and 130 MPa in case of MS bars of diameter > 20 mm; and

b) $0.55f_y$ in case high strength bars, where f_y is the characteristic strength of steel.

10.3.2.2 *Compression*

Compressive stress in reinforcement shall not exceed the following:

a) 130 MPa in case of MS bars; and

b) 190 MPa in case high strength bars.

10.3.3 *Size of Reinforcement*

a) The maximum size of reinforcement used in masonry shall be 25 mm diameter bars and minimum size shall not be less than 8 mm; and

b) The diameter of reinforcement shall not exceed one-half the least clear dimension of the cell, bond beam, or collar joint in which it is placed.

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10.3.4 *Spacing of Reinforcement*

a) In beams, clear distance between parallel bars shall not be less than the diameter of the bars, or less than 25 mm; and

b) In columns and pilasters, clear distance between vertical bars shall not be less than 1.5 times the bar diameter, nor less than 35 mm.

10.3.5 *Anchorage*

10.3.5.1 *Development length of bars*

The development length L_{dtm} shall be taken as:

$$L_{\text{dtm}} = \begin{cases} 0.25 d_{\text{b}} F_{\text{s}} \geq 300 \text{ mm} & \text{Fe415, Fe500 and Fe550} \\ 0.40 d_{\text{b}} F_{\text{s}} \geq 300 \text{ mm} & \text{Fe250} \end{cases}$$

where

d_b = nominal diameter of bar (mm); and

F_s = permissible tensile/compressive stress in steel (MPa).

10.3.5.2 For MS bars, L_d shall be increased by 60 percent.

10.3.5.3 *Standard hooks*

a) Standard hooks shall be formed by one of the following methods (Fig. 69):

1) 180° turn plus extension of at least 4 bar diameters but not less than 64 mm at free end of bar;

2) A 90° turn plus extension of at least 10 bar diameters at free end of bar; and

3) For stirrup and tie anchorage only a 90° or a 135° turn plus an extension of at least 5 bar diameters at the free end of the bar.

b) The diameter of bend, measured to the inside of the bar other than stirrups and ties, shall not be less than 5 bar diameters for 6 mm through 20 mm diameter bars. For 25 mm bars, a minimum bend diameter of 6 times bar diameters shall be used;

c) Inside diameter of bend for 12 mm diameter or smaller stirrups and ties shall not be less than 4 bar diameters. Inside diameter of bend for 16 mm diameter or larger stirrups and ties shall not be less than that given in (b); and

d) Hooks shall not be permitted in the tension portion of any beam, except at the ends of simple or cantilever beams or at the freely supported ends of continuous or restrained beams.

10.3.5.4 *Anchorage of reinforcement*

a) Deformed bars may be used without end anchorages provided development length requirement is satisfied. Hooks should normally be provided for plain bars in tension;

b) Bends and hooks shall conform to the requirements of Part C/Section 5 of this NBCS, and the following anchorage values shall be used:

1) *Bend* — The anchorage value of bend shall be taken as 4 times the diameter of the bar for each 45° bend subject to a maximum of 16 times the diameter of the bar,

2) *Hook* — The anchorage value of a standard U-type hook shall be equal to 16 times the diameter of the bar, and of the 135° hook end equal to 12 times diameter of the bar; and

c) For transverse reinforcement complete development length and anchorage shall be deemed to have been provided when the bar is provided with standard hook as described in **10.3.5.2**.

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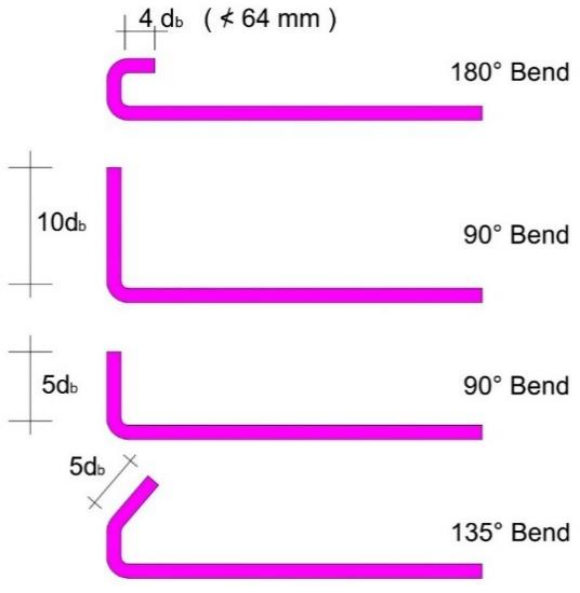

FIG. 69 STANDARD HOOK OF A REINFORCING BAR

10.3.6 *Lap Splices*

10.3.6.1 Where splices are provided in the reinforcing bars, they shall as far as possible be away from the sections of maximum stress and be staggered. It is recommended that splices in flexural members not be at sections where the bending moment is more than 50 percent of the moment of resistance; and not more than half the bars shall be spliced at a section. Where more than one-half of the bars are spliced at a section or where splices are made at points of maximum stress, special precautions shall be taken, such as increasing the length of lap and/or using closely spaced stirrups around the length of the splice.

10.3.6.2 Lap length including anchorage value of hooks for bars in flexural tension shall be L_{dtm} (*see* **10.3.5.1**) or $30d_b$ whichever is greater and for direct tension shall be $2L_{dtm}$ or $30d_b$ whichever is greater. The straight length of the lap shall not be less than $15d_b$ or 200 mm.

10.3.6.3 The following provisions shall also apply:

a) Where lap occurs for a tension bar located at:

   1) top of a section as cast and the minimum cover is less than twice the diameter of the lapped bar, the lap length shall be increased by a factor of 1.4; and

   2) corner of a section and the minimum cover to either face is less than twice the diameter of the lapped bar or where the clear distance between adjacent laps is less than 75 mm or 6 times the diameter of lapped bar, whichever is greater, the lap length should be increased by a factor of 1.4.

b) Where both conditions **10.3.6.3**(a)(1) and **10.3.6.3**(a)(2) apply, the lap length should be increased by a factor of 2.0; and

c) Splices in tension members (which are not part of the lateral force resisting system) shall be enclosed in closed loop hoops made of bars not less than 8 mm diameter with spacing not more than 100 mm.

10.3.7 *Curtailment of Tension Reinforcement*

10.3.7.1 In any member subjected to bending, every reinforcing bar should extend, except at end supports, beyond the point at which it is no longer needed, for a distance equal to the effective depth of the member or 12 times the diameter of the bar, whichever is the greater. The point at which reinforcement is theoretically no longer needed is where the design resistance moment of the section, considering only the continuing bars, is equal to the applied design moment. But, reinforcement should not be curtailed in a tension zone unless at least one of the following conditions is satisfied for all arrangements of design load considered:

a) reinforcing bars extend at least the anchorage length appropriate to their design strength from the point at which they are no longer required to resist bending;

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b) design shear capacity at the section where the reinforcement stops are greater than twice the shear force due to design loads, at that section; and

c) continuing reinforcing bars at the section where the reinforcement stops provides double the area required to resist the bending moment at that section.

10.3.7.2 Where there is little or no end fixity for a member in bending, at least 25 percent of the area of the tension reinforcement required at mid-span should be carried through to the support. This reinforcement may be anchored in accordance with **10.3.5.3** or by providing:

a) an effective anchorage length equivalent to 12 times the bar diameter beyond the centre line of the support, where no bend or hook begins before the centre of the support; or

b) an effective anchorage length equivalent to 12 times the bar diameter plus *d*/2 from the face of the support, where d is the effective depth of the member, and no bend begins before *d*/*2* inside the face of the support.

Where the distance from the face of a support to the nearer edges of a principal load is less than twice the effective depth, all the main reinforcement in a member subjected to bending should continue to the support and be provided with an anchorage equivalent to 20 times the bar diameter.

10.3.8 *Members Subjected to Flexure and Axial Forces*

10.3.8.1 A member which is subjected to axial stress less than $0.1f_m$, may be designed for bending only.

10.3.8.2 *Beams*

Reinforcement in masonry designed as beams shall provided over a support where the masonry is continuous, whether the beam has been designed as continuous or not. Where this occurs, an area of steel not less than 50 percent of the area of the tension reinforcement required at mid-span shall be provided in the top of the masonry over the support and anchored in accordance with **10.3.5.3**. In all cases, at least one quarter of the reinforcement required at mid-span shall be carried through to the support and similarly anchored.

10.3.8.3 *Columns*

The design of reinforced column shall meet the requirements given hereunder.

10.3.8.3.1 Vertical reinforcement shall not be less than 0.25 percent nor exceed 4 percent of the net area of column cross-section. The minimum number of bars shall be four.

10.3.8.3.2 *Lateral ties*

Lateral ties shall be provided in the column as per the following:

a) Longitudinal reinforcement shall be enclosed by lateral ties of at least 8 mm diameter. Vertical spacing of ties along the length of column shall be minimum of:

   1) 16 times diameter of longitudinal bar;

   2) 48 times diameter of lateral tie; and

   3) Least dimension of the column.

b) Arrangement of lateral ties is such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a lateral tie with an included angle of not more than 135º.

10.3.9 *Members Subjected to Shear*

10.3.9.1 Reinforced masonry walls may be designed taking contribution of shear reinforcement.

10.3.9.2 Where contribution of shear reinforcement is considered in resisting shear force the minimum area of shear reinforcement in the direction of force shall be determined by the following:

$$A_{v,\min} = \frac{V_s}{F_s d},$$

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where

V_s = total applied shear force;

F_s = permissible stress in steel reinforcement as per **10.4.1.4**; and

d = distance from extreme compression fiber to centroid of tension reinforcement.

10.3.9.3 The maximum spacing of shear reinforcement shall not be greater than 0.5*d* or 120 mm, whichever is smaller.

10.3.9.4 In cantilever beams, maximum shear shall be used whereas for members subjected to uniformly distributed load it may be assumed that maximum shear load occurs at a distance of 0.5*d* from the face of support when the following conditions are met:

a) support reaction causes compression in the end region of the member; and

b) no concentrated load between face of support and a distance of 0.5*d* from it.

10.3.10 *Reinforcement Detailing*

10.3.10.1 *General*

Reinforcement shall be located such that it acts compositely with the masonry and various ways in which it can be used in reinforced masonry are shown in Fig. 70.

10.3.10.2 *Protection of reinforcement*

Where steel reinforcing bars are embedded in filled cavity (or pockets) or special bond construction, the bars shall have the minimum clear cover of 10 mm in mortar or a minimum clear cover of 15 mm or bar diameter, whichever is more in cement concrete (grout) so as to achieve good bond and corrosion resistance.

For the reinforcement steel placed in mortar bed joint, the minimum depth of mortar cover from the reinforcing steel to the face of masonry shall be 15 mm. Also mortar cover above and below reinforcement placed in bed joints shall not be less than 2 mm.

Reinforcing steel shall be corrosion resistant or protected adequately against corrosion. Reinforcement shall be galvanized reinforcing steel for protection against corrosion.

10.4 Structural Design

10.4.1 *Permissible Compressive Force*

Compressive force in reinforced masonry due to axial load shall not exceed that given by following equation:

$$P_0 = k_s\left(0.25 f_m A_n + 0.65 f_s A_s\right)$$

Where

k_s = stress reduction factor as per Table 10.

f_m = compressive strength of masonry;

A_n = net area of masonry;

f_s = permissible steel tensile stress; and

A_s = area of steel.

10.4.1.1 *Effective compressive width for locally concentrated reinforcement*

When the reinforcement in masonry is concentrated locally such that it cannot be treated as a flanged member (Fig. 71), the reinforced section shall be considered as having a width of not more than:

a) centre-to-centre bar spacing; and

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b) 6 times the wall thickness.

10.4.1.2 *Combined permissible axial and flexural compressive stress*

For reinforced members subjected to combined axial load and flexure, the compressive stress in masonry due to combined action of axial load and bending shall not exceed $1.25f_a$ and compressive stress in masonry due to axial load only shall not exceed f_a. *See* Annex E.

10.4.1.3 *Permissible tensile stress*

Provisions of 5.4.2 shall apply.

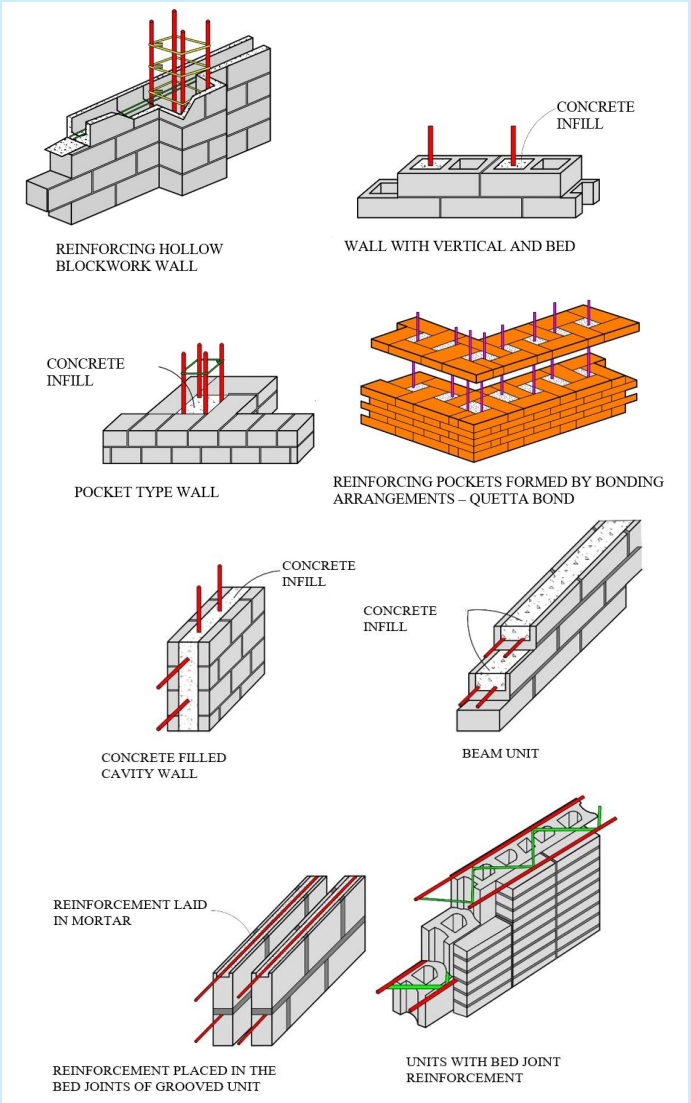

FIG. 70 REINFORCEMENT DETAILS

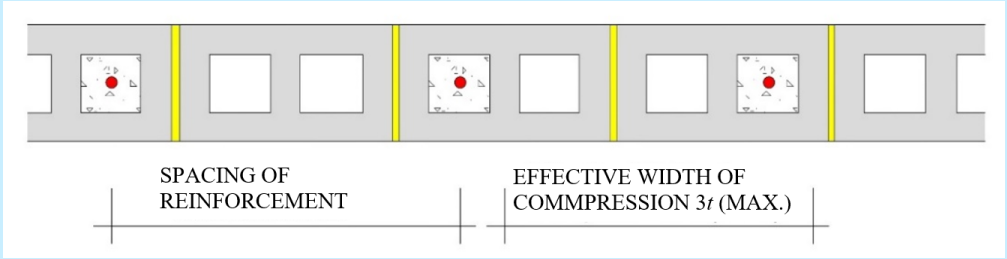

FIG. 71 EFFECTIVE COMPRESSIVE WIDTH

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10.4.1.4 *Permissible shear stress*

For members in flexure, the permissible shear stress shall be taken as:

$$f_v = \begin{cases} 0.083\sqrt{f_m} \le 0.25 \text{ MPa} & \text{Walls without web reinforcement} \\ 0.250\sqrt{f_m} \le 0.75 \text{ MPa} & \text{Walls with web reinforcement} \end{cases}$$

For walls in compression and bending, the Permissible Shear Stress for reinforced masonry walls shall be according to Table 30.

If there is tension in any part of a section of masonry, the area under tension shall be ignored while estimating the shear stress on the section.

Table 30 Permissible Shear Stress in Reinforced Masonry Walls under Compression and Bending

(*Clause* 10.4.1.4)

Sl No.	Type of Wall	M/Vd	Permissible Shear Stress F_v (Mpa)	Maximum Permissible Shear Stress $F_{v,max}$ (Mpa)
(1)	(2)	(3)	(4)	(5)
i)	Without web reinforcement	< 1.0	$\frac{1}{36}\left(4 - \frac{M}{Vd}\right)\sqrt{f_m}$	$\left(0.6 - 0.2\frac{M}{Vd}\right)$
		> 1.0	$0.083\sqrt{f_m}$	0.2
ii)	With web reinforcement	< 1.0	$\frac{1}{24}\left(4 - \frac{M}{Vd}\right)\sqrt{f_m}$	$\left(0.6 - 0.2\frac{M}{Vd}\right)$
		> 1.0	$0.125\sqrt{f_m}$	0.4

10.5 Earthquake Design Requirements

10.5.1 The requirements of this section shall apply to the design and construction of reinforced masonry to improve its performance when subjected to earthquake loads. These provisions are in addition to the general requirements of **5** of Part C/Section 1 of this NBCS. *See* Annex F.

10.5.2 *Different Performance Levels of Masonry Shear Walls*

Masonry buildings rely on masonry shear walls for the lateral load resistance and can be detailed for the following three levels of earthquake performance, which can be appropriately chosen for a building considering its importance, location, and acceptable degree of damage. Table 31 summarizes the requirement for these structural walls and *R* values recommended for use in **5** of Part C/Section 1 of this NBCS.

Table 31 Reinforcement and 'R' Values for Reinforced Masonry Buildings with different Wall Types and Earthquake Zones

(*Clause* 10.5.2)

Sl No.	Type of Wall	Description	Reinforcement	Earthquake Zone	R
(1)	(2)	(3)	(4)	(5)	(6)
i)	RMB 1	Reinforced masonry walls with minimum reinforcement	As per **10.5.2.1**	II and III	2.5

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Table 31 (*Concluded*)

Sl No.	Type of Wall	Description	Reinforcement	Earthquake Zone	R
(1)	(2)	(3)	(4)	(5)	(6)
ii)	RMB 2	Reinforced masonry walls with design reinforcement	As per 10.5.2.2	All	3.0
iii)	RMB 3	Reinforced masonry walls with special design reinforcement	As per 10.5.2.2 and 10.5.2.3	All	3.0

10.5.2.1 *Masonry buildings with walls having prescriptive reinforcement* (*RMB* 1)

The design of masonry walls shall comply with requirements of unreinforced masonry wall 4 and those given in **10.5.2.1(b)** and **10.5.2.1(c)** (*see* Fig. 72).

The requirements of this section apply to the design and construction of reinforced masonry to improve its performance when subjected to earthquake loads. These provisions are in addition to the general requirements of 4.

a) Elastic force reduction factor R shall be taken as 3 for *Reinforced Masonry Walls*.

b) *Minimum reinforcement requirements* (Fig. 72) — The vertical reinforcement of at least 100 mm^2 in cross-sectional area shall be provided at a maximum spacing of 3 m on centre at the following critical sections:

   1) wall corners;

   2) within 400 mm of each vertical face on either side of the openings extending from plinth to roof or floor level; and

   3) within 200 mm of the vertical end of the walls.

c) *Minimum horizontal reinforcement requirement* — The horizontal reinforcement of at least 100 mm^2 in cross-sectional area shall be provided along the full height of the wall at a maximum centre-to-centre vertical spacing of 400 mm and continuously along the full perimeter of the building, and at the following critical sections:

   1) in bond beam (below window openings and above plinth level in the first storey or floor slab level in the second and higher storeys) and anchoring into the vertical reinforcement provided around the openings;

   2) within 400 mm of each horizontal face above and below of the openings, extending at least 500 mm or $40d_b$ from the vertical face of the openings, where d_b is diameter of the horizontal bar being used; and

   3) within 400 mm of the top of the walls at structurally connected roof and floor levels.

10.5.2.2 *Masonry buildings with walls having design reinforcement* (*RMB* 2)

The masonry walls with design reinforcement shall be designed with the following requirements:

a) requirements specified in 10.1 to 10.4;

b) elastic force reduction factor (R) shall be taken as 3; and

c) requirements specified in 10.5.2.1(b) and 10.5.2.1(c).

10.5.2.3 *Masonry buildings with walls having special design reinforcement* (*RMB* 3)

Design of special reinforced masonry wall shall comply with the requirement of reinforced masonry as outlined in 10.5.2.2 and the following:

a) The masonry shall be uniformly reinforced in both horizontal and vertical direction such that the sum of reinforcement area in both directions shall be at least 0.2 percent of the gross cross-sectional area of the wall and minimum reinforcement area in each direction shall be not less than 0.07 percent of the gross cross-sectional area of the wall;

b) Maximum spacing of horizontal and vertical reinforcement shall be smaller of:

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1) one-third length of the wall;

2) one-third height of the wall; and

3) 1.2 m.

c) Minimum cross-sectional area of reinforcement in vertical direction shall be one-third of the required shear reinforcement placed in horizontal direction; and

d) Shear reinforcement placed in horizontal direction shall be anchored around vertical reinforcing bars with a 135° standard hook.

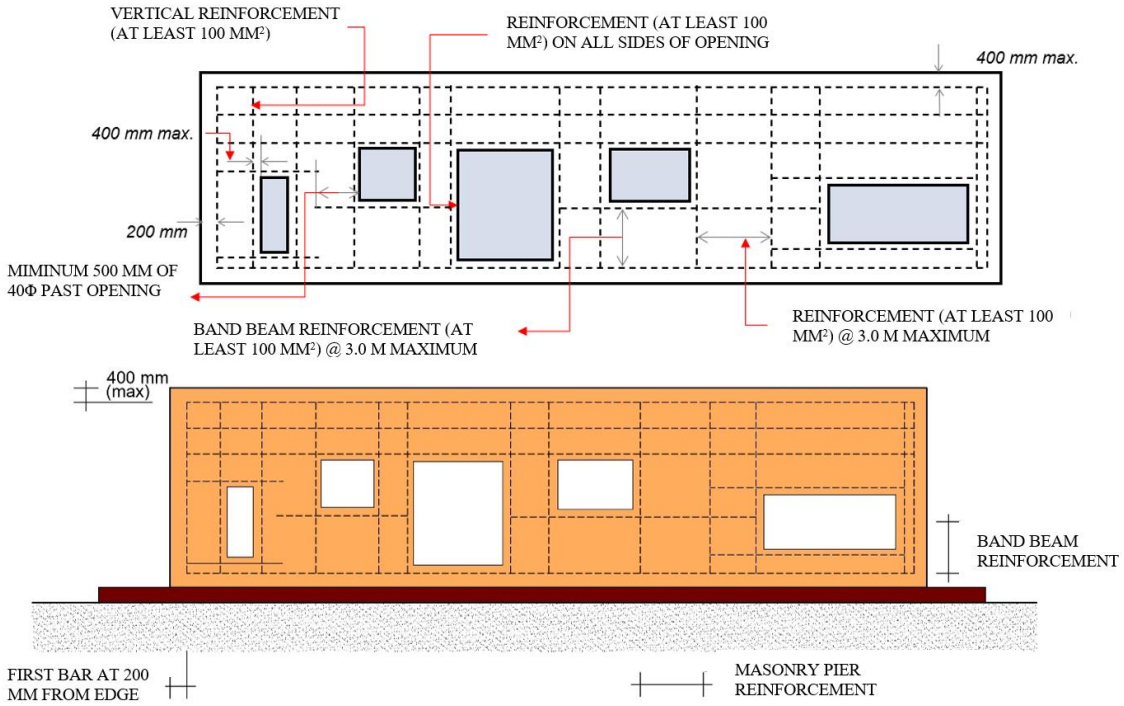

FIG. 72 REQUIREMENTS OF MINIMUM REINFORCEMENT IN MASONRY WALLS (RMB 1)

11 GUIDELINES FOR IMPROVING EARTHQUAKE RESISTANCE OF LOW STRENGTH MASONRY BUILDINGS

The term 'low strength masonry' includes fired brickwork laid in clay mud mortar and random rubble; uncoursed, undressed or semi-dressed stone masonry in weak mortars; such as cement sand, lime sand and clay mud. Special features of design and construction for improving earthquake resistance of buildings of low strength masonry are given in 11.1 to 11.6.5.7.

11.1 For the purpose of this clause, the terminology given in 8.1 shall apply.

11.2 General Principles

The general principles given in 9.2.1 to 9.2.5 shall be observed in construction of earthquake resistant buildings.

11.2.1 *Lightness*

Since the earthquake force is a function of mass, the weight of the building shall be as minimum as possible, consistent with structural safety and functional requirements. Roofs and upper storeys of buildings, in particular, should be designed as light as possible.

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11.2.2 *Continuity of Construction*

11.2.2.1 As far as possible, the parts of the building should be tied together in such a manner that the building acts as one unit.

11.2.2.2 For integral action of building, roof and floor slabs should be continuous throughout as far as possible.

11.2.2.3 Additions and alterations to the structures should be accompanied by the provision of positive measures to establish continuity between the existing and the new construction.

11.2.3 *Projecting and Suspended Parts*

11.2.3.1 Projecting parts should be avoided as far as possible. If the projecting parts cannot be avoided, they should be properly reinforced and firmly tied to the main structure, and their design should be in accordance with good practice [C-4(7)].

NOTE — In cases where stability of projecting parts against overturning is achieved by counterweight in the form of wall, slab, etc, the overturning should be checked by increasing the weight of projecting part and decreasing the weight of stabilizing mass simultaneously in accordance with the vertical seismic coefficient specified in **4.4.2** of good practice [C-4(7)].

11.2.3.2 Ceiling plaster shall preferably be avoided. When it is unavoidable, the plaster shall be as thin as possible.

11.2.3.3 Suspended ceiling shall be avoided as far as possible. Where provided they shall be light, adequately framed and secured.

11.2.4 *Building Configuration*

11.2.4.1 In order to minimise torsion, the building should have a simple rectangular plan and be symmetrical both with respect to mass and rigidity so that the centre of mass and rigidity of the building coincide with each other. It will be desirable to use separate blocks of rectangular shape particularly in seismic zones IV and above.

NOTE — For small buildings, minor asymmetry in plan and elevation may be ignored. Designing such buildings against torsion may be difficult and uncertain.

11.2.5 *Fire Safety*

Fire frequently follows an earthquake and therefore, buildings shall be constructed to make them fire resistant in accordance with the provisions of Part F ‘Fire and Life Safety’ of this NBCS, as relevant.

11.3 Special Construction Features

11.3.1 *Foundations*

11.3.1.1 For the design of foundations, the provisions of Part C/Section 1 and Section 2 of this NBCS shall generally be followed.

11.3.1.2 The subgrade below the entire area of the building shall preferably be of the same type of the soil. Wherever this is not possible, the buildings should preferably be separated into units and then the units should be located separately.

11.3.1.3 Loose fine sand, soft silt and expansive clays should be avoided. If unavoidable, the following measures may be taken to improve the soil on which the foundation of the building may rest:

a) Sand piling/under-reamed piling/stone columns, etc; and

b) Soil stabilization.

11.3.2 *Roofs and Floors*

11.3.2.1 Flat roof or floor should not preferably be made of tiles or ordinary bricks supported on steel, timber or reinforced concrete joists, nor they shall be of a type which in the event of an earthquake is likely to be loosened and parts or all of which may fall. If this type of construction cannot be avoided, the joists should be blocked at ends and bridged at intervals such that their spacing is not altered during an earthquake.

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11.3.2.1.1 For pitched roofs, corrugated iron or asbestos sheets should be used in preference to country, Allahabad or Mangalore tiles or other loose roofing units. All roofing materials shall be properly tied to the supporting members. Heavy roofing materials shall generally be avoided.

11.3.2.2 *Pent roofs*

11.3.2.2.1 All roof trusses should be supported on and fixed to timber band, reinforced concrete, or reinforced brick band. The holding down bolts should have adequate size and length as required for earthquake and wind forces.

Where a trussed roof adjoins a masonry gable, the ends of the purlins should be carried on and secured to a plate or bearer which shall be adequately bolted to reinforced concrete or reinforced brick band at the top of gable end masonry.

11.3.2.2.2 At tie level all the trusses and the gable end should be provided with diagonal braces in plan so as to transmit the lateral shear due to earthquake force to the gable walls acting as shear walls.

11.3.2.3 *Jack arches*

Jack arched roofs or floors, where used should be provided with mild steel ties in all spans along with diagonal braces in plan to ensure diaphragm actions.

11.3.3 *Staircases*

11.3.3.1 The inter-connection of the stairs with the adjacent floors should be appropriately treated by providing sliding joints at the stairs to eliminate their bracing effect on the floors. Ladders may be made fixed at one end and freely resting at the other.

11.3.3.2 *Built-in staircase*

When stairs are built monolithically with floors, they can be protected against damage by providing rigid walls at the stair opening. The walls enclosing the staircase, should extend through the entire height of the stairs and to the building foundations.

11.4 Box Type Construction

This type of construction consists of prefabricated or *in-situ* masonry walls along both the axes of the building. The walls support vertical loads and also act as shear walls for horizontal loads acting in any direction. All traditional masonry construction falls under this category. In prefabricated construction attention shall be paid to the connection between wall panels so that transfer of shear between them is ensured.

11.5 Categories of Buildings

For the purpose of specifying the earthquake resisting features in masonry and wooden buildings, the buildings have been categorized in four categories B to E based on the seismic zone and the importance of building I, where I is importance factor applicable to the building, the classification as given in **8.5** shall be applicable. The building categories are given in Table 14.

11.6 Low Strength Masonry Construction

11.6.1 Two types of construction are included herein, namely:

a) Brick construction using weak mortar; and

b) Random rubble and half-dressed stone masonry construction using different mortars such as clay mud, lime-sand and cement sand.

11.6.1.1 These constructions should not be permitted for important buildings and should preferably be avoided for building category D and shall not be used for category E (*see* Table 14).

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11.6.1.2 It will be useful to provide damp-proof course at plinth level to stop the rise of pore water into the superstructure.

11.6.1.3 Precautions should be taken to keep the rain water away from soaking into the wall so that the mortar is not softened due to wetness. An effective way is to take out roof projections beyond the walls by about 500 mm.

11.6.1.4 Use of a water-proof plaster on outside face of walls will enhance the life of the building and maintain its strength at the time of earthquake as well.

11.6.1.5 Ignoring tensile strength, free standing walls should be checked against overturning under the action of design seismic coefficient, α_h, allowing for a factor of safety of 1.5.

11.6.2 *Brickwork in Weak Mortars*

11.6.2.1 The fired bricks should have a compressive strength not less than 3.5 MPa. Strength of bricks and wall thickness should be selected for the total building height.

11.6.2.2 The mortar should be lime-sand (1 : 3) or clay mud of good quality. Where horizontal steel is used between courses, cement-sand mortar (1 : 3) should be used with thickness so as to cover the steel with 6 mm mortar above and below it. Where vertical steel is used, the surrounding brickwork of 1 × 1 or 1 ½ × 1 ½ brick size depending on wall thickness should preferably be built using 1 : 6 cement-sand mortar.

11.6.2.3 The minimum wall thickness shall be one brick in one storey construction, and one brick in top storey and 1 ½ brick in bottom storeys of up to three storey construction. It should also not be less than 1/16 of the length of wall between two consecutive perpendicular walls.

11.6.2.4 The height of the building shall be restricted to the following, where each storey height shall not exceed 3.0 m:

a) For Categories B and C: three storeys with flat roof; and two storeys plus attic for pitched roof.

b) For Categories D: two storeys with flat roof; and one storey plus attic for pitched roof.

11.6.2.5 *Special bond in brick walls*

For achieving full strength of masonry, the usual bonds specified for masonry should be followed so that the vertical joints are broken properly from course to course. To obtain full bond between perpendicular walls, it is necessary to make a sloping (stepped) joint by making the corners first to a height of 600 mm and then building the wall in between them. Otherwise the toothed joint should be made in both the walls, alternatively in lifts of about 450 mm (*see* Fig. 20).

11.6.3 *Stone Masonry* (*Random Rubble or Half-Dressed*)

11.6.3.1 The construction of stone masonry of random rubble or dressed stone type should generally follow good practice [C-4(11)].

11.6.3.2 The mortar should be cement-sand (1 : 6), lime sand (1 : 3) or clay mud of good quality.

11.6.3.3 The wall thickness '*t*' should not be larger than 450 mm. Preferably it should be about 350 mm, and the stones on the inner and outer wythes should be interlocked with each other.

NOTE — If the two wythes are not interlocked, they tend to delaminate during ground shaking, bulge apart (*see* Fig. 73) and buckle separately under vertical load leading to complete collapse of the wall and the building.

11.6.3.4 The masonry should preferably be brought to courses at not more than 600 mm lift.

11.6.3.5 'Through' stones of full length equal to wall thickness should be used in every 600 mm lift at not more than 1.2 m apart horizontally. If full length stones are not available, stones in pairs each of about ¾ of the wall thickness may be used in place of one full length stone so as to provide an overlap between them (*see* Fig. 74).

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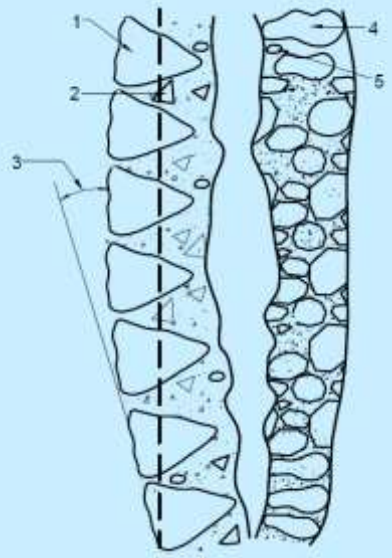
11.6.3.6 In place of 'through' stones, 'bonding elements' of steel bars 8 mm to 10 mm diameter bent to S-shape or as hooked links may be used with a cover of 25 mm from each face of the wall (*see* Fig. 74). Alternatively, wood bars of 38 mm × 38 mm cross section or concrete bars of 50 mm × 50 mm section with an 8 mm diameter rod placed centrally may be used in place of 'through' stones. The wood should be well treated with preservative so that it is durable against weathering and insect action.

11.6.3.7 Use of 'bonding' elements of adequate length should also be made at corners and junctions of walls to break the vertical joints and provide bonding between perpendicular walls.



1. HALF-DRESSED CONICAL STONE,
2. SMALL ALIGNMENT,
3. ROTATION OF WYTHE,
4. RANDOM RUBBLE,
5. MUD OR WEAK LIME MORTAR.

FIG. 73 WALL DELAMINATED WITH BUCKLED WYTHES

11.6.3.7 Use of 'bonding' elements of adequate length should also be made at corners and junctions of walls to break the vertical joints and provide bonding between perpendicular walls.

11.6.3.8 Height of the stone masonry walls (random rubble or half-dressed) should be restricted as follows, with storey height to be kept 3.0 m maximum, and span of walls between cross walls to be limited to 5.0 m:

a) *For category B* — Two storeys with flat roof or one storey plus attic, if walls are built in lime-sand or mud mortar; and one storey higher if walls are built in cement-sand 1 : 6 mortar; and

b) *For categories C and D* — Two storeys with flat roof or two storeys plus attic for pitched roof, if walls are built in 1 : 6 cement mortar; and one storey with flat roof or one storey plus attic, if walls are built in lime-sand or mud mortar, respectively.

11.6.3.9 If walls longer than 5 m are needed, buttresses may be used at intermediate points not farther apart than 4.0 m. The size of the buttress be kept of uniform thickness. Top width should be equal to the thickness of main wall, *t*, and the base width equal to one sixth of wall height.

11.6.4 *Opening in Bearing Walls*

11.6.4.1 Door and window openings in walls reduce their lateral load resistance and hence should preferably, be small and more centrally located. The size and position of openings shall be as given in Table 32 and Fig. 21.

11.6.4.2 Openings in any storey shall preferably have their top at the same level so that a continuous band could be provided over them including the lintels throughout the building.

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11.6.4.3 Where openings do not comply with the guidelines of Table 32, they should be strengthened by providing reinforced concrete lining as shown in Fig. 22 with 2 high strength deformed steel bars of 8 mm diameter.

11.6.4.4 The use of arches to span over the openings is a source of weakness and shall be avoided, otherwise, steel ties should be provided.

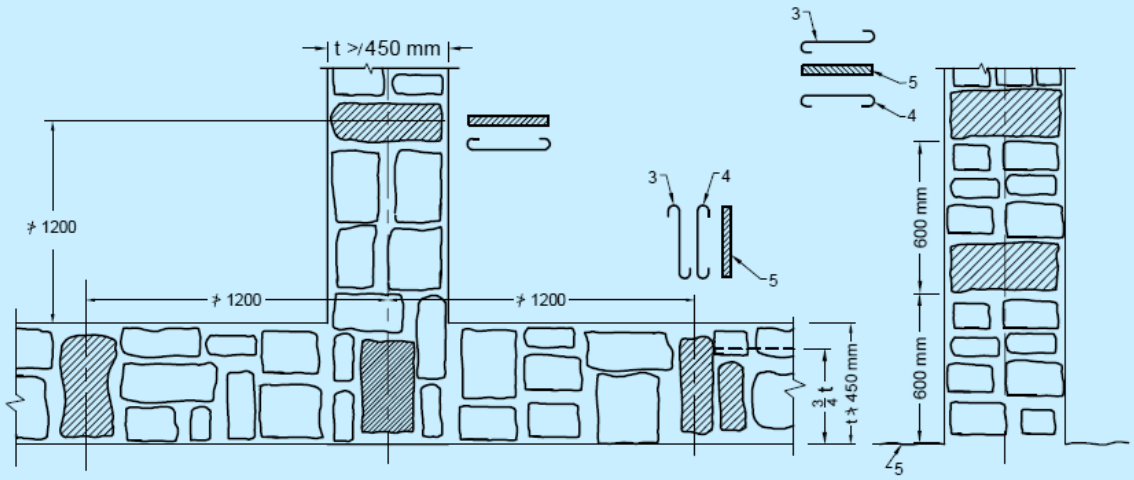

FIG.74(A) SECTIONAL PLAN OF WALL FIG. 74(B) CROSS-SECTION OF WALL

Keys:

1. Through Stone
2 Pair of Overlapping Stone
3. S-Shape Tie
4. Hooked Tie
5. Wood Plank
6. Floor Level.

All dimensions in millimetres.

FIG. 74 THROUGH STONE AND BAND ELEMENTS

11.6.5 *Seismic Strengthening Arrangements*

11.6.5.1 All buildings to be constructed of masonry shall be strengthened by the methods as specified for various categories of buildings, listed in Table 33 and detailed in subsequent clauses. Fig. 23 and Fig. 24 show, schematically, the overall strengthening arrangements to be adopted for category D buildings, which consist of horizontal bands of reinforcement at critical levels and vertical reinforcing bars at corners and junctions of walls.

11.6.5.2 Lintel band is a band provided at lintel level on all internal and external longitudinal as well as cross walls except partition walls. The details of the band are given in **11.6.5.5**.

11.6.5.3 Roof band is a band provided immediately below the roof or floors. The details of the band are given in **11.6.5.5**. Such a band need not be provided underneath reinforced concrete or reinforced brick slabs resting on bearing walls, provided that the slabs cover the width of end walls fully.

11.6.5.4 Gable band is a band provided at the top of gable masonry below the purlins. The details of the band are given in **11.6.5.5**. This band shall be made continuous with the roof band at the eaves level.

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11.6.5.5 *Details of band*

11.6.5.5.1 *Reinforced band*

The band should be made of reinforced concrete of grade not leaner than M15 or reinforced brickwork in cement mortar not leaner than 1 : 3. The bands should be of full width of the wall, not less than 75 mm in depth and should be reinforced with 2 high strength deformed steel bars of 8 mm diameter and held in position by 6 mm diameter bar links, installed at 150 mm apart as shown in Fig. 25.

NOTES

1 In coastal areas, the concrete grade shall be of grade in accordance with Part C/Section 5 of this NBCS and the filling mortar of 1 : 3 ratio (cement- sand) with water proofing admixture.

2 In case of reinforced brickwork, the thickness of joints containing steel bars should be increased to 20 mm so as to have a minimum mortar cover of 6 mm around the bar. In bands of reinforced brickwork, the area of steel provided should be equal to that specified above for reinforced concrete bands.

3 For full integrity of walls at corners and junctions of walls and effective horizontal bending resistance of bands, continuity of reinforcement is essential. The details as shown in Fig. 25 are recommended.

11.6.5.5.2 *Wooden band*

As an alternative to reinforced band, the lintel band could be provided using wood beams in one or two parallel pieces with cross elements as shown in Fig. 75.

11.6.5.6 Plinth band is a band provided at plinth level of walls on top of the foundation wall. This is to be provided where strip footings of masonry (other than reinforced concrete or reinforced masonry) are used and the soil is either soft or uneven in its properties as frequently happens in hill tracts. Where used, its section may be kept same as in **11.6.5.5.1**. This band serves as damp proof course as well.

11.6.5.7 *Vertical reinforcement*

Vertical steel at corners and junctions of walls which are up to 350 mm thick should be provided as specified in Table 34. For walls thicker than 350 mm, the area of the bars should be proportionately increased.

The vertical reinforcement should be properly embedded in the plinth masonry of foundations and roof slab or roof band so as to develop its tensile strength in bond. It should pass through the lintel bands and floor slabs or floor level bands in all storeys. Bars in different storeys may be welded or suitably lapped.

NOTES

1 Typical details of providing vertical steel in brickwork at corners and T-junctions are shown in Fig. 26.

2 For providing vertical bar in stone masonry, use of a casing pipe is recommended around which masonry be built to height of 600 mm (*see* Fig. 53). The pipe is kept loose by rotating it during masonry construction. It is then raised and the cavity below is filled with M15 (or 1 : 2 : 4) grade of concrete mix and rodded to compact it.

Table 32 Size and Position of Openings in Bearing Walls (*see* Fig. 21)

(*Clauses* 11.6.4.1, 11.6.4.3 *and Table* 34)

Sl No.	Description	Building Category	
		B and *C*	*D*
(1)	(2)	(3)	(4)
i)	Distance b_5 from the inside corner of outside wall, *Min,* mm	230	600

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Table 32 (*Concluded*)

Sl No.	Description	Building Category	
		B and *C*	*D*
(1)	(2)	(3)	(4)
ii)	Total length of openings, ratio; *Max* $(b_1 + b_2 + b_3)/l_1$ or $(b_6 + b_7)/l_2$ shall not exceed:		
	a) one storeyed building; and	0.46	0.42
	d) b) 2 and 3 storeyed building	0.37	0.33
iii)	Pier width between consecutive openings, b_4, mm	450	560
iv)	Vertical distance between two openings one above the other, h_3, *Min*, mm	600	600
v)	Width of opening of ventilator, b_8, *Max*, mm	750	750

Table 33 Strengthening Arrangements Recommended for Low Strength Masonry Buildings

(*Clause* 11.6.5.1)

Sl No.	Building Category	Number of Storeys	Strengthening to be Provided
(1)	(2)	(3)	(4)
i)	*B*	1 and 2	*b*,*c*,*f*,*g*
		3	*b*,*c*,*d*,*f*,*g*
ii)	*C*	1	*b*,*c*,*f*,*g*
		2 and 3	*b*,*c*,*d*,*f*,*g*
iii)	*D*	1 and 2	*b*,*c*,*d*,*f*,*g*

where

b = lintel band (*see* **11.6.5.2**);

c = roof band and gable band where necessary (*see* **11.6.5.3** and **11.6.5.4**);

d = vertical steel at corners and junctions of walls (*see* **11.6.5.7**);

f = bracing in plan at tie level of pitched roofs (*see* **11.3.2.2.2**); and

g = plinth band, where necessary (*see* **11.6.5.6**).

NOTE — For building of category *B* in two storeys constructed with stone masonry in weak mortar, it will be desirable to provide vertical steel of 10 mm dia in both storeys.

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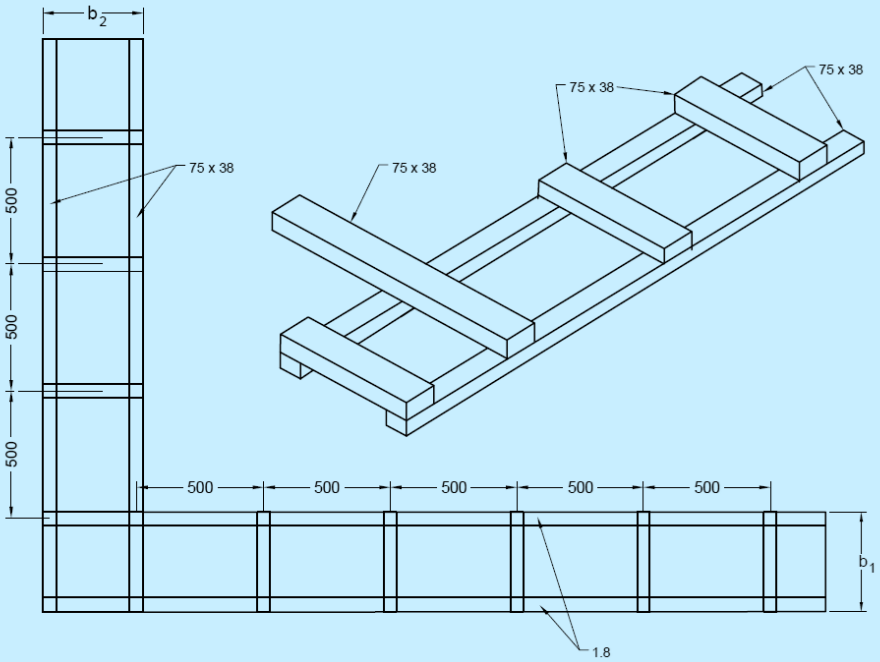

FIG. 75(A) PERSPECTIVE VIEW FIG. 75(B) PLAN OF BAND

All dimensions in millimetres.

FIG. 75 WOODEN BAND FOR LOW-STRENGTH MASONRY BUILDINGS

Table 34 Vertical Steel Reinforcement in Low Strength Masonry Walls

(*Clauses* 11.6.5.7)

Sl No.	No. of Storeys	Storey	Diameter of HSD Single Bar; in mm, at Each Critical Section for		
			Category *B*	Category *C*	Category *D*
(1)	(2)	(3)	(4)	(5)	(6)
i)	One	—	Nil	Nil	10
ii)	Two	a) Top	Nil	10	10
		b) Bottom	Nil	10	12
iii)	Three	a) Top	10	10	10
		b) Middle	10	10	12
		c) Bottom	12	12	12

NOTES

1 The diameters given above are for high strength deformed bars with yield strength 415 MPa. For mild-steel plain bars, use equivalent diameters.

2 The vertical bars should be covered with concrete of M15 grade or with mortar 1 : 3 (cement-sand) in suitably created pockets around the bars (*see* Fig. 26 and Fig. 76). This will ensure their safety from corrosion and good bond with masonry.

3 For category *B* two storey stone masonry buildings, *see* Note under Table 32.

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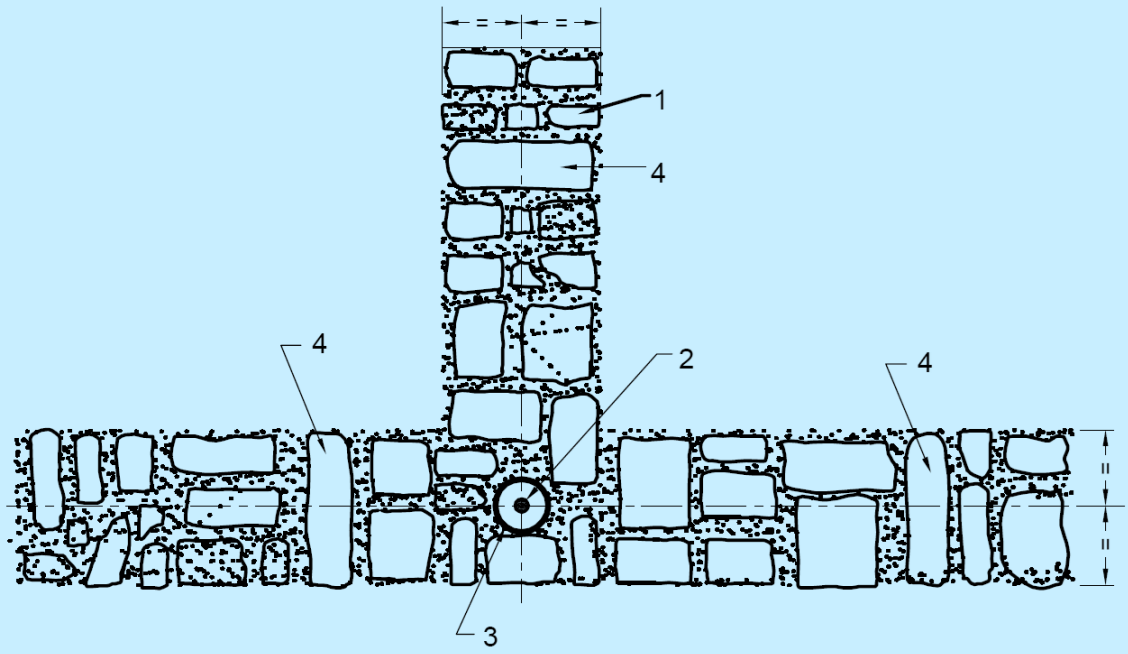

1. STONE WALL,
2. VERTICAL STEEL BAR,
3. CASING PIPE,
4. THROUGH STONE OR BONDING ELEMENT.

FIG. 76 TYPICAL CONSTRUCTION DETAIL FOR INSTALLING VERTICAL STEEL BAR IN RANDOM RUBBLE STONE MASONRY

12 MASONRY WALLS USING RAT-TRAP BOND TECHNOLOGY

This type of masonry shall be applicable for masonry structures of not more than 2 storeys constructed in seismic zones II and III. For buildings in other seismic zones, this type of masonry wall shall be used only in non-load bearing applications such as partition walls. The provisions for this type of masonry shall be as given in Annex G.

13 SUSTAINABILITY IN MASONRY CONSTRUCTION

The sustainability of masonry structures is addressed by quantifying embodied carbon in the structural system and the sources/types of materials used. For guidelines for selection of materials for masonry and assessing the embodied carbon in the masonry systems, the relevant provisions of the relevant guidelines on sustainability may be followed (Handbook on Approach to Sustainability).

14 NOTATIONS AND SYMBOLS

The various notations and letter symbols used in the text of the Section shall have the meaning as given in Annex H.

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ANNEX A

(*Clause* 4.7)

SOME GUIDELINES FOR ASSESSMENT OF ECCENTRICITY OF LOADING ON WALLS

A-1 Where a reinforced concrete roof and floor slab of normal span (not exceeding 30 times the thickness of wall) bear on external masonry walls, the point of application of the vertical loading shall be taken to be at the centre of the bearing on the wall. When the span is more than 30 times the thickness of wall, the point of application of the load shall be considered to be displaced from the centre of bearing towards the span of the floor to an extent of one-sixth the bearing width.

A-2 In case of a reinforced concrete slab of normal span (that is, less than 30 times the thickness of the wall), which does not bear on the full width of the wall and 'cover tiles or bricks' are provided on the external face, there is some eccentricity of load. The eccentricity may be assumed to be one-twelfth of the thickness of the wall.

A-3 Eccentricity of load from the roof/floor increases with the increase in flexibility and thus deflection of the slabs. Also, eccentricity of loading increases with the increase in fixity of slabs/beams at supports. Precast RCC slabs are better than *in-situ* slabs in this regard because of very little fixity. If supports are released before further construction on top, fixity is reduced.

A-4 Interior walls carrying continuous floors are assumed to be axially loaded except when carrying very flexible floor or roof systems. The assumption is valid also for interior walls carrying independent slabs spanning from both sides, provided the span of the floor on one side does not exceed that on the other by more than 15 percent. Where the difference is greater, the displacement of the point of application of each floor load shall be taken as one-sixth of its bearing width on the wall and the resultant eccentricity calculated therefrom.

A-5 For timber and other lightweight floors, even for full width bearing on wall, an eccentricity of about one-sixth may be assumed due to deflection. For timber floors with larger spans, that is, more than 30 times the thickness of the wall, eccentricity of one-third the thickness of the wall may be assumed.

A-6 In multi-storeyed buildings, fixity and eccentricity have normally purely local effect and are not cumulative. They just form a constant ripple on the downward increasing axial stress. If the ripple is large, it is likely to be more serious at upper levels where it can cause cracking of walls than lower down where it may or may not cause local over-stressing.

NOTE — The resultant eccentricity of the total loads on a wall at any level may be calculated on the assumption that immediately above a horizontal lateral support, the resultant eccentricity of all the vertical loads above that level is zero.

A-7 For a wall corbel to support some load, the point of application of the load shall be assumed to be at the centre of the bearing on the corbel.

ANNEX B

(*Clauses* 5.4.1 *and* 5.4.5)

CALCULATION OF BASIC COMPRESSIVE STRESS OF MASONRY BY PRISM TEST

B-1 DETERMINATION OF COMPRESSIVE STRENGTH OF MASONRY BY PRISM TEST

When compressive strength of masonry (f_m) is to be established by tests, it shall be done in advance of the construction, using prisms built of similar materials under the same conditions with the same bonding arrangement as for the structure. In building the prisms, moisture content of the units at the time of laying, the consistency of the mortar, the thickness of mortar joints and workmanship shall be the same as will be used in the structure. Assembled specimen shall be at least 400 mm high and shall have a height to thickness ratio (h/t) of at least 2 but not more than 5. If the h/t ratio of the prisms tested is less than 5 in case of brickwork and more than 2 in case of blockwork, compressive strength values indicated by the tests shall be corrected by multiplying with the factor indicated in Table 35.

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Table 35 Correction Factors for Different *h*/*t* Ratios
(*Clause* B-1)

Sl No.	Ratio of height to thickness (h/t)	Correction factors for brickwork[1)]	Correction factors for blockwork[1)]
(1)	(2)	(3)	(4)
i)	2.0	0.73	1.00
ii)	2.5	0.80	—
iii)	3.0	0.86	1.20
iv)	3.5	0.91	—
v)	4.0	0.95	1.30
vi)	5.0	1.00	1.37

[1)] Interpolation is valid for intermediate values.

Prisms shall be tested after 28 days between sheets of nominal 4 mm plywood, slightly longer than the bed area of the prism, in a testing machine, the upper platform of which is spherically seated. The load shall be evenly distributed over the whole top and bottom surfaces of the specimen and shall be applied at the rate of 350 kN/m to 700 kN/m. The load at failure should be recorded.

B-2 CALCULATION OF BASIC COMPRESSIVE STRESS

Basic compressive stress of masonry shall be taken to be equal to 0.25 f_m where f_m is the value of compressive strength of masonry as obtained from prism test.

ANNEX C

(*Clauses* 5.3.3 *and* 5.4.1.5)

GUIDELINES FOR DESIGN OF MASONRY SUBJECTED TO CONCENTRATED LOADS

C-1 EXTENT OF DISPERSAL OF CONCENTRATED LOAD

For concentric loading, maximum spread of a concentrated load on a wall may be taken to be equal to $b+4t$ (b is width of bearing and t is thickness of wall), or stretch of wall supporting the load, or centre-to-centre distance between loads, whichever is less.

C-2 INCREASE IN PERMISSIBLE STRESS

C-2.1 When a concentrated load bears on a central strip of wall, not wider than half the thickness of the wall and is concentric, bearing stress in masonry may exceed the permissible compressive stress by 50 percent, provided the area of supporting wall is not less than three times the bearing area.

C-2.2 If the load bears on full thickness of wall and is concentric, 25 percent increase in stress may be allowed.

C-2.3 For loading on central strip wider than half the thickness of the wall but less than full thickness, increase in stress may be worked out by interpolation between values of increase in stresses as given in **C-2.1** and **C-2.2**.

C-2.4 In case concentrated load is from a lintel over an opening, an increase of 50 percent in permissible stress may be taken, provided the supporting area is not less than 3 times the bearing area.

C-3 CRITERIA OF PROVIDING BED BLOCK

C-3.1 If a concentrated load bears on one end of a wall, there is a possibility of masonry in the upper region developing tension. In such a situation, the load should be supported on an RCC bed block (of M20 Grade) capable of taking tension.

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C-3.2 When any section of masonry wall is subjected to concentrated as well as uniformly distributed load and resultant stress, computed by making due allowance for increase in stress on account of concentrated load, exceeds the permissible stress in masonry, a concrete bed block (of M20 Grade) should be provided under the load in order to relieve stress in masonry. In concrete, angle of dispersion of concentrated load is taken to be 45° to the vertical.

C-3.3 In case of cantilevers and long span beams supported on masonry walls, indeterminate but very high edge stresses occur at the supports and in such cases it is necessary to relieve stress on masonry by providing concrete bed block of M20 Grade concrete. Similarly when a wall is subjected to a concentrated load from a beam which is not sensibly rigid (for example, a timber beam or an RS joist), a concrete bed block should be provided below the beam in order to avoid high edge stress in the wall because of excessive deflection of the beam.

ANNEX D

(*Clause* 5.5.5)

GUIDELINES FOR APPROXIMATE DESIGN OF NON-LOAD BEARING WALL

D-1 PANEL WALLS

A panel wall may be designed approximately as under, depending upon its support conditions and certain assumptions:

a) when there are narrow tall windows on either side of panel, the panel spans in the vertical direction. Such a panel may be designed for a bending moment of $PH/8$, where P is the total horizontal load on the panel and H is the height between the centres of supports. Panel wall is assumed to be simply supported in the vertical direction;

b) when there are long horizontal windows between top support and the panel, the top edge of the panel is free. In this case, the panel should be considered to be supported on sides and at the bottom, and the bending moment should depend upon height to length ratio of panel and flexural strength of masonry. Approximate values of bending moments in the horizontal direction for this support condition, when ratio (μ) of flexural strength of wall in the vertical direction to that in horizontal direction is assumed to be 0.5, are given in Table 36; and

c) When either there are no window openings or windows are of 'hole-in-wall' type, the panel is considered to be simply supported on all four edges. In this case also, amount of maximum bending moment depends on height to length ratio of panel and ratio (μ) of flexural strength of masonry in vertical direction to that in the horizontal direction. Approximate values for maximum bending moment in the horizontal direction for masonry with μ = 0.50, are given in Table 37.

Table 36 Bending Moments in Laterally Loaded Panel Walls, Free at Top Edge and Supported on Other Three Edges
[*Clause* D-1 (b)]

Sl No.	$\frac{\text{Height of Panel, } H}{\text{Length of Panel, } L}$	Bending Moment
(1)	(2)	(3)
i)	0.30	$P.L/25$
ii)	0.50	$P.L/18$
iii)	0.75	$P.L/14$
iv)	1.00	$P.L/12$
v)	1.25	$P.L/11$
vi)	1.50	$P.L/10.5$
vii)	1.75	$P.L/10$

NOTE — For H/L ratio less than 0.30, the panel should be designed as a free-standing wall and for H/L ratio exceeding 1.75, it should be designed as a horizontally spanning member for a bending moment value of $PL/8$.

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Table 37 Bending Moments in Laterally Loaded Panel Walls Supported on All Four Edges

[*Clause* D-1 (c)]

Sl No.	$\frac{\text{Height of Panel, } H}{\text{Length of Panel, } L}$	Bending Moment
(1)	(2)	(3)
viii)	0.30	*P.L*/72
ix)	0.50	*P.L*/36
x)	0.75	*P.L*/24
xi)	1.00	*P.L*/18
xii)	1.25	*P.L*/15
xiii)	1.50	*P.L*/13
xiv)	1.75	*P.L*/12

NOTE — When *H*/*L* is less than 0.30, value of bending moment in the horizontal direction may be taken as nil and panel wall may be designed for a bending moment value of *PH*/8 in the vertical direction; when *H*/*L* exceeds 1.75, panel may be assumed to be spanning in the horizontal direction and designed for bending moment of *PL*/8.

D-2 CURTAIN WALLS

Curtain walls may be designed as panel walls taking into consideration the actual supporting conditions.

D-3 PARTITION WALLS

D-3.1 These are internal walls usually subjected to much smaller lateral forces. Behaviour of such wall is similar to that of panel wall and these could, therefore, be designed on similar lines. However, in view of smaller lateral loads, ordinarily these could be apportioned empirically as follows:

a) Walls with adequate lateral restraint at both ends but not at the top :

   1) the panel may be of any height, provided the length does not exceed 40 times the thickness;

   2) the panel may be of any length, provided the height does not exceed 15 times the thickness (that is, it may be considered as a free-standing wall); or

   3) where the length of the panel is over 40 times and less than 60 times the thickness, the height plus twice the length may not exceed 135 times the thickness;

b) Walls with adequate lateral restraint at both ends at the top:

   1) the panel may be of any height, provided the length does not exceed 40 times the thickness;

   2) the panel may be of any length, provided the height does not exceed 30 times the thickness; or

   3) where the length of the panel is over 40 times and less than 110 times the thickness, the length plus three times the height should not exceed 200 times the thickness; and

c) When walls have adequate lateral resistant at the top but not at the ends, the panel may be of any length, provided the height does not exceed 30 times the thickness.

D-3.2 Strength of bricks used in partition walls should not be less than 3.5 N/mm^2 or the strength of masonry units used in adjoining masonry, whichever is less. Grade of mortar should not be leaner than M2.

D-4 CANTILEVER PROJECTIONS

D-4.1 The provisions given under **5** of Part C/Section 1 of this NBCS shall be applicable. As a general guide, the details of connection between parapets and slabs for existing structures shall be as given in Fig. 77.

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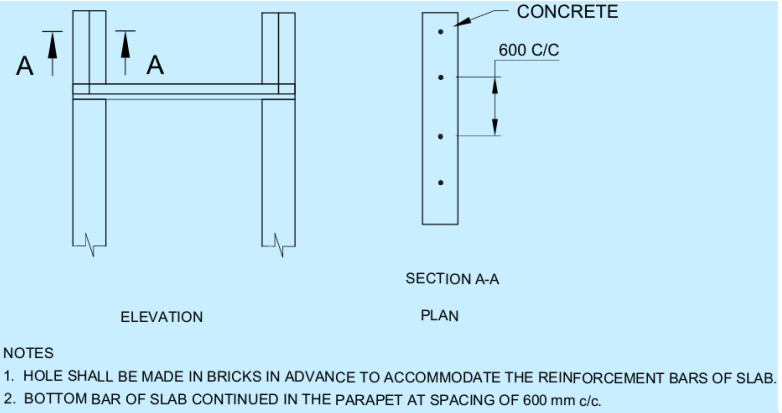

FIG. 77 PARAPET WALLS

ANNEX E

(*Clause* 10.4.1.2)

TYPICAL ITERATIVE METHOD FOR WALLS SUBJECTED TO FLEXURE AND AXIAL LOAD

E-1 There is no closed form solution available for the computation of stress in walls due to combined action of bending and axial compression. Engineers normally prefer to use Design charts for computation of stresses in such conditions, but design charts for most masonry systems are not readily available. Therefore, iterative methods are used as most common tools for masonry structures. Two types of iterative procedures are given below, which can be used to analyse the adequacy of the section under specified load and determine the amount of reinforcement required, wherever necessary.

E-2 ITERATIVE PROCEDURE 1

Fig. 78 shows a diagram of a wall subjected to an axial load and bending moment. The same diagram is applicable to both in-plane and out-of-plane bending of walls. There are three possible conditions for the wall:

a) wall is uncracked;

b) wall is cracked with the steel in compression (the extent of the crack has not reached the steel); or

c) wall is cracked with the steel in tension.

Fig. 78 shows the interaction between bending moment and axial load. Loading conditions beyond the limits of the diagram are beyond the allowable stresses. The condition of the wall can be quickly diagnosed by using the non-dimensional parameter *M/Pd*. This parameter represents a straight line radiating from the diagram's origin as shown in Fig. 79. By summing moments, it can be shown that certain values of *M/Pd* divide the diagram into three regions. These values are given in Table 38. The allowable moments for regions 1 and 2 can be obtained in closed form with simple equations. But in region 3, it is complicated a bit. The process consists of initial assumption of thickness, length and other necessary parameters and determination of the amount of steel required.

For region 3, the process starts with another assumption that the tension in steel controls. By making an initial guess about the location of the neutral axis, the expressions for M_p and a_2 provide an iterative process that quickly converges. The equations are derived from the summation of forces and moments and by using linear stress-strain relationships. The equations should always converge for all values of *P,* l_w*, M,* and *n*. If convergence is not obtained, it is likely that the assumed initial value of *a* has exceeded *d*. The initial value of *a* is to be reduced and same procedure is to be repeated. The iteration will often result in a negative steel area. This signifies that the element does not need reinforcement to resist the loads within the allowable stresses, even though the section is cracked (region 3).

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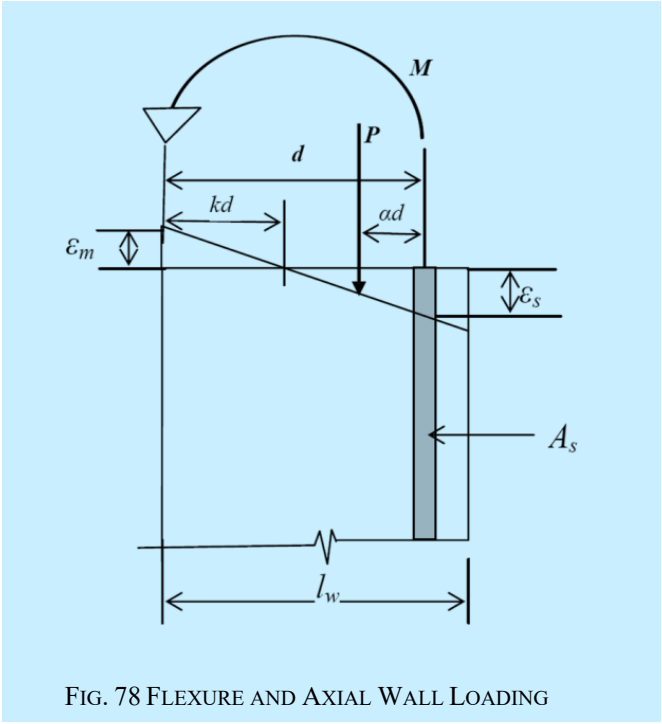

FIG. 78 FLEXURE AND AXIAL WALL LOADING

Following convergence, a check is made of the initial assumption that tension in steel controls. If not valid, then the compression controls and a new set of equations that do not require iteration are used.

E-3 The following is a detailed step-by-step procedure of the analysis methodology described above:

a) Step 1 — Determine the wall condition:

1) calculate α as the distance from the axial load to the centroid of the tension steel divided by *d*;
2) calculate the quantity M/*Pd*; and
3) use Table 38 to determine the region for analysis. (*see* Fig. 78).

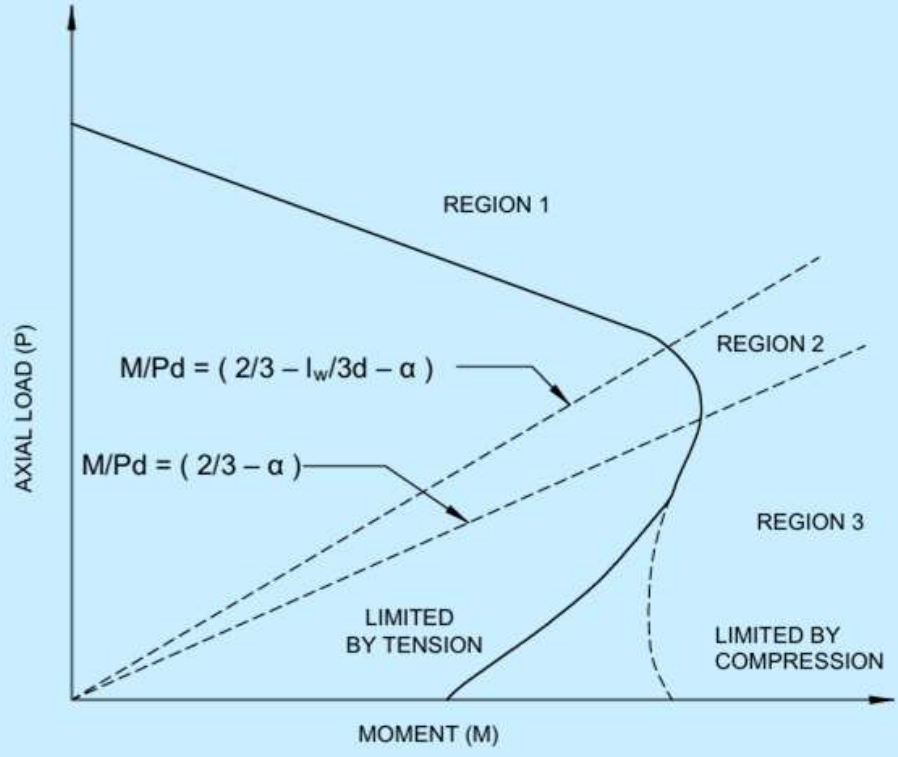

FIG. 79 INTERACTION DIAGRAM

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Table 38 Flexure and Axial Loading — Wall Analysis

(*Clauses* E-2 *and* E-3)

Sl No. (1)	Region (2)	Condition of Wall (3)	Test (4)
i)	1	Wall is in compression and not cracked	$\frac{M}{Pd} \le \left[1 - \frac{l_w}{3d} - \alpha\right]$
ii)	2	Wall is cracked but steel is in compression	$\frac{M}{Pd} \le \left[\frac{2}{3} - \alpha\right]$
iii)	3	Wall is cracked and steel is in tension	$\frac{M}{Pd} > \left[\frac{2}{3} - \alpha\right]$

b) Step 2 — Calculate the allowable moment:

1) Region 1 — The moment is limited by flexural compression in the masonry;

$$M_m = \frac{bl_w^2}{6} F_b - P\frac{l_w}{6}$$

If M_m is greater than M applied, the section is satisfactory.

2) Region 2 — The moment is also limited by flexural compression in the masonry; and

$$M_m = P(1-\alpha)d - \frac{2}{3}\left[\frac{P^2}{F_b b}\right]$$

If M_m is greater than M applied, the section is satisfactory.

3) Region 3 — The moment may be limited by either the compression in the masonry or tension in the steel. An iterative approach as below shall be adopted.

i) Assume a compression centroid location, a; and

ii) Perform the following iteration that assumes the tension in the steel controls (remember that A_s may be negative).

$$M_p = P\left(\frac{l_w}{2} - \alpha\right)$$

$$A_s = \frac{M - M_p}{F_s(d-a)}$$

$$\zeta = \frac{(P + A_s F_s)n)}{F_b b}$$

$$a_2 = \frac{\sqrt{\zeta^2 + 2\zeta d} - \zeta}{3}$$

where

a = estimate of internal compression load centroid distance from the extreme compression fibre;

A_s = area of trim or edge steel;

b = width of the wall for out-of-plane bending (thickness of wall for in-plane bending);

d = distance from the extreme compression fibre to the steel centroid;

F_s = allowable steel tension stress;

I_w = width of the wall for in-plane bending (thickness of wall for out of plane bending);

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M = applied moment;

M_p = moment of applied axial load with respect to the centroid of internal compression force; and

P = applied axial load at the centre of the wall.

Use a_2 for a, and repeat until the value of a converges.

c) Step 3 — If the iteration converges and the resulting a is less than the following value, the wall is limited by the tension reinforcement and the analysis is complete. Otherwise continue to step 4:

$$a = \frac{d}{3\left(1+\frac{F_s/F_b}{n}\right)}$$

d) Step 4 — If the value of a is larger than the above value, determine the required steel area using the following:

$$a = \frac{d}{2} - \sqrt{\left(\frac{d^2}{4} - \frac{2(P\alpha d + M)}{3F_b b}\right)}$$

If $\sqrt{\left(\frac{d^2}{4} - \frac{2(P\alpha d + M)}{3F_b b}\right)}$ is negative, there is inadequate compression capacity.

Increase b or F_b or both. The steel area is:

$$A_s = \frac{\left(\frac{3F_b ab}{2} - P\right)}{\left[nF_b\left(\frac{d}{3a} - 1\right)\right]}$$

In all cases the wall trim steel should not be less than minimum values.

E-4 ITERATIVE PROCEDURE

The following is another procedure to determine the state of stress in wall elements subjected to combined action of bending and axial compression. Free body diagram of a wall with axial load, bending moment, and tensile and compressive forces is given in Fig. 80. The location of C is based on the neutral axis location kd. The location of T is at the centroid of the reinforcement.

The problem can be solved by taking moments about the centerline of the wall, the tensile reinforcement (assuming that masonry compressive stresses control), or the compressive force (assuming that the stresses in the reinforcement control). By inserting the allowable value of stresses, which is assumed to control, a quadratic equation in terms of kd may be developed and solved.

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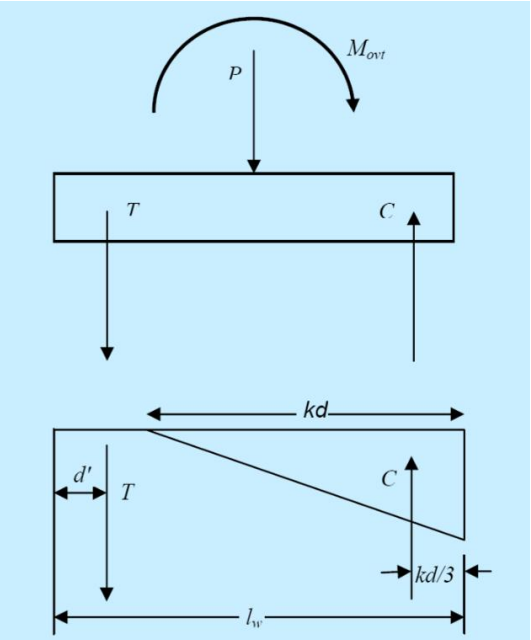

FIG. 80 FLEXURE BODY DIAGRAM OF A WALL

For example if P is at the centerline of the wall, taking moments about the centroid of the tensile reinforcement results in the equation:

$$C\left[\left(l_w - d'\right) - \left(\frac{kd}{3}\right)\right] = P\left(\frac{l_w}{2} - d'\right) + M$$

but $C = ½\ F_m bkd$ and $l_w - d' = d$. Then:

$$½ F_m bkd\left(d - \frac{kd}{3}\right) = P\left(\frac{l_w}{2} - d'\right) + M$$

Dividing both side by $½\ F_m b$ results in

$$kd^2 - \frac{(kd)^2}{3} = \frac{P\left(\frac{l_w}{2} - d'\right) + M}{½ F_m b}$$

Multiplying by –3 and rearranging terms:

$$(kd)^2 - 3d(kd) + \frac{3\left[P\left(\frac{l_w}{2} - d'\right) + M\right]}{½ F_m b} = 0$$

$$kd = \frac{3d - \sqrt{(-3d)^2 - 4\left(\frac{3\left[P\left(\frac{l_w}{2} - d'\right) + M\right]}{½ F_m b}\right)}}{2}$$

Inserting the values for P, M, l_w, b, d and F_m yields kd. Note that this method is iterative since a location of d' shall be assumed.

Having solved for kd one can calculate C.

By $\Sigma F_v = 0$, $T = C - P$

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From strain compatibility:

$$f_s = \left(\frac{1-k}{k}\right) nF_m$$, Thus f_s can be calculated.

If $f_s < F_s$, the masonry compressive stress controls and the solution is finished by calculating $A_s = T/f_s$. If however, $f_s > F_s$ the assumption that masonry stresses control is incorrect, and the neutral axis location is wrong.

A good starting assumption to calculate f_m is to take T assuming that masonry compressive stresses control, and to divide by F_s to get a trial A_s. This allows the designer to choose a bar layout and verify the initial assumption of d', the distance to the centroid of reinforcement.

$$A_s = T/F_s$$

Since $C = T + P$ the effects of the axial load can be included by using an effective amount of reinforcement.

$$(A_s)_{eff} = A_s + \frac{P}{F_s} \; (or \frac{T+P}{F_s})$$

$$\rho_{eff} = \frac{(A_s)_{eff}}{bd}$$

$$k = \left[(n\rho)^2 + 2n\rho\right]^{\frac{1}{2}} - n\rho$$

$$j = 1 - \frac{k}{3}$$

The steel stress may be checked using

$$f_s = \frac{M'}{(A_s)_{eff}\, jd}$$

where

$$M' = P\left(\frac{l_w}{2} - d'\right) + M$$

$$T_{eff} = (A_s)_{eff} \times f_s$$

$$T = T_{eff} - P$$

The masonry stress may be checked by using:

$$f_m = \frac{2M'}{bjkd^2}$$

If $f_s > F_s$, a second iteration may be required may be required with a larger bar size or an extra bar. In a shear wall if an extra bar is added, the value of d', and the moments calculated using:

$$P\left(\frac{l_w}{2} - d'\right) + M$$

will need to be re-evaluated. Then one can reiterate from:

$$(A_s)_{eff} = A_s + \frac{P}{F_s}$$

to reduce f_s so that it is less than F_s.

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ANNEX F

(*Clause* 10.5.1)

DISTRIBUTION OF LATERAL FORCES IN THE PIERS OF A MASONRY WALL

F-1 Masonry walls are often seen to be perforated to make arrangement for windows and doors. The distribution of lateral force in a masonry wall is dependent on the position of the openings and the relative rigidity of the masonry piers created due to the presence of the openings in the masonry wall. The relative rigidity is dependent on the height by length ratio (h/L ratio) of the piers and the end conditions of those masonry piers as the deflection of the masonry piers due to horizontal loading changes due to the end condition of the piers (*see* Fig. 81). Here is a simple process is described which can be used to distribute the lateral force in a wall which can be considered to be consist of some piers with some specific arrangements.

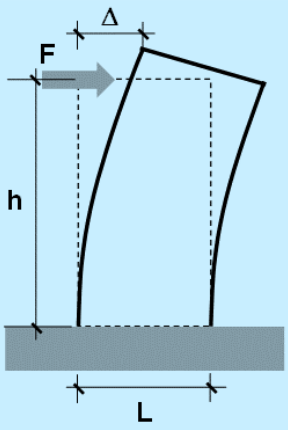


FIG. 81(A) CANTILEVERED WALL OR PIER (TOP FREE TO ROTATE)

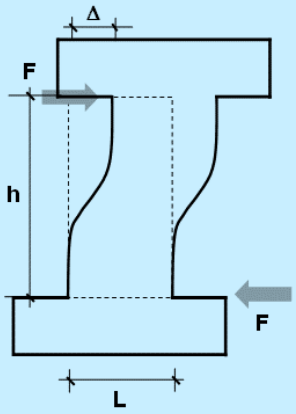


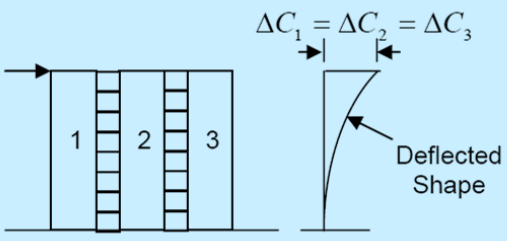
FIG. 81(B) PIER BETWEEN OPENINGS (TOP RESTRAINED FROM ROTATION)

FIG. 81 DEFLECTION DUE TO END CONDITIONS OF PIERS

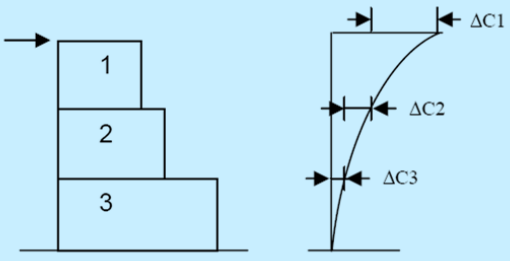
F-2 In any kind of placing of opening, the wall can be represented as a horizontal and vertical combination of piers with their respective end condition which will be used to find out their rigidities. Where large openings occur, it is difficult to obtain effective coupling of the wall segments or piers. If the wall is analysed as a horizontal combination of piers as shown in the Fig. 82 (a) and the combined rigidity

$$R = R_{c1} + R_{c2} + R_{c3}$$

where R_{c1} , R_{c2} , R_{c3} are the rigidities of the piers 1, 2, 3, respectively.



(a) HORIZONTAL COMBINATION OF WALL SEGMENTS



(b) VERTICAL COMBINATION OF WALL SEGMENTS

FIG. 82 WALL COMBINATIONS FOR CALCULATING RIGIDITIES OF WALLS WITH OPENINGS

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If the segments are combined vertically, as shown in Fig. 82 (b), the combined rigidity can be calculated as:

$$R = \frac{1}{\Delta C_1 + \Delta C_2 + \Delta C_3} = \frac{1}{\frac{1}{R_{c1}} + \frac{1}{R_{c2}} + \frac{1}{R_{c3}}}$$

Where R_{c1}, R_{c2} , R_{c3} are the rigidities of the piers 1, 2, 3 respectively. Combination of these two types can be used to find the effective relative stiffness of a masonry wall.

The expression ignores the rotations that occur at the tops of segment 2 and 3 and therefore overestimates the rigidity of the wall. Along with this, it is valid only for the application of loads at the top level of the building.

As already said the rigidity R of the pier is dependent on its dimensions, modulus of elasticity E, modulus of rigidity G, and the support conditions. For a cantilever pier the displacement due to combined action of bending and shear is

$$\Delta_c = \frac{Fh^3}{3EI} + \frac{1.2Fh}{GA}$$

where

A = area of the pier = Lt;

F = horizontal force applied to the pier;

G = modulus of Rigidity = $0.4E_m$;

h = height of the pier;

I = moment of inertia of pier = $tL^3/12$;

L = length of the pier; and

t = thickness of pier.

So, displacement of the cantilever pier is:

$$\Delta C = \frac{1}{Et}\left[4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)\right]$$

The rigidity of a wall is proportional to the inverse of the deflection. For cantilever walls the rigidity will be:

$$R_c = \frac{1}{\Delta C} = \frac{Et}{\left[4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)\right]}$$

$$\Rightarrow \frac{R_c}{Et} = \frac{1}{\left[4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)\right]}$$

For a pier with both ends fixed against rotation the deflection due to combined action of bending and shear will be:

$$\Delta f = \frac{Fh^3}{12EI} + \frac{1.2Fh}{GA} = \frac{F}{Et}\left[\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)\right]$$

$$\Rightarrow \frac{R_f}{E_t} = \frac{1}{\left[\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)\right]}$$

Values of R/E_t for different h/L ratio can be taken from the chart in the Fig. 83.

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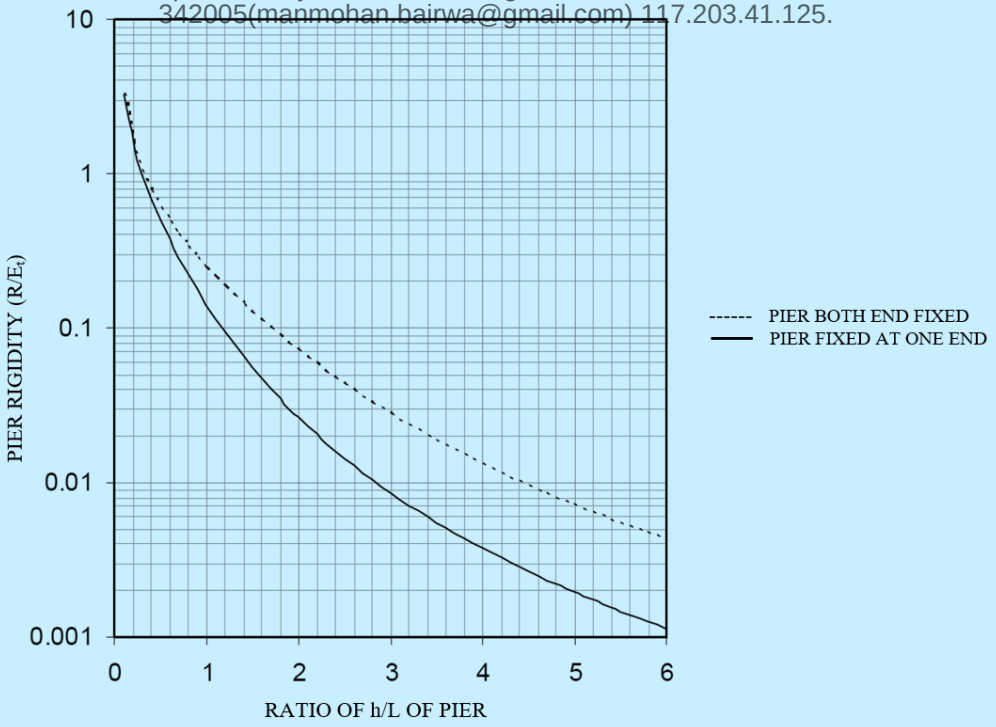

FIG. 83 CHARTS FOR CALCULATING WALL RIGIDITIES

From the relations given in equations above it is found that the relative contributions of the bending and shear deformation depend on the wall aspect ratio (*h/L*) and therefore, the rigidity varies over the height of the building. For high *h/L* ratios, the effect of shear deformation is very small and calculation of pier rigidities based on flexural stiffness is relatively accurate. For very squat walls (with *h/L< 0.25*), rigidities based on shear deformation are reasonably accurate, but for intermediate walls with *h/L* from 0.25 to 4, both components of relative rigidity should be considered.

F-3 The method is explained further explained by one illustration. For the wall as shown in Fig. 84.

$$R_{\text{wall}} = \frac{1}{\Delta_{1c} + \Delta_{2,3,4,5,6(f)}} = \frac{1}{\frac{1}{R_{1c}} + \frac{1}{R_{2,3,4,5,6(f)}}} \quad \text{(Vertical combination)}$$

where

$$R_{2,3,4,5,6(f)} = R_{2(f)} + R_{3,4,5,6(f)} \quad \text{(Horizonta l combination)}$$

$$\text{and } R_{3,4,5,6(f)} = \frac{1}{\Delta_{3,4,5(f)} + \Delta_{6(f)}} = \frac{1}{\frac{1}{R_{3,4,5(f)}} + \frac{1}{R_{6(f)}}} \quad \text{(Vertical combination)}$$

$$R_{3,4,5(f)} = R_{3(f)} + R_{3(f)} + R_{3(f)} \text{ (Horizontal combination)}$$

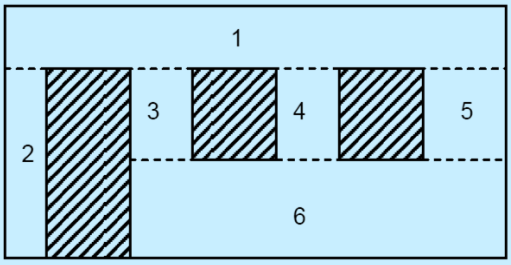

FIG. 84 WALL WITH OPENINGS (CALCULATION OF RIGIDITY AND LOAD DISTRIBUTION)

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ANNEX G

(*Clause* 12)

MASONRY WALLS USING RAT-TRAP BOND

G-1 GENERAL

The following covers the construction of masonry walls using rat-trap bond. Only masonry with bricks (burnt clay bricks, fly ash-clay bricks, fly ash-lime bricks and stabilized soil blocks) is dealt herein. Use of other types of bricks/blocks and mud mortar are not specifically covered hereunder.

G-2 NECESSARY INFORMATION

G-2.1 For efficient planning, design and execution of the work, detailed information with regard to the following shall be furnished to those responsible for the work:

a) layout plans showing the orientation of the structure;

b) dimensioned details of the structure with details of sections (to a suitably large scale that is 1 : 20), levels of foundation, finished ground level, clear floor to floor heights of rooms, sizes of openings, etc;

c) type and class of brickwork, types of bond and final finish for the brickwork; the mixes of mortar to be used, etc, full size details of architectural features, mouldings and other special work. Also course plans at different levels of sill and lintel to be made available for proper understanding and execution, if required; and

d) location and other details of openings, chases, embedment of service lines, such as for water supply, drainage, electrical installations, etc, and location and details of hearths, flues and chimneys in the brickwork.

G-2.2 All information as in **G-2.1** shall be made available to those who are responsible for the masonry work. Necessary drawings and instructions for planning the work shall be furnished. Arrangements shall also be made for the proper exchange of information between those engaged in masonry work and all those whose work will affect or will be affected.

NOTE — If necessary a brief training of the masons may be carried out orienting them with the technology know-how.

G-3 MATERIALS

G-3.1 Bricks

Burnt-clay bricks, heavy duty bricks and bricks made out of other suitable materials like fly ash, stabilized soil block, etc, conforming to accepted standards [C-4(12)] with compressive strength greater than 6.0 MPa shall be used.

G-3.2 Mortars

Shall conform to grade H2 mortar for good practice [C-4(3)].

G-3.3 Storage of Materials

Storage of materials shall be in accordance with good practice [C-4(2)].

G-3.4 Materials for Damp-Proof Courses

Materials for damp proof courses shall be as specified in [C-4(5)].

G-3.5 Materials for Flashing and Weathering

Lead flashing with sheet thickness between 1.6 mm and 2 mm or bitumen felts shall be in accordance with the accepted standards [C-4(13)].

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G-3.6 Metal Reinforcement

Metal reinforcement for use in brick masonry shall be as per accepted standards [C-4(14)].

G-4 DESIGN CONSIDERATIONS

G-4.1 Selection of Bricks

The brick shall be selected in accordance with Table 1 of good practice [C-4(4)].

When the requirements for strength of masonry predominate in the particular situation of use, the bricks shall be of such class to give the required strength for masonry and shall be selected in accordance with **4**.

G-4.2 Rat-Trap Bond Details

G-4.2.1 The rat-trap bond is laid by placing the bricks on edge forming a cavity of 80 mm (for a traditional brick size) with alternate course of stretchers and headers (Fig. 85). The headers and stretchers are staggered in subsequent layers to impart strength and stability to the walls.

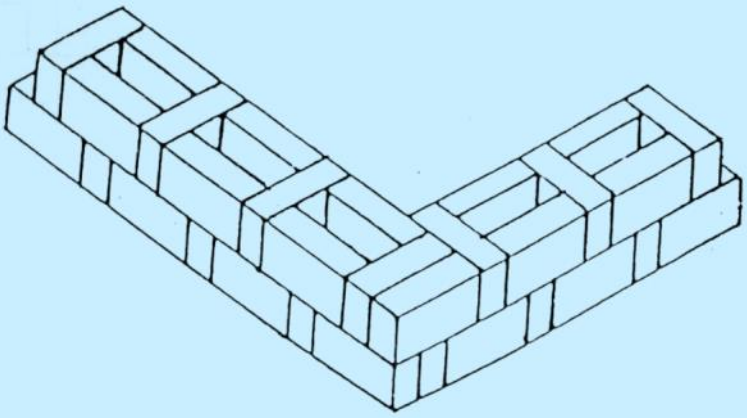

FIG. 85 ARRANGEMENT IN RAT-TRAP BOND

The primary object of bond is to give strength to masonry, but when exposed, it also creates aesthetically pleasing view. This masonry technology is modular in nature, so the dimensions of the wall to be based on the module dimensions in multiples.

G-4.2.2 A typical module detail is shown in Fig. 86 with conventional bricks of size 230 mm × 110 mm × 75 mm. The brick adjacent to the openings has to be solid as per Fig. 87. So, the opening dimensions also to be in multiple of the rat-trap bond module. This will be important as opening dimensions are predefined depending on the module dimension. In brickwork the cross joints in any course shall not be nearer than a quarter of brick length from those in the course below or above it.

G-4.3 Precautions

This type of masonry construction shall have to be carried out by trained masons to avoid wastage of mortar falling into the gap. Rat-trap bond shall never be carried out with brick which are below their minimum strength requirements.

G-4.4 Strength of Rat-Trap Bond

The compressive strength of rat-trap bond will be different from that of solid wall of similar thickness. Rat-trap bond wall provides continuous cavity over the entire wall height and hence it is convenient to reinforce if necessary to resist lateral loads. The compressive strength of the unreinforced rat-trap bond wall can be determined either based on the cavity wall design guidelines following **5.5.1.2** or the basic compressive stress can be assessed following the tests on prism specimen given at **G-7**.

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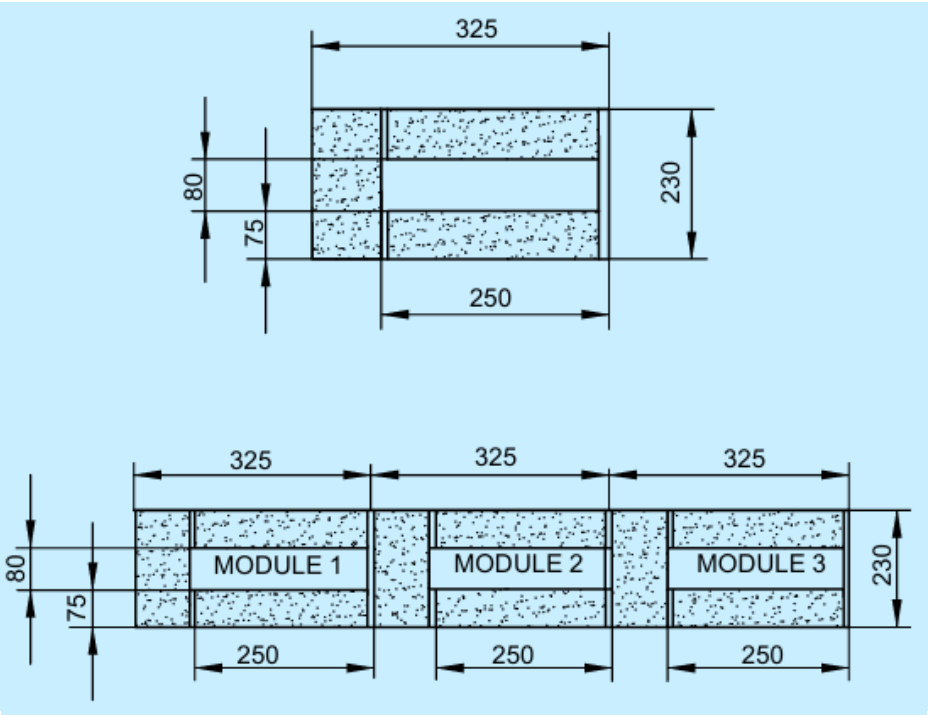
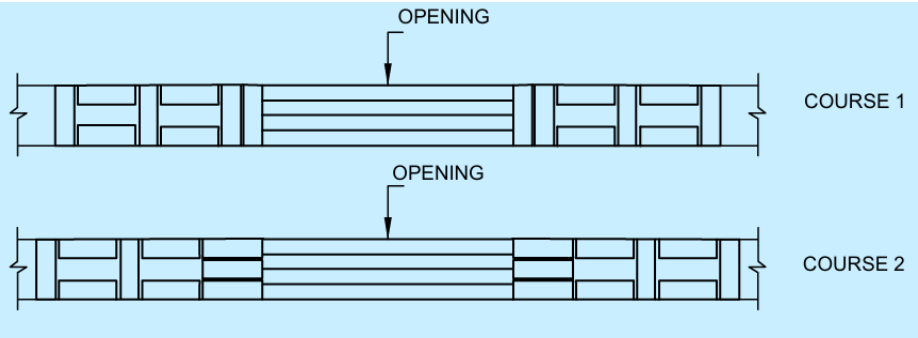


PLAN

FIG. 86 TYPICAL MODULE DETAILS OF RAT-TRAP BOND



PLAN

FIG. 87 OPENING IN RAT-TRAP BOND MASONRY

G-4.5 Thickness of Joint

No bed joint shall be thicker than 12 mm. Further the thickness of the bed joints shall be such that four courses and three joints taken consecutively shall measure as follows:

a) *In the case of traditional bricks* — Four times the actual thickness of the brick plus 30 mm.;

b) *In the case of modular bricks* — 390 mm conforming to the relevant accepted standards. The vertical mortar joints to be of the thickness of 10 mm and be applied directly to the brick before placing it. This requires some special skill, but defines the strength as well as the stability of the Rat-trap bond wall. Besides, applying the mortar joint directly to the brick is a basic skill requirement of a mason and determines the speed of work and finally the cost of the Rat-trap bond wall; and

c) The *face* jolts of brickwork may be finished by 'jointing' or by 'pointing'. In case where brick size is less, the elevation of the header bricks to be made even in line with the stretcher brick by cement mortar same grade as bedding mortar. The main reasons for pointing the surface of block or brickwork joints are to increase its weather resistance and to give a neat finish to the work. Pointing can be carried out as construction of the brickwork proceeds, using ordinary mortar in which the bricks are bedded.

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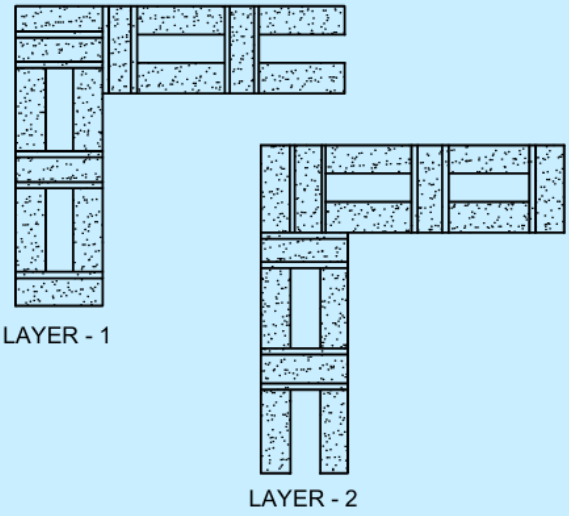
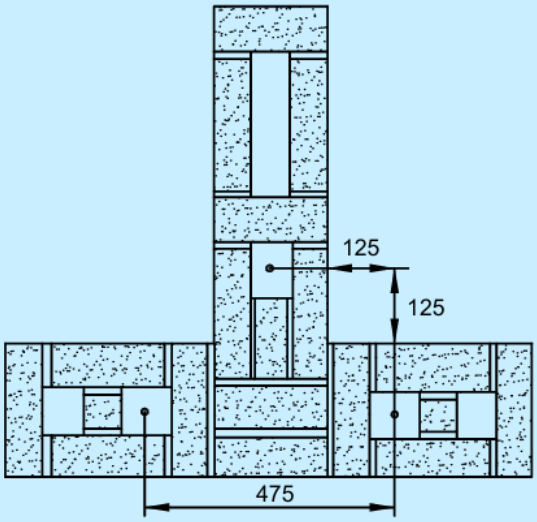


L - JUNCTION DETAIL FOR ALTERNATE LAYERS



All dimensions in millimetres.

FIG. 88 TYPICAL LAYING OF RAT-TRAP BOND

G-4.6 Structural and Functional Characteristics of Brickwork

G-4.6.1 *Structural Stability and Strength*

Reference may be made to Part C/Section 2 of this NBCS, and 4 of this Section.

G-4.6.2 *Resistance to Moisture Penetration*

The suitability of various walls under different conditions of exposure is indicated in Table 3 of good practice [C-4(4)].

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G-4.6.3 *Thermal Stresses*

The coefficient linear expansions of brick vary from 5×10^{-6}/°C to 11×10^{-6}/°C. Variations of temperature tend to produce linear changes in walls which, when restrained, may lead to internal stress resulting in cracks especially when the walls exceed 30 m. The effects of these stresses shall be taken into consideration for a proper design.

G-4.7 Control of Shrinkage Cracking

G-4.7.1 To confine cracks to the joints and to dissipate these into a large number of fine joints, it is desirable that the mortar used shall be weaker than the bricks.

G-4.7.2 Cracking due to shrinkage normally will occur at openings or other points where the vertical or horizontal section of a wall changes. A long wall with a few openings will tend to show wider cracks above and below the openings than a similar wall with many openings. Metal reinforcement may be embedded in brickwork at points where cracking is likely to occur.

G-4.8 Thermal Insulation

For requirements of thermal insulation in walls, the provisions of good practice [C-4(15)] shall apply.

G-4.9 Fire Resistance

For requirements regarding resistance to fire, the provisions of good practice [C-4(16)] shall apply.

G-4.10 Sound Insulation

Requirements for insulation against airborne sound are laid down in good practice [C-4(17)] shall apply.

G-5 LAYING OF BRICKWORK

G-5.1 Brick shall be laid on a full bed of mortar. When laying the bricks, it shall be slightly pressed so that the mortar gets into all the pores of the brick surface to ensure proper adhesion. Cross joints and wall joints shall be properly flushed and packed with mortar so that no hollow spaces are left. Properly filled joints ensure maximum strength and resistance to penetration of moisture which takes place mainly through joints.

G-5.2 The following specific guidelines shall be followed:

a) In areas with a very high ground water table, it is advisable to build the first 4 to 5 courses of the wall, with either English or a Flemish bond. This is to avoid ground and spill water related dampness problems;

b) Always clean the surface where the first layer of bricks is to be laid, which is usually above the plinth level;

c) Arrange the joints between the bricks in such a way that only full bricks (shiners) fit. Ensure that the bonding is correct;

d) To properly maintain the brick cross, lay the first two masonry courses by marking mark each of the shiner and the header bricks at the centre;

e) Place the shiner/stretcher and header brick now in such a way that the marks are exactly matching. The remaining courses may then be laid to match with the first two courses;

f) Spread a full bed of mortar, and furrow it with a trowel. This will ensure that plenty of mortar is on the bottom of the bricks for the first course;

g) Lay the corner bricks first. Carefully position the corner bricks first. Be sure to lay all bricks with the frog inside the cavity of the wall in order to create an aesthetically pleasing outer wall surface;

h) The vertical mortar joints shall be applied directly to the brick before placing it. This requires some special skill, but defines the strength as well as the stability of the rat-trap bond wall;

j) The maximum rat-trap bond brick wall height per day should not exceed more than 9 layers; and

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k) Cavities of rat-trap bond shall be closed for the first course after damp proof course, below sill level, below and above lintel and at the top most course; this can be made by a layer of headers or headers.

G-5.3 Protection Against Damage

Care shall be taken during construction that edges of jambs, sills, heads, etc are not damaged. In inclement weather, newly built work shall be covered with gunny bags or tarpaulin so as to prevent the mortar from being washed away.

G-6 INSPECTION

G-6.1 General

The object of inspection of brick masonry work is to ensure its satisfactory performance and also to ascertain whether all the recommended practice of workmanship is adopted at every stage. As the correct strength of masonry cannot be ascertained without destruction, a close supervision during the course of construction is necessary to ensure satisfactory performance. The strength of brickwork depends on the strength of individual brick, strength of mortar, bond and workmanship.

G-6.2 Inspection of Materials

In case of large works, samples of bricks, sands, *Surkhi*, lime, cement, etc, which goes to form the brickwork, shall be periodically tested in a laboratory to make sure that they conform to the requirements stated in **G-3**. Simple field tests may suffice in the case of small works.

G-6.3 Inspection of Workmanship

A close supervision while the work is in progress will ensure a better quality work with the materials available for use. The following shall be observed at the time of inspection:

a) all loose materials, dirt and set lumps of mortar which may be lying over the surface over which brickwork is to be freshly started, shall be removed with a wire brush;

b) all the bricks shall be thoroughly soaked in potable water before use;

c) the surface over which the brickwork is to be started shall be made moist;

d) plastic mortar results in thorough bedding of the brick and more complete filling of the joints which ensure greater strength. Care shall be taken to see that the required quantity of water is added to the mortar at the mixing platform itself and not over the courses;

e) all the joints shall, as far as possible, be thin and the specifications mentioned in **G-4.5** in this regard shall be strictly adhered to;

f) care shall be taken to see that there is no through joints and the lap is not less than half the width of the brick, and that all the vertical joints are properly filled with mortar;

g) the verticality of the walls and horizontality of the courses shall be checked very often with plumb bob and spirit level respectively;

h) no portion of the work shall be left more than 1 m lower than the other. Where the masonry of one part has to be delayed the work shall be 'raked back' suitably at an angle not exceeding 45° according to bond and not toothed;

j) where plastering is required to be done all the vertical as well as horizontal joints shall be raked to a depth of about 10 mm while the mortar is wet, and this will ensure satisfactory adhesion between the plaster and brickwork; and

k) care shall be taken to ensure that the brick work is kept wet for first seven days using appropriate curing methods.

G-7 TESTING METHOD FOR RAT-TRAP BOND WALL SAMPLES

G-7.1 A Prism specimen as shown in Fig. 89 shall be used. The prism specimen should have similar mortar and bricks/blocks as that to be used in the rat-trap bond wall of the structure. The prism height (H) to width (T) ratio

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should be between 2 and 5 and based on H/T ratio the prism strength should be corrected following the correction factors given in **B-1** for blockwork.

The number of prism specimens to be tested and the necessary correction factors to be applied to derive modified compressive strength of the prism shall be ascertained following the procedure given in **5.4**. The modified mean prism compressive strength shall be divided by 4 (as per **B-2**) to assess the basic compressive stress for the rat-trap bond wall, which can be used in calculating the compressive strength of rat-rap bond masonry wall. A similar test as above should be done in an English bond per coupon (*see* Fig. 90).

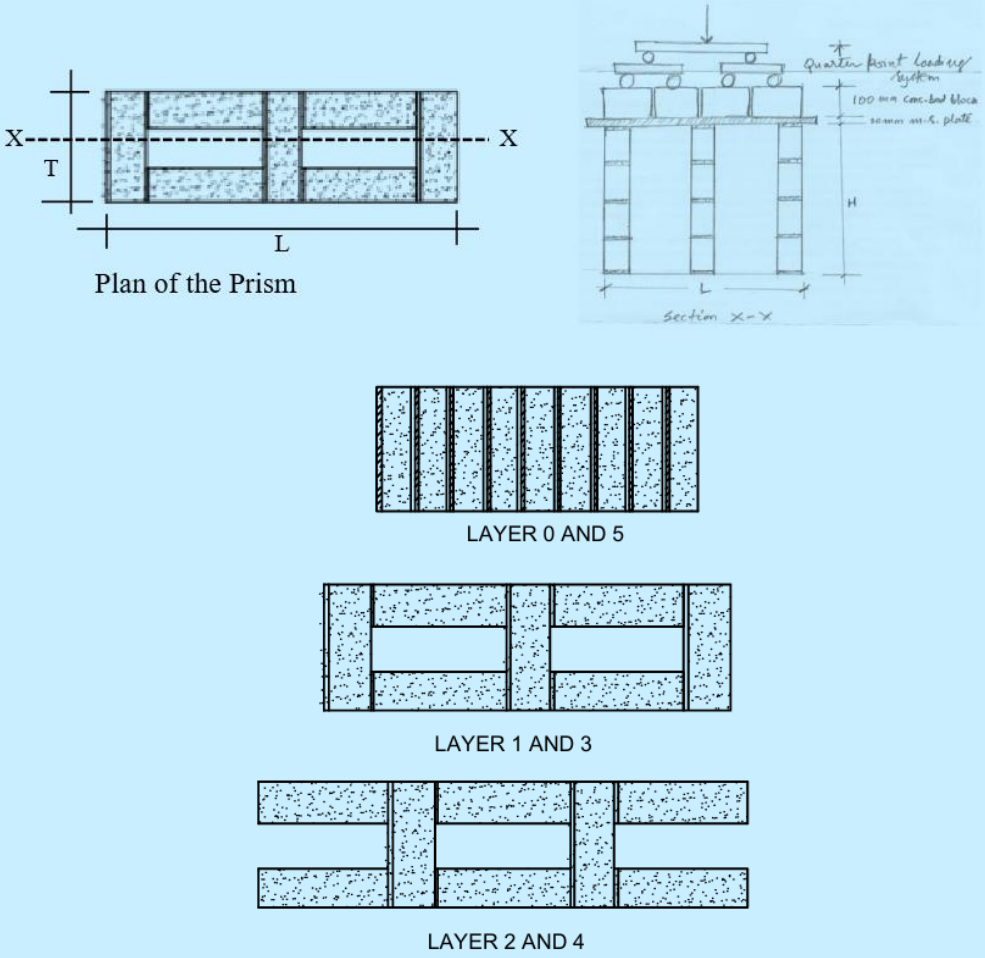

FIG. 89 PRISM SIZE AND LOADING ARRANGEMENT FOR THE RAT-TRAP BOND PRISM SPECIMEN

G-7.2 The load on rat-trap bond specimen should be not less than 85 percent of the load capacity of English bond wall compressive strength.

G-7.3 While designing the compressive stress on rat-trap bond wall, it should be ensured that the stress is not more than 0.85 of that of solid wall as per **4** of this Section.

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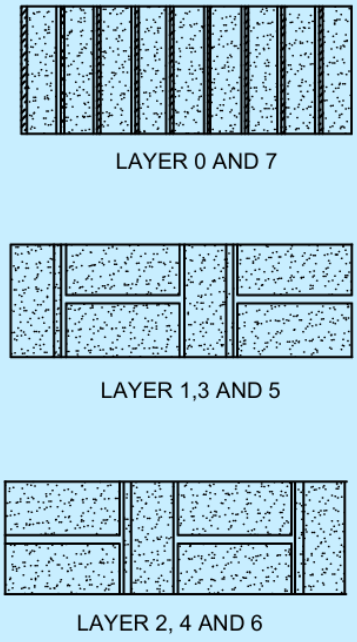

FIG. 90 ENGLISH BOND TEST SPECIMEN

ANNEX H

(*Clause* 14)

NOTATIONS, SYMBOLS AND ABBREVIATIONS

H-1 The following notations, letter symbols and abbreviations shall have the meaning indicated against each, unless otherwise specified in the text of this Section of this NBCS:

A	=	Area of a section
b	=	Width of bearing
DPC	=	Damp proof course
e	=	Resultant eccentricity
f_b	=	Basic compressive stress
f_c	=	Permissible compressive stress
f_d	=	Compressive stress due to dead loads
f_s	=	Permissible shear stress
f_m	=	Compressive strength of masonry (in prism test)

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GL	=	Ground level
H	=	Actual height between lateral supports
H'	=	Height of opening
*H*1, *H*2	=	High strength mortars
h	=	Effective height between lateral supports
k_a	=	Area factor
k_p	=	Shape modification factor
k_s	=	Stress reduction factor
L	=	Actual length of wall
*L*1, *L*2	=	Lower strength mortars
*M*1, *M*2	=	Medium strength mortars
P	=	Total horizontal load
PL	=	Plinth level
RCC	=	Reinforced cement concrete
RS	=	Rolled steel
S_p	=	Spacing of piers/buttresses/cross walls
SR	=	Slenderness ratio
t	=	Actual thickness
t_p	=	Thickness of pier
t_w	=	Thickness of wall
W	=	Resultant load
W_1	=	Axial load
W_2	=	Eccentric load
w_p	=	Width of piers/buttresses/cross walls
μ	=	Ratio of flexural strength of wall in the vertical direction to that in the horizontal direction

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LIST OF STANDARDS

The following list records those standards which are acceptable as 'good practice' and 'accepted standards' in the fulfilment of the requirements of this NBCS. The latest version of a standard shall be adopted at the time of enforcement of this NBCS. The standards listed may be used by as a guide in conformance with the requirements of the referred clauses in this NBCS.

In the following list, the number appearing in the first column within parentheses indicates the number of the reference in this Section.

	IS No.	*Title*
(1)	IS 1077 : 2025	Common burnt clay building bricks — Specification (*sixth revision*)
	IS 1725 : 2023	Specification for stabilized soil blocks used in general building construction (*third revision*)
	IS 2180 : 1988	Specification for heavy duty burnt clay building bricks (*third revision*)
	IS 2185	Concrete masonry units — Specification:
	(Part 1) : 2005	Hollow and solid concrete blocks (*third revision*)
	(Part 2) : 1983	Hollow and solid light weight concrete blocks (*first revision*)
	(Part 3) : 1984	Autoclaved cellular (aerated) concrete blocks (*first revision*)
	(Part 4) : 2008	Cellular preformed foam cellular concrete blocks
	IS 2222 : 1991	Specification for burnt clay perforated building bricks (*third revision*)
	IS 2849 : 1983	Specification for non-load bearing gypsum partition blocks (solid and hollow types) (*first revision*)
	IS 3115 : 1992	Specification for lime based blocks (*second revision*)
	IS 3316 : 1974	Specification for structural granite (*first revision*)
	IS 3620 : 1979	Specification for laterite stone block for masonry (*first revision*)
	IS 3952 : 2013	Burnt clay hollow bricks and blocks for walls and partitions — Specification (*third revision*)
	IS 4139 : 1989	Specification for calcium silicate bricks (*second revision*)
	IS 12440 : 1988	Specification for precast concrete stone masonry blocks
	IS 12894 : 2002	Specification for pulverized fuel ash lime bricks (*first revision*)
	IS 13757 : 1993	Specification for burnt clay fly ash building bricks
	IS 16720 : 2018	Pulverized fuel ash-cement bricks — Specification
(2)	IS 4082 : 1996	Recommendations on stacking and storage of construction materials and components at site (*second revision*)
(3)	IS 2250 : 1981	Code of practice for preparation and use of masonry mortars (*first revision*)
(4)	IS 2212 : 1991	Code of practice for brickwork (*first revision*)
	IS 1597	Code of practice for construction of stone masonry:
	(Part 1) : 1992	Rubble stone masonry (*first revision*)
	(Part 2) : 1992	Ashlar masonry (*first revision*)
	IS 2572 : 2005	Code of practice for construction of hollow and solid concrete block masonry (*first revision*)
	IS 2110 : 1980	Code of practice for in-situ construction of walls in building with soil-cement (*first revision*)

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	IS 2849 : 1983	Specification for non-load bearing gypsum partition blocks (solid and hollow types) (*first revision*)
	IS 3630 : 1992	Construction of non-load bearing gypsum block partitions — Code of practice (*first revision*)
	IS 6041 : 1985	Code of practice for construction of autoclaved cellular concrete block masonry (*first revision*)
	IS 6042 : 1969	Code of practice for construction of lightweight concrete block masonry
(5)	IS 2212 : 1991	Code of practice for brickwork (*first revision*)
(6)	IS 3414 : 1968	Code of practice for design and installation of joints in buildings
(7)	IS 10440 : 1983	Code of practice for construction of RB and RBC floors and roofs
(8)	IS 1904 : 2021	General requirements for design and construction of foundations in soils — Code of practice (*third revision*)
(9)	IS 883 : 2016	Code of practice for design of structural timber in building (*fifth revision*)
(10)	4326 : 2013	Earthquake resistant design and construction of buildings — Code of practice
(11)	IS 1597 (Part 2) : 1992	Code of practice for construction of stone masonry: Part 2 Ashlar masonry (*first revision*)
(12)	IS 1077 : 1992	Common burnt clay building bricks — Specification (*fifth revision*)
	IS 2180 : 1988	Specification for heavy duty burnt clay building bricks (*third revision*)
	IS 1725 : 2023	Stabilized soil blocks used in general building construction — Specification (*third revision*)
(13)	IS 405 (Part 2) : 1992	Lead sheets and strips — Specification: Part 2 for other than chemical purposes (*third revision*)
	IS 1322 : 1993	Bitumen felts for water proofing and damp-proofing (*fourth revision*)
(14)	IS 432	Specification for mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement:
	(Part 1) : 1982	Mild steel and medium tensile steel bars (*third revision*)
	(Part 2) : 1982	Part 2 Hard- drawn steel wire (*third revision*)
	IS 1566 : 1982	Specification for hard-drawn steel wire fabric for concrete reinforcement (*second revision*)
	IS 412 : 1975	Specification for expanded metal steel sheets for general purposes (*second revision*)
	IS 2062 : 2011	Hot rolled medium and high tensile structural steel — Specification (*seventh revision*)
(15)	IS 3792 :1978	Guide for heat insulation of non-industrial buildings (*first revision*)
(16)	IS 1642 : 2013	Code of practice for fire safety of buildings (general): Details of construction (*second revision*)
(17)	IS 1950 : 1962	Code of practice for sound insulation of non-industrial buildings

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 5 STRUCTURAL CONCRETE

BUREAU OF INDIAN STANDARDS

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National Building Code Sectional Committee, CED 46

FOREWORD

This NBCS (Part C/Section 5) covers the structural design aspects of plain and reinforced concrete and also the prestressed concrete.

This Section 5 was first published in 1970 having Subsections 5A 'Plain and reinforced concrete' and 5B 'Prestressed concrete' of SP 7 (Part 6) and were subsequently revised in 1983, 2005 and 2016 as a chapter of National Building Code of India.

This Section is largely based on IS 456 : 2000 'Code of practice for plain and reinforced concrete (*fourth revision*)' and IS 1343 : 2012 'Prestressed concrete — Code of practice (*second revision*)' both of which are currently under revision and are being merged in the fifth revision of IS 456, likely to be titled as 'Structural concrete — Code of practice' at the time of the publication of this NBCS.

Major changes have been envisaged in the revision of IS 456 (amalgamating IS 1343). In the absence of availability of finalized version of revised IS 456, at the time of the revision of this publication, the provision of design as per existing IS 456 : 2000 and IS 1343 : 2012 have been continued through appropriate reference to the same.

In the 2016 version of the Code, in addition to the provisions of IS 456, provisions relating to special concrete namely self-compacting concrete, high performance concrete and steel fibre reinforced concrete were introduced.

In this revision, the publication has been renamed as National Building Construction Standards (NBCS).

In this fourth revision of this Section, Subsections 5A and 5B have been merged into a single Section 5, 'Structural concrete', and the following additional changes are made:

a) Special concretes, namely self-compacting concrete**,** high-performance concrete, and steel fibre reinforced concrete, which were earlier covered in **16A**, **16B** and Annex A of the erstwhile NBC, are now included in Annex B, Annex C, and Annex D respectively;

b) Informative Annex (Annex A) on updated provisions regarding materials for making concrete is introduced;

c) Criteria for structural safety of tall concrete buildings is introduced in Annex E. This provides prescriptive requirements for design of reinforced concrete tall buildings. The following salient aspects, which are based on the prescriptive approach, are addressed in the Annex E:

   1) Structural systems admissible for tall building in different seismic zones;

   2) General requirements including: (i) height limitations of different structural systems, (ii) elevation and plan aspect ratios, (iii) lateral drift, (iv) storey stiffness and strength, (v) modes of vibration, (vi) floor systems, (vii) materials, and (viii) progressive collapse mechanism;

   3) Wind and seismic effects: (i) load combinations, and (ii) acceptable serviceability criteria for lateral accelerations;

   4) Methods of structural analysis to be adopted, and section properties (in cracked and uncracked states) of reinforced concrete member to be considered in analysis;

   5) Structural design aspects for various applicable structural systems;

   6) Issues to be considered in design of foundations; and

   7) Systems needed for structural health monitoring.

The upcoming fifth revision of IS 456 introduces a holistic design philosophy that emphasizes that concrete structures shall not only be strong but also safe, durable, serviceable under expected loads, and resilient in the face of extreme and accidental events. To this end, the standard stipulates that structural design shall meet six design criteria that are strength, serviceability, durability, robustness, integrity, and restorability. Each of these criteria

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is associated with performance objectives and corresponding design requirements, ensuring that structural behaviour is assessed comprehensively across its entire lifecycle, including performance during and after any adverse events.

The design philosophy outlined in the above revised standard includes the classification of the design process into the prescriptive process and the closed-loop process. The prescriptive process follows conventional rule-based checks and is applicable to structures where only the basic three criteria (strength, serviceability, and durability) are of concern. In contrast, the closed-loop process is intended for important or critical structures and involves a more rigorous approach, requiring not only quantitative compliance for the first three criteria but also qualitative and performance-based verification for robustness, integrity, and restorability. This two-tier approach introduces flexibility in application while ensuring that safety and resilience are not compromised, especially in vital infrastructure. By integrating modern concerns like post-disaster repairability (restorability), progressive collapse prevention (integrity), and resistance to accidental actions (robustness), the design philosophy ensures that structures are not just built to withstand known demands but also prepared for uncertainties. It promotes reliability, sustainability, and long-term performance.

The revision of IS 456 highlights that structural analysis should correctly capture how structures behave under all types of loads, including static, dynamic, and accidental forces. For regular structures, linear static analysis is generally sufficient. However, for structures affected by dynamic forces like earthquakes, wind, or machine vibrations, dynamic analysis is necessary. In critical cases, the standard also recommends using non-linear dynamic analysis to better reflect the actual behaviour of materials and structures. This approach moves towards more detailed and performance-based analysis to improve safety, serviceability, durability, and resilience in concrete structures.

Apart from drawing from IS 456 : 2000 and IS 1343 : 2012, this revision incorporates provision relating to tall concrete buildings, drawn from IS 16700 : 2023 ‘Criteria for structural safety of tall concrete buildings (*first revision*)’.

All standards, whether given herein above or cross-referred to in the main text of this Section, are subject to the revision. The parties to agreement based on this Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

As the regulation of land development and buildings is a State subject and is dealt by the States and local bodies such as urban local bodies within their jurisdiction through building regulatory documents like building regulations, building byelaws, and fire regulations, this document is voluntary in nature and is non-binding. The implementation depends on adoption by concerned parties or stipulations in a contract or by appropriate adoption by the concerned authorities.

For the purpose of deciding whether a particular requirement of this Subsection is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 2022 ‘Rules for rounding off numerical values (*second revision*)’. The number of significant places retained in the rounded off value should be the same as that of the specified value in this section.

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Important Explanatory Note for Users of the NBCS

In any Part/Section of this NBCS, where reference is made to **'good practice'** in relation to design, constructional procedures or other related information, and where reference is made to **'accepted standard'** in relation to material specification, testing, or other related information, the Indian Standards listed at the end of the Part/Section shall be used as a guide to the interpretation.

At the time of publication, the editions indicated in the standards were valid. All standards are subject to revision and parties to agreements based on any Part/ Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

In the list of standards given at the end of a Part/Section, the number appearing within parentheses in the first column indicates the number of the reference of the standard in the Part/Section. For example:

a) Good practice [C-5(2)] refers to the Indian Standard(s) give at serial number (2) of the list of standards given at the end of this Part C/Section 5, that is, IS 456 : 2000 'Plain and reinforced concrete — Code of practice (*fourth revision*)'.

b) Accepted standard [C-5(4)] refers to the Indian Standard(s) give at serial number (4) of the list of standards given at the end of this Part C/Section 5, that is, IS 269 : 2015 'Ordinary Portland cement — Specification (*sixth revision*)'.

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 5 STRUCTURAL CONCRETE

1 SCOPE

This NBCS (Part C/Section 5) deals with the structural use of plain, reinforced, and prestressed concrete.

2 TERMINOLOGY

For the purpose of this Section, the definitions given in accepted standards [C-5(1)] shall apply.

3 MATERIALS

All materials used in plain, reinforced and prestressed concrete construction shall conform to Part B 'Building Materials' of this NBCS as available.

4 STRUCTRAL DESIGN USING PLAIN, REINFORCED AND PRESTRESSED CONCRETE

The provision relating to design and general structural use of plain, reinforced and prestressed concrete, including on:

a) materials, including special concrete, workmanship, inspection and testing;

b) general design consideration;

c) special design requirements for structural members and systems; and

d) structural design: limit state method.

shall be in accordance with good practices contained in section 2, section 3, section 4 and section 5 of the good practice [C-5(2)] and section 2, section 3 and section 4 of the good practice [C-5(3)].

NOTE — At the time of publication of this section, IS 456 and IS 1343 were under revision, with IS 1343 being amalgamated into IS 456. Once the revised IS 456 is published, it shall replace the provisions given in 4.

The informative Annex A to this Section suggests the updated list of materials and few tests. Users are encouraged to investigate the possibility of applying the provisions given in Annex A which are listed in consonance with the provisions regarding materials in the good practice [C-5(2)].

5 STRUCTURAL SAFETY OF TALL REINFORCED CONCRETE (RC) BUILDINGS

For criteria for structural safety of tall reinforced concrete (RC) buildings of heights greater than 50 m and less than 250 m, normally intended for use as residential, office and other commercial buildings, reference may be made to Annex E and accepted standard [C-5(4)].

6 EARTHQUAKE RESISTANT DESIGN AND DETAILING OF STRUCTURES

The provisions on earthquake-resistant design and detailing of concrete buildings are covered in [C-5(5)].

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ANNEX A

(*Foreword* and *Clause* 4)

MATERIALS FOR MAKING CONCRETE

A-1 AGGREGATES

A-1.1 Normal Weight Aggregates

Aggregates derived from natural sources and other than natural sources (manufactured aggregates such as iron slag, copper slag, bottom ash from thermal power plants, recycled concrete aggregates and recycled aggregates), shall comply with the requirements of the accepted standard [C-5(6)]. These are natural or synthetic aggregates with in-place density (unit weight) between 22.40 kN/m^3 to 27.00 kN/m^3.

A-1.2 Heavyweight Aggregates

Natural or synthetic aggregates with in-place density (unit weight) more than 27.00 kN/m^3 and can range up to 45.00 kN/m^3 for aggregates other than steel. Steel aggregates (if used) will have further higher density. Heavy weight aggregate is most commonly used for radiation shielding, counterweights and other applications where a high mass-to-volume ratio is desired. Goethite, limonite, barite, illmenite, magnetite, hematite, ferrophosphorus and steel are some examples of heavy weight aggregate.

A-1.3 Lightweight Aggregates

Natural or synthetic with in-place density (unit weight) between 14.40 kN/m^3 to 18.40 kN/m^3. The lightweight is due to the cellular or high internal porous structure, which gives this type of aggregate a low specific gravity. The most important aspect of lightweight aggregate is the porosity. They have high absorption values, which requires a modified approach to concrete proportioning. Light weight aggregates namely sintered fly ash aggregates may be used for making light weight structural concrete (a type of special concrete). Sintered flyash coarse aggregates when used shall conform to the accepted standard [C-5(7)].

A-2 CEMENT

A-2.1 For general use in plain, reinforced and prestressed concrete construction, cement used shall be any of the following and the type selected should be appropriate for the intended use:

a) Ordinary Portland cement conforming to the accepted standard [C-5(8)];

b) Portland slag cement conforming to the accepted standard [C-5(9)];

c) Portland pozzolana cement (fly ash based) and Portland pozzolana cement (calcined clay based), conforming to the accepted standard [C-5(10)];

d) Composite cement conforming to the accepted standard [C-5(11)] and containing a declared clinker content not less than 45 percent, fly ash content not more than 25 percent and a minimum 28-day compressive strength of 43 MPa can be used for reinforced concrete construction; and

e) Portland calcined clay limestone cement conforming to the accepted standard [C-5(12)]. At locations where temperatures are predominantly below 15 °C for 6 months, the use of Portland calcined clay limestone cement shall not be permitted in underground structures and structural elements in contact with ground water.

A-2.2 Other cements as given below can be used for specific applications:

a) Sulphate resisting portland cement conforming to the accepted standard [C-5(13)];

b) Microfine ordinary portland cement conforming to the accepted standard [C-5(14)];

c) Hydrophobic cement conforming to the accepted standard [C-5(15)];

d) Low heat Portland cement conforming to the accepted standard [C-5(16)];

e) Rapid-hardening portland cement conforming to the accepted standard [C-5(17)];

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f) High alumina cement conforming to the accepted standard [C-5(18)];

g) Supersulphated cement conforming to the accepted standard [C-5(19)]; and

h) White cement conforming to the accepted standard [C-5(20)].

A-3 WATER

A-3.1 Water used for mixing and curing shall be clean and free from excessive amounts of total dissolved solids (TDS), oils, acids, alkalis, salts, sugar, organic materials or other substances that may be deleterious to steel and setting and strength development of concrete.

A-3.2 Types of Water

In general, the suitability of water for the production of concrete depends upon its origin. Water to be used for mixing and curing of concrete shall be any of the types described below. Any type of water to be used for mixing and curing shall be tested as per stated procedures and shall meet all the requirements given in **A-3.3**.

A-3.2.1 *Potable Water*

This water is considered as suitable for use in concrete if meeting the requirements of the accepted standard [C-5(21)] wherein the chloride content shall be limited to 500 PPM and alkalinity is less than or equal to 25 ml (**A-3.3**). Such water needs no further testing. In case of any doubt it shall be tested for conformity according to **A-3.3**.

A-3.2.2 *Natural Surface Water*

This water, if found suitable, can be used in concrete but shall satisfy the requirements of **A-3.3**.

A-3.2.3 *Treated Water*

Appropriately treated water, conforming to **A-3.3**, from industrial, municipal and other sources can be used in concrete.

A-3.2.4 *Water Recovered from Processes in the Concrete Industry*

This includes:

a) water that was part of surplus concrete;

b) water used to clean the inside of the stationery mixers, mixing drums of truck mixer or agitators and concrete pumps; and

c) process water from sawing, grinding and water blasting of hardened concrete.

This water may be taken from:

1) Basins provided with suitable equipment that distributes the solid matter evenly throughout the water; and

2) Sedimentation basins or similar installations, provided the water is left in the basin for a sufficient amount of time to allow the solids to settle properly.

Water recovered from process in the concrete industry contains varying concentrations of very fine particles, the size of which is generally less than 0.25 mm. Water recovered from processes in the concrete industry or combined water can be used provided the following requirements are met:

i) The additional mass of solid material in the concrete resulting from the use of water recovered from processes in the concrete industry shall be less than 1 percent mass fraction of the total mass of aggregates present in the concrete; and

ii) The amount of recovered water shall be spread as evenly as possible over a day's production.

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A-3.2.5 *Water from Other Sources*

A-3.2.5.1 *Recycled water*

This includes water recovered from concrete production operation or water generated from sewage, greywater or storm water systems and treated to a standard that is appropriate for its intended use.

A-3.2.5.2 *Desalinated water*

Desalinated water conforming to **A-3.3** can be used. However normal sea water or brackish water shall not be used for mixing of concrete because of presence of harmful salts.

A-3.3 Requirements for Preliminary Inspection of Mixing and Curing Water

A-3.3.1 *Preliminary Inspection*

The water shall be examined for oil and fats, detergents, colour, suspended matter, odour and humic matter shall meet the requirements given in Table 1 in accordance with test procedures stated below.

A small subsample shall be assessed as soon as possible after sampling. Bring any material that has settled back into suspension by shaking the sample. Pour 80 ml of the sample into a 100 ml measuring cylinder. Seal with a suitable stopper and shake the cylinder vigorously for 30 s. Smell the sample for any odours other than those of clean water. If in doubt about the odour, test the water for its odour level in accordance with national regulations for potable water. The odour level of the water shall be lower than the maximum level accepted for potable water. Observe the surface for foam. Set the cylinder in a place free from vibration and allow to stand for 30 min. After 2 min, check the sample for the continuing presence of foam and signs of any oils or fats. After 30 min have elapsed, observe the apparent volume of the settled solids and the colour of the water. Measure the pH using indicator paper or a *p*H metre. Then, add 0.5 ml hydrochloric acid, mix and then smell or test for the presence of hydrogen sulphide.

Put 5 ml of the sample into a test tube. Bring it to a temperature between 15 °C and 25 °C by allowing it to stand indoors. Add 5 ml of 3 percent sodium hydroxide solution, shake and leave for 1 h. Observe the colour.

Table 1 Requirements for Preliminary Inspection of Mixing Water

(*Clause* A-3.3.1)

Sl No.	Parameter	Requirements
(1)	(2)	(3)
i)	Oils and fats	Not more than visible traces
ii)	Detergents	Any foam should disappear within 2 min
iii)	Colour	Water not from sources classified as potable: the colour shall be assessed qualitatively as pale yellow or pale
iv)	Odour	Water from sources classified as potable: no smell, except the odour allowed for potable water and a slight smell of cement; where blast-furnace slag is present in the water, a slight smell of hydrogen sulphide Water from other sources: no smell, except the odour allowed for potable water; no smell of hydrogen sulphide after addition of hydrochloric acid
v)	*p*H value	6.0 to 8.5
vi)	Humic matter	The colour shall be assessed qualitatively as yellowish/brown or paler after addition of NaOH

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A-3.3.2 Permissible limits for solids shall be as given in Table 2.

Table 2 Maximum Permissible Limit for Solids for Mixing and Curing Water

(*Clauses* A-3.3.2 *and* A-3.4)

Sl No.	Parameter	Tested as per Accepted Standard	Maximum Permissible Limit	
			For Mixing Water	For Curing Water
(1)	(2)	(3)	(4)	(5)
i)	Organic	[C-5(22)]	200 mg/l	200 mg/l
ii)	Inorganic	[C-5(22)]	3 000 mg/l	3 000 mg/l
iii)	Sulphate (as SO_3^-)	[C-5(23)]	400 mg/l	800 mg/l
iv)	Chloride (as Cl^-)	[C-5(24)]	2 000 mg/l for concrete not containing embedded steel and 500 mg/l for reinforced concrete work[1)]	500[1)] mg/l for reinforced concrete or prestressed concrete work and 2 000 mg/l for concrete not containing embedded steel
v)	Suspended matter	[C-5(25)]	2 000 mg/l	2 000 mg/l

[1)] 1 000 mg/l chloride in water for reinforced concrete work (but not for prestressed concrete) can be permitted provided the chloride content of hardened concrete is checked at the time of concrete mix design and conform to requirement of total chloride content in the concrete.

A-3.4 Testing and Requirements of Curing Water

Permissible limits for solids shall be as given in Table 2. Water used for curing should not produce any objectionable stain or unsightly deposition on the concrete surface. The presence of tannic acid or iron compounds is objectionable.

A-3.5 Sampling of Water Concrete Making and Curing

A sample of water not less than 5 litre shall be taken. The sample of water taken for testing shall represent the water proposed to be used for concreting with due account being paid to seasonal variation. The sample shall be stored in a clean container previously rinsed out with similar water.

A-4 ADMIXTURES

A-4.1 Admixtures can be used to:

a) Alter the composition of concrete by replacing some of the cement in the concrete by pozzolanic and hydraulic reaction (and are called mineral admixtures or supplementary cementitious materials);

b) Enhance a certain function of concrete (and are called chemical admixtures); and

c) The following types of admixtures shall be used in production of concrete.

A-4.2 Mineral Admixtures/Supplementary Cementitious Materials (SCMs)

SCMs as given below may be used as part replacement of cement. They can be pozzolanic material or reactive mineral admixture or combination of both.

A-4.2.1 *Fly Ash* (*Pulverized Fuel Ash*)

Fly ash conforming to the accepted standard [C-5(26)] may be used as part replacement of ordinary Portland cement. For concrete having steel, the replacement level of fly ash shall be restricted to 35 percent by weight of

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cementitious material wherein fly ash shall have a minimum fineness of 280 m^2/kg and lime reactivity of 4.5 N/mm^2 or more.

A-4.2.2 *Silica Fume*

Silica fume conforming to the accepted standard [C-5(27)] may be used as part replacement of cement. Silica fume is usually used in proportion of 3 percent to 10 percent of the cementitious content of a mix. The total replacement by fly ash and silica fume shall not be more than 35 percent for reinforced cement concrete. The total replacement by GGBF slag and silica fume/Ultrafine ground granulated blast furnace slag (UFGBS) shall not be more than 60 percent for reinforced cement concrete.

The above limits shall also apply where PPC or PSC is used. The fly ash or slag content mentioned on the bag shall be considered.

Only for mass concrete, structures below ground (not exposed to atmosphere) and for plain concrete, the use of more than one type of mineral admixture with ordinary Portland cement or a mineral admixture with blended cements such as PPC and PSC can be permitted after necessary trials (to control heat of hydration). However, silica fume may be used as an additional mineral admixture in reinforced concrete subjected to limits specified above. Uniform blending of the mineral admixtures with the cement should be ensured. In such cases, fly ash and GGBS together may be used within following limits:

Total replacement (*Max*)	=	45 percent of total cementitious content
Fly ash (*Max*)	=	20 percent
GGBS (*Max*)	=	25 percent

A-4.2.3 *Metakaolin*

Metakaolin conforming to the accepted standard [C-5(28)] may be used as pozzolanic material in concrete as part replacement of ordinary Portland cement only. The replacement level shall be maximum up to 15 percent.

A-4.2.4 *Ground Granulated Blast Furnace Slag* (*GGBS*)

GGBS conforming to the accepted standard [C-5(29)] may be used as part replacement of ordinary Portland cement. For concrete having embedded steel, the replacement level of GGBS shall be restricted to 50 percent by weight of cementitious materials. For resistance to sulphate and alkali aggregate reaction, GGBS up to 60 percent can be used.

A-4.2.5 *Ultrafine Ground Granulated Blast Furnace Slag* (*UFGBS*)

UFGBS conforming to the accepted standard [C-5(30)] may be used as part replacement of cement. The replacement level shall be up to 20 percent.

A-4.2.6 *Ultrafine Fly Ash*

Ultrafine fly ash produced through classification route and conforming to the accepted standard [C-5(31)] may be used as part replacement of ordinary Portland cement. The replacement level shall be up to 20 percent.

A-4.3 Chemical Admixtures

A-4.3.1 Chemical admixtures, as mentioned in Table 3 if used, shall comply with the accepted standard [C-5(32)]. Previous experience with and data on such materials should be considered in relation to the likely standards of supervision and workmanship to the work being specified.

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Table 3 Types of Chemical Admixtures and their Constituents

(*Clause* A-4.3.1)

Sl No.	Type of Admixture	Typical Active Constituents	
(1)	(2)	(3)	
i)	Air-entraining	a)	Salts of wood resins
		b)	Some synthetic detergents
		c)	Salts of sulphonated lignin
		d)	Salts of petroleum acids
		e)	Salts of proteinaceous material
		f)	Fatty and resinous acids and their salts
		g)	Tall oils and gum rosin salts
		h)	Alkylbenzene sulfonates
		j)	Salts of sulphonated hydrocarbons
ii)	Retarding	a)	Unrefined lignosulphonate containing sugars
		b)	Modified derivatives of unrefined lignosulphonate
		c)	containing sugars
		d)	Hydroxycarboxylic acids and their salts
		e)	Carbohydrates including sugars
		f)	Heptonates related to sugars and starches
iii)	Accelerating	a)	Calcium chloride
		b)	Triethanolamine
		c)	Sodium thiocyanate
		d)	Sodium formate or calcium formate
		e)	Sodium nitrite or calcium nitrite
		f)	Calcium nitrate
		g)	Aluminates
		h)	Silicates
iv)	Water-reducing	a)	Lignosulfonic acids and their salts
		b)	Hydroxylated carboxylic acids and their salts
		c)	Polysaccharides
		d)	Melamine polycondensation products
		e)	Naphthalene polycondensation products
		f)	Polycarboxylates
v)	High range water-reducing : Normal and retarding	a)	Melamine sulfonate polycondensation products
		b)	Naphthalene sulfonate polycondensation products
		c)	Polycarboxylates
vi)	Shrinkage reducing	a)	Polyoxyalkylene alkyl ether
		b)	Propylene glycol

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Table 3 (*Concluded*)

(*Clause* A-4.3.1)

Sl No.	Type of Admixture	Typical Active Constituents
(1)	(2)	(3)
vii)	Corrosion inhibiting	a) Amine carboxylates aminoester organic emulsion
		b) Calcium nitrite
		c) Organic alkyidicarboxylic
		d) Chromates
		e) Phosphates
		f) Hypophosphites
		g) Alkalis
		h) Fluorides
viii)	Permeability-reducing admixture	
	1) *Non-hydrostatic conditions* (*PRAN*)	a) Long-chain fatty acid derivatives (stearic, oleic, caprylic capric)
		b) Soaps and oils (tallows, soya-based)
		c) Petroleum derivatives (mineral oil, paraffin)
		d) Fine particle fillers (silicates, talc)
	2) *Hydrostatic conditions* (*PRAH*)	Crystalline hydrophilic polymers (latex, water-soluble, or liquid polymer)

A-4.3.2 Chemical admixtures can be achieved by a spectrum of materials and manufacturing processes, but shall comply with accepted standard [C-5(28)]. Further, **c**hemical admixtures can be achieved by a spectrum of materials and manufacturing processes, but shall comply with the accepted standard [C-5(32)]. Further,

a) The workability, compressive strength and the slump loss of concrete with and without the use of admixtures shall be established during the trial mixes before use of admixtures;

b) The relative density of liquid admixtures shall be checked for each drum containing admixtures and compared with the specified value before acceptance;

c) If two or more admixtures are used simultaneously in the same concrete mix, data shall be obtained to assess their interaction and to ensure their compatibility;

d) Admixtures shall neither impair durability of concrete nor combine with the constituent to form harmful compounds nor increase the risk of corrosion of reinforcement; and

e) The chloride content of admixtures shall be independently tested for each batch before acceptance.

A-4.3.3 The amount of admixture added to a mix shall be recorded in the production record. Additional dose of admixture may be added at project site and mixed adequately in mixer itself to regain the workability of concrete (if necessary) with mutual agreement between the producer/supplier and the purchaser/user of concrete. But, the producer/supplier shall assure the ultimate quality of concrete supplied by him and maintain record of quantity and time of addition.

A-4.3.4 Shrinkage compensating admixtures shall not be used in structural concrete.

A-5 FIBRES

Fibres may be added to concrete for special applications to enhance select properties. But, such fibres shall be approved by the competent authority on the recommendation of a duly constituted competent peer review committee. In any case, the presence of fibres shall not be taken advantage of in the estimation of strength.

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A-6 REINFORCEMENT

A-6.1 Reinforcing bars and prestressing strands shall be specified in accordance with the provisions of good practices [C-5(2) and (3)], respectively.

A-6.2 Reinforcement couplers shall comply with the relevant clauses of accepted standard [C-5(33)].

A-6.3 If glass fibre reinforced polymer bars are used, the material shall conform to the requirement of the accepted standard [C-5(34)].

A-6.4 Welded wire reinforcement shall conform to the requirements of the accepted standard [C-5(35)].

A-6.5 Prestressing accessories shall comply with the requirements of good practice [C-5(3)].

ANNEX B

(*Foreword*)

SELF COMPACTING CONCRETE

B-1 GENERAL

The self compacting concrete (SCC) is highly flowable, non-segregating concrete that fills uniformly and completely every corner of formwork by its own weight without the need for any compaction and encapsulates reinforcement or any other embedment. The provisions relating to its application areas, features, mix proportioning principles, and production and engineering properties are given hereunder. The provisions for concrete making materials and concrete as given in ‘materials, workmanship, inspection and testing’ section of the good practice [C-5(2)] shall apply to SCC except where different provisions or departures have been indicated hereunder.

B-2 APPLICATION AREAS

SCC may be used in in-situ concrete or for precast concrete applications. It is particularly appropriate for sections with highly congested reinforcement such as, nuclear power plant structures, machine foundations, piers and abutments; and areas having restricted access to concrete placement and compaction like shafts in hydropower structures. SCC is also suitable for the construction of tunnel lining sections and casting of concrete-filled steel tubular columns. SCC may also be used for columns and beams in normal construction with appropriate formwork design.

SCC may be produced in a batching plant on site or in ready-mixed concrete plant and transported to site by transit mixers. It may be placed either by pumping or other placement methods into horizontal or vertical forms.

B-3 FEATURES OF FRESH SELF COMPACTING CONCRETE

The features of fresh SCC are:

a) Filling ability (flowability);

b) Passing ability;

c) Segregation resistance; and

d) Viscosity.

B-3.1 Filling Ability (Flowability)

Filling ability of SCC determines its ability to flow into and fill all spaces within the formwork, under its own weight. The filling ability is tested using slump-flow test.

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B-3.1.1 *Procedure*

Slump flow test shall be carried out in accordance with the accepted standard [C-5(36)].

B-3.1.2 *Classes of Slump-Flow and their Application*

There are three classes of slump-flow, as follows:

a) *SF*1 (slump flow 550 mm to 650 mm) – This class of SCC is appropriate for:

   1) unreinforced or slightly reinforced concrete structures that are cast from the top with free displacement from the delivery point (such as slabs); and

   2) casting by a pump injection system (such as tunnel linings), and 3) sections those are small enough to prevent long horizontal flow (example, piles and some deep foundations).

b) *SF*2 (slump flow 650 mm to 750 mm) is suitable for many normal applications (example, walls, columns); and

c) *SF*3 (slump flow 750 mm to 850 mm) is typically produced with a small maximum size of aggregates (less than 20 mm) and is used for vertical applications in structures having congested reinforcement, structures with complex shapes, etc. *SF*3 will often give better surface finish than *SF*2 for normal vertical applications but segregation resistance is more difficult to control.

B-3.2 Passing Ability (Free from Blocking at Reinforcement)

Passing ability describes the capacity of the fresh mix to flow through confined spaces and narrow openings such as areas of congested reinforcement without segregation. If there is little or no reinforcement, there may be no need to specify passing ability as a requirement. L-box test is generally carried out to check the passing ability and the procedure shall be in accordance with the accepted standard [C-5 (36)]. In this test, the height of the concrete left in the vertical section (h_1) and at the end of the horizontal section (h_2) is measured. The ratio of h_2/h_1 is calculated as the blocking ratio; which shall be between 0.8 and 1.0.

B-3.3 Segregation Resistance (Stability)

Segregation Resistance is the ability of fresh concrete to remain homogeneous in composition while in its fresh state. Segregation resistance is generally carried out using sieve test to check this property of fresh concrete.

B-3.3.1 *Procedure*

Segregation resistance (sieve) test shall be carried out in accordance with accepted standard [C-5(36)].

B-3.3.2 *Test Results*

There are two classes of segregation resistance, namely *SR*1 and *SR*2. For *SR*1 class, segregation resistance should be above 15 percent and $\leq$ 20 percent, and for *SR*2 it should be $\leq$ 15 percent.

*SR*1 is generally applicable for thin slabs and for vertical applications with a flow distance of less than 5 m and a confinement gap greater than 80 mm. *SR*2 is preferred in vertical applications if the flow distance is more than 5 m with a confinement gap greater than 80 mm in order to take care of segregation during flow. Segregation resistance becomes an important parameter with higher slump-flow classes and/or the lower viscosity class, or if placing conditions promote segregation. If none of these apply, it is usually not necessary to specify a segregation resistance class.

B-3.4 Viscosity

Viscosity can be assessed by the V-funnel flow time. Concrete with a low viscosity will have a very quick initial flow and then stop. Concrete with a high viscosity may continue to creep forward over an extended time.

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B-3.4.1 *Procedure*

Viscosity by V-funnel test is carried out in accordance with accepted standard [C-5 (36)]. A V-shaped funnel is filled with fresh concrete and the time taken for the concrete to flow out of the funnel is measured and recorded as the V-funnel flow time.

B-3.4.2 *Test Results*

The viscosity is divided into two classes, namely V_1 and V_2. For V_1 class, the time taken to pass the concrete from V-funnel shall be $\leq$ 8 s and for V_2 class, it shall be above 8 s and $\leq$ 25 s.

V_1 has good filling ability even with congested reinforcement. It is capable of self levelling and generally has excellent surface finish. V_2 class viscosity is more likely to exhibit thixotropic effects, which may be helpful in limiting the formwork pressure or improving segregation resistance. But it may cause negative effects on surface finish and sensitivity to stoppages or delays between successive lifts.

B-4 MIX PROPORTIONING

B-4.1 Mix Proportion Principles

The principles listed below shall be followed:

a) Lower coarse aggregate content;

b) Increased paste content;

c) Low water/powder ratio (*see* Note);

d) High range superplasticizer; and

e) Viscosity modifying admixture, if required.

NOTE — Powder is the material of particle size smaller than 0.125 mm. It includes this size fraction in the cement, mineral admixtures and aggregate.

B-4.2 Mix Design Procedure

For mix design procedure, reference may be made to good practice [C-5(37)].

B-5 PRODUCTION OF SELF COMPACTING CONCRETE

B-5.1 General

SCC is less tolerant to changes in constituent characteristics and batching variances as compared to conventional concrete. Accordingly, it is important that all aspects of the production and placing processes are carefully supervised.

The production of SCC should be carried out in plants where the equipment, operation and materials are suitably controlled under a Quality Assurance Scheme. It is important that all personnel who will be involved in the production and delivery of SCC are adequately trained including practical demonstrations.

B-5.2 Constituents

SCC is more sensitive than conventional concrete to variation in the physical properties of its constituents and especially to changes in aggregate moisture content, grading and shape. So, more frequent production checks are necessary. The moisture content of aggregates should be continuously monitored and the mix adjusted to account for any variation.

Storage of constituent materials for SCC shall be same as that for materials for conventional concrete. However, as the SCC mix is more sensitive to variations, special precautions shall be taken for storage of aggregates, which should be properly stored to avoid cross-contamination between different types and sizes and protected from weather to minimize the fluctuation of surface moisture content and movement of fines. Ground stock should be

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stored in purpose-built partitioned bays, which should allow free drainage of excess moisture in the aggregates and rainwater.

B-5.3 Mixing Equipment and Trial Mixes

SCC shall be produced with efficient forced action concrete mixers. The time necessary to achieve complete mixing of SCC may be longer than for conventional concrete due to reduced frictional forces and to fully activate the superplasticizer.

B-5.4 Formwork

Formwork should be watertight and grout tight when placing SCC. The need to design the formwork for water tightness is greater than conventional formwork so as to avoid honeycombs and surface defects. The high fluid nature of SCC may lead to higher formwork pressure than conventional concrete, especially when the casting rate is high. As a result of the highly fluid nature of SCC, a high placement rate is likely and should be anticipated. Formwork designs, that accommodate the expected liquid head formwork pressure, can allow unrestricted placement rates and permit fast casting rate of the SCC. It is recommended to design the formwork for full liquid head.

B-5.5 Curing

Curing is essential for all concrete, and early protection of exposed surfaces is essential to preventing rapid moisture loss that could lead to plastic shrinkage cracking. The top-surface of elements made with SCC can dry quickly because of the increased quantity of paste, the low water/fines ratio and the lack of bleed water at the surface. Initial curing should therefore commence as soon as practicable after placing and finishing in order to minimize the risk of surface crusting and shrinkage cracks caused by early age moisture evaporation.

B-6 ENGINEERING PROPERTIES

B-6.1 General

SCC and conventional concrete of similar compressive strength have comparable properties.

B-6.2 Compressive Strength

SCC with a similar water-cement ratio usually have a slightly higher strength compared to conventional concrete due to the lack of vibration giving an improved interface between the aggregate and hardened paste.

B-6.3 Tensile Strength

SCC may be supplied with any specified concrete grade. For a given concrete grade, the tensile strength may be safely assumed to be the same as the one for a conventional concrete as the volume of paste (cement + fines + water) has no significant effect on tensile strength.

B-6.4 Static Modulus of Elasticity

Increasing the paste volume may decrease the modulus of elasticity, E_c. As SCC has higher paste content than conventional concrete, some difference can be expected and the E_c may be somewhat lower and should be adequately covered by the safe assumptions in design.

If SCC does have a slightly lower E_c than conventional concrete, in prestressing work, this will affect the relationship between the compressive strength and the camber due to pre-tensioning or post-tensioning. For this reason, careful control should be exercised over the strength at the time when the pre-tensioning and post-tensioning strands or wires are released.

B-6.5 Creep

Due to the higher volume of cement paste, the creep coefficient for SCC may be expected to be higher than for conventional concrete of equal strength, but such differences are small and shall be covered by the safe assumptions in design.

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B-6.6 Shrinkage

As concrete compressive strength is related to the water-cement ratio, in SCC with a low water-cement ratio drying shrinkage reduces, but with increased cementitious content and increased water content the drying shrinkage and autogenous shrinkage can increase.

B-6.7 Durability

SCC is free from shortcomings due to improper compaction and result in consistently low and uniform permeability, offering less weak points for deleterious actions of the environment.

ANNEX C

(*Foreword*)

HIGH PERFORMANCE CONCRETE

C-1 GENERAL

High performance concrete is a concrete whose ingredients, proportions and production methods are specifically chosen to meet special performance and uniformity requirements. The higher performance requirements may be:

Sl No.	*Higher Performance Requirements*	*Remarks/Applications*
(1)	(2)	(3)
i)	High resistance to deterioration of reinforced concrete due to carbonation	This is to help achieve longer service life. This is particularly applicable in semi-arid regions
ii)	High resistance to chloride attack/ingress	This is for longer service life of reinforced concrete exposed to severe chloride environment/coastal environment
iii)	High resistance to abrasion	This may be required in concrete structures, like parts of hydraulic structures exposed to continuous abrasive action due to silt, such as spillways/glacis; and concrete pavements exposed to heavy traffic
iv)	High resistance to impact	This may be required in structures like blast resistant structures and spillways
v)	Improvement in flexure and shear performance	This is required in bridge girders, joints of RCC framed structure, shear walls, etc
vi)	High resistance to cracking due to shrinkage (plastic and drying shrinkage)	This may be required in concrete/mortar used for repair of distressed concrete structures, etc

High strength concrete generally improves performance of all above parameters. The concrete mix proportioning for high strength concrete may be carried out in accordance with the good practice [C-5(37)]. However, special measures are needed to further improve the specific performance requirements. As a guide, the measures given in **C-2** may be adopted. For further details, expert literature may be referred.

C-2 MEASURES FOR IMPROVING SPECIFIC PERFORMANCE REQUIREMENTS

C-2.1 Resistance to Carbonation

Resistance to carbonation of concrete is dependent mainly upon:

a) Type of binder, content of binder and *p*H value of concrete; and

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b) Permeability of concrete.

Addition of silica fume, GGBS and fly ash improve denseness and reduce permeability, thus increasing resistance to ingress of carbon dioxide and moisture (needed for carbonation). The addition of fly ash above 35 percent tends to lower the alkalinity, and thus the resistance to carbonation. Lower water-cement ratios and higher grades of concrete can be adopted to improve performance in this case. Minimum OPC content is essential for adequate carbonation resistance for different exposure conditions.

Carbonation resistance can be checked by using accelerated carbonation tests (using 3 percent to 4 percent CO_2 concentrations for minimum 70 days exposure) at mix proportioning stage. For quality assurance purpose, rapid tests like rapid chloride ion penetration test (RCPT), air permeability test or electrical resistivity test may be used. Their values may be established by correlation with accelerated carbonation test values at mix proportioning stage.

C-2.2 Resistance to Chloride Ingress

To improve performance of concrete exposed to chloride environment, two aspects need to be considered:

a) Total initial chloride content in concrete should be minimized; and

b) Permeability of concrete should be low.

Whereas chloride content of concrete can be controlled by checking chloride content of mix ingredients, the permeability of concrete can be reduced by careful selection of mix proportions and by use of supplementary cementing materials, either through use of blended cements like PPC and PSC or by direct addition of mineral admixtures, like fly ash, GGBS and silica fume. To control permeability, durability test of resistance to chloride ions shall be carried out by using chloride diffusion test by immersion or ponding method (using 3 percent NaCl exposure for 90 days) at mix proportioning stage. For quality assurance purpose rapid tests corresponding values like rapid chloride ion penetration test (RCPT) or electrical resistivity test shall be carried out. Corresponding values for these tests shall be established by developing a correlation with chloride diffusion test (by immersion/ponding) at mix proportioning stage.

C-2.3 Resistance to Abrasion

To ensure good performance against abrasion, following points should be considered:

a) Use of higher grades of concrete;

b) Use of good quality strong aggregate;

c) Use of silica fume; and

d) Use of steel fibres (suited for increasing abrasion resistance in spillways/glacis, etc).

Test methods for measuring abrasion resistance are:

a) Revolving disc type test; and

b) Under water abrasion resistance test using steel balls.

For further details regarding test methods and criteria, specialist literature may be referred.

C-2.4 Improving Performance in Flexure, Shear, Impact, Ductility and Energy Absorption

Resistance to impact is generally increased when the strength of concrete is high and energy absorption is high. Energy absorption and toughness can be increased by use of steel fibres in concrete. Whereas addition of steel fibres, glass or carbon fibres improves flexural and shear strength also, the use of synthetic fibres like polypropylene fibres do not significantly improve the above properties. Polypropylene fibres do however reduce shrinkage cracking and are suitable for concrete/mortar used for concrete repairs.

For checking impact resistance, falling weight impact test can be used, wherein number of blows for crack initiation in small concrete slab panel are counted.

For further details regarding test methods and criteria, specialist literature may be referred. The use of steel fibre reinforced concrete shall be further governed by **C-2.4.1**.

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C-2.4.1 *Use of Steel Fibre Reinforced Concrete* (*SFRC*)

Following properties of concrete can be improved by using SFRC:

a) Ductility;

b) Toughness and energy absorption;

c) Abrasion; and

d) Flexural strength and shear strength.

Compressive and direct tensile strength gain in flexural and direct tensile strength is very significant when higher percentage of steel fibres are used which give the concrete a strain hardening effect (increase in ultimate load after initial crack). However significant improvements are achieved in strain softening zone also.

Steel fibres having aspect ratio 60 to 80 may be incorporated in concrete of different grades to improve the ductility of concrete. Fibre content varying from 0.5 percent to 2.5 percent volume may be incorporated in to the concrete to improve different mechanical and durability characteristics. However higher addition can cause problems in uniform mixing and placing of concrete which need to be checked at mix proportioning stage before using the mix. Corrosion of steel fibres can also occur particularly in lower grades, so use of coated fibres and higher grades of concrete are recommended. Alternatively, a plain concrete cover of few millimetre can also be provided.

Design of steel fibre reinforced concrete (SFRC) for structural performance shall be in accordance with Annex D.

ANNEX D

(*Foreword and Clause* C-2.4.1)

DESIGN OF STEEL FIBRE REINFORCED CONCRETE

D-1 GENERAL

This Annex shall apply where steel fibres are used to improve the performance and capacity of reinforced and prestressed concrete structures.

The design of steel fibre reinforced concrete (SFRC) at both the ultimate and serviceability limit states shall be based on the stress (σ) ≠ strain (ϵ) relationships for SFRC as specified in **D-3.3** (*see* Note 1).

Steel fibres shall not be relied upon in plastic hinge regions for strength requirements (*see* Note 2).

Steel fibres shall not be relied upon at constructions joints for either serviceability or strength requirements. Design procedure in this clause is for steel-fibre-reinforced concrete with a softening classification only (*see* Fig. 1). Hardening SFRC and the use of synthetic fibres is beyond the scope of this annex.

NOTES

1 When using brittle fibres that rely on a fibre pull-out failure mechanism to obtain member ductility, care is required for cases where a significant number of fibres fracture or where fibres result in local crushing of the concrete due to the local forces imposed on the matrix by the fibres. Fibre fracture may occur where the bond between the fibre and the matric is high, and this is more likely in a high strength concrete combined with fibres of high bond capacity and of lower strength steels.

2 The anchorage capacity of steel fibres may be lost in areas where crack widths exceed about 3.5 mm. Cracking of this magnitude can be expected in regions where plastic rotations are expected.

D-2 DEFINITIONS

For the purposes of this Annex the following definitions shall apply:

D-2.1 Crack Mouth Opening Displacement (CMOD) — The width of a crack shall be measured at its mouth in a flexural tensile test. To measure the CMOD, a displacement transducers/clip gauge shall be mounted along the longitudinal axis at the mid width of the test specimen (beam with nominal size of 150 mm × 150 mm and length L so that 550 mm < L < 700 mm).

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D-2.2 Crack Opening Displacement (COD) — The width of a single localized crack, taken as an average on four sides, for a direct tensile test on a strain softening SFRC dog-bone shaped specimen in accordance with D-3.3.7.

D-2.3 Steel Fibre Reinforced Concrete (SFRC) — A mixture of concrete and steel fibres.

D-2.4 Hardening Behaviour — A material that displays an enhanced strength with increasing crack widths beyond that measured at the point of initial cracking of the cementitious matrix.

D-2.5 Softening Behaviour — A material that exhibits a loss of strength upon cracking.

D-2.6 Target Dosage — The specified quantity of fibres in kilograms per cubic metre of concrete (kg/m^3).

D-3 PROPERTIES OF SFRC

D-3.1 General

SFRC shall be classified in terms of both its characteristic compressive (cylinder) strength (f'_c) (*see* D-3.2) and its characteristic residual tensile strength ($f'_{1.5}$) (*see* D-3.3.3).

D-3.2 Compressive Strength

The characteristic compressive strength of SFRC at 28 days (f'_c) shall be determined in accordance with the accepted standard [C-5(38)].

In the absence of more accurate data, the mean value of the in situ compressive strength (f_{cmi}) shall be taken as 90 percent of the mean value of the cylinder strength ($f$cm).

NOTE — The compressive strength of steel fibre reinforced concrete (SFRC) should be determined by means of standard tests on concrete cylinders. The addition of steel fibres to concrete in conventional dosages (less than 100 kg/m^3) does not change the compressive properties of the concrete.

D-3.3 Tensile Properties

D-3.3.1 *Classification*

SFRC shall be classified as either softening or hardening as shown in Fig. 1. Hardening SFRC is outside the scope of this Annex.

NOTES

1 A hardening material is defined as one with a tensile strength equal to or greater than 1.1 times the strength of the matrix without fibres and taken at a crack opening displacement (COD) of equal to or greater than 0.3 mm.

2 A hardening material is one where multiple cracks can occur prior to crack localization. For material hardening to occur, the fibre dosage is typically such that the strength of the SFRC is at least 10 percent greater than the strength of the same concrete mix without fibres. The value of COD of 0.3 is sufficiently high such that matrix contribution to the strength of the composite is minimal and the tension at this point is substantially taken by the fibres alone.

D-3.3.2 *Matrix Tensile Strength*

The matrix tensile strength of the softening SFRC (f_{ct}) shall be obtained using flexural tensile testing, in accordance with good practice [C-5(2)].

When only the characteristic compressive (cylinder) strength (f'_c) has been determined, the mean and characteristic flexural tensile strength of SFRC shall be calculated in accordance with the good practice [C-5(2)].

NOTE — For strain softening SFRC, the peak strength of the composite may be approximated as the strength of the concrete mix without fibres.

D-3.3.3 *Residual Tensile Strength*

The standard characteristic residual tensile strength grades ($f'_{1.5}$) are 0.4 MPa, 0.6 MPa, 0.8 MPa, 1.2 MPa, 1.6 MPa and 2.0 MPa.

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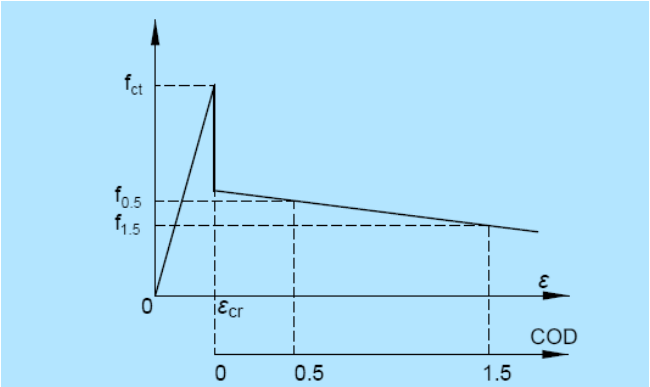
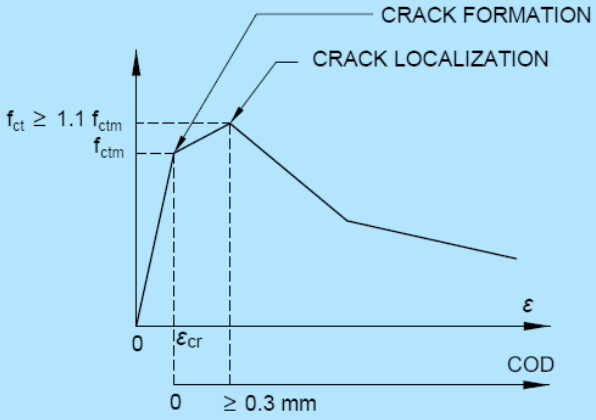
The characteristic residual tensile strengths of concrete at 28 days ($f'_{1.5}$) shall be determined statistically from tests carried out in accordance D-3.3.4 or D-3.3.5.

NOTE — The characteristic residual tensile strength depends on the fibre-matrix bond strength, which is usually a function of the compressive strength of the parent concrete, as well as the fibre content. It may be unrealistic to specify a high value of f'1.5 when using a relatively low value of f'c.



1A STRAIN SOFTENING SFRC



1B STRAIN HARDENING SFRC

FIG. 1 CLASSIFICATION OF SFRC

D-3.3.4 *Determination of Strength by Direct Testing*

The characteristic residual tensile strength ($f'_{1.5}$) shall be obtained using direct tensile tests as specified in D-3.3.7.

Alternatively, where matched direct and indirect testing has been undertaken in accordance with D-3.3.6 for similar SFRC mixtures, the characteristic residual tensile may be determined as:

$$f'_{1.5} = k_{R,4}\, f'_{R,4}$$

Where $f'_{R,4}$ is determined in accordance with D-3.3.8 and calculated statistically, and the factor $k_{R,4}$ determined from D-3.3.6.

For the purposes of this clause, similar SFRC mixtures are defined as having the same:

a) Fibre type and content;

b) Water to cementitious material ratio;

c) Maximum aggregate particle size; and

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d) Compressive strength (f'_c).

NOTES

1 In calculating the characteristic strength, the population may be treated normally distributed and the strength determined in accordance with accepted standard. A confidence level of 75 percent shall be used that 95 percent of the population exceeds the characteristic value.

2 The sample standard deviation shall not exceed 25 percent.

D-3.3.5 *Determination of Strength by Indirect Testing*

The characteristic residual tensile strength ($f'_{1.5}$) may be obtained using indirect tests as specified in **D-3.3.8** and calculated as:

$$f'1.5 = 0.4f'_{R,4} - 0.07f'_{R,2}$$

D-3.3.6 *Residual Tensile Strength-Residual Flexural Strength Relationship*

The relationship between residual tensile strength and the residual flexural strength shall be obtained by matched using the same SFRC mixture. Residual tensile strength specimens shall be prepared and tested in accordance with **D-3.3.7**. Residual flexural tensile tests shall be in accordance with the procedure as mentioned below:

The average width of the specimen and distance between the tip of the notch and the top of the specimen in the mid-span section shall be determined from two measurements to nearest 0.1 mm width and distance in the notch port of the specimen using calipers.

When the CMOD is measured, the displacement transducer shall be mounted along the longitudinal axis at mid width of test specimen such that the distance 'y' between the bottom of specimen and line of measurement is 5 mm or less as mentioned in Fig. 2.

a) The concrete mix shall be batched to ensure a uniform distribution of fibres, the SFRC shall be placed in the moulds in a manner that does not interfere with the distribution of the fibres and, the SFRC shall be compacted using lightly applied external vibration; and

b) A minimum of 12 specimens shall be tested.

The reference factor $k_{R,4}$ shall be determined as:

$$k_{R,4} = f_{1.5m}/f_{R,4m}$$

where

$f_{1.5m}$ = mean residual tensile strengths corresponding to a COD of 1.5 mm, determined in accordance with **D-3.3.7**.

$f_{R,4m}$ = mean residual flexural tensile strengths corresponding to a CMOD of 3.5 mm, determined in accordance with **D-3.3.8**.

NOTE — The residual tensile strength (as obtained from a direct tensile test) is a fundamental material property of SFRC. This clause allows the concrete producer and steel fibre manufacturer to correlate the direct tensile strength of their particular SFRC mix design with the flexural tensile strength.

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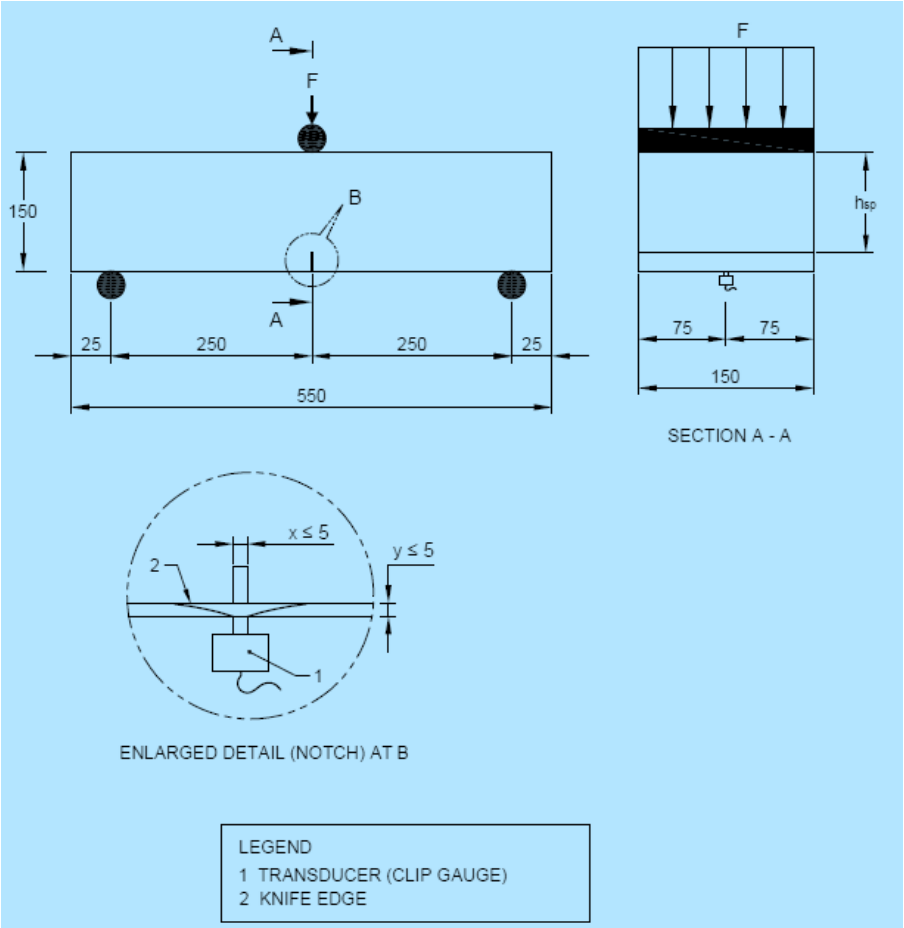

FIG. 2 ARRANGEMENT FOR RESIDUAL FLEXURAL TENSILE TEST

D-3.3.7 *Residual Tensile Strength Test*

The residual tensile strength shall be obtained using the testing arrangement shown in Fig. 3 and with the following criteria:

a) The critical section shall be where the cross sectional area is a minimum;

b) The SFRC mix shall be batched to ensure a uniform distribution of fibres, and shall be placed in the moulds in a manner that does not interfere with the distribution of the fibres. The SFRC shall be compacted using lightly applied external vibration;

c) The specimen shall be connected to the testing machine in such a manner that the machine does not apply a load to the specimen during the process of tightening of the grips and prior to testing;

d) One end of the specimen shall be connected to the testing machine through a universal joint such that no moment is applied to the end of the specimen;

e) Displacement measurements shall be taken on each of the four sides with the COD taken as the average of these measurements;

f) A minimum of twelve specimens shall be tested;

g) Tests where the failure of the specimen is outside of the testing region, or where the results are influenced by the test specimen boundaries, shall be retested;

h) The characteristic values of the tensile strength $f_{0.5}$ and $f_{1.5}$, corresponding to CODs of 0.5 mm and 1.5 mm, respectively, shall be determined statistically as the 95 percentile confidence value assuming the population is normally distributed; and

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j) The mean values of $f_{0.5m}$ and $f_{1.5m,}$ corresponding to CODs of 0.5 mm and 1.5 mm, respectively, shall be determined statistically as the 50th percentile confidence value assuming the population is normally distributed.

The stress results obtained from the test shall be multiplied by the three-dimensional orientation factor k_1, where,

$$k_1 = \frac{1}{0.94 + 0.6\ell_f/b} \leq 1$$

And ℓ_f is the length and b is taken as the average of the width and depth of the specimen taken at the critical section.

NOTE — The factor k_1 removes the influence of the boundaries on the fibre distribution and converts the results of the test to a state where the fibres can be considered to be randomly orientated in three-dimensional space.

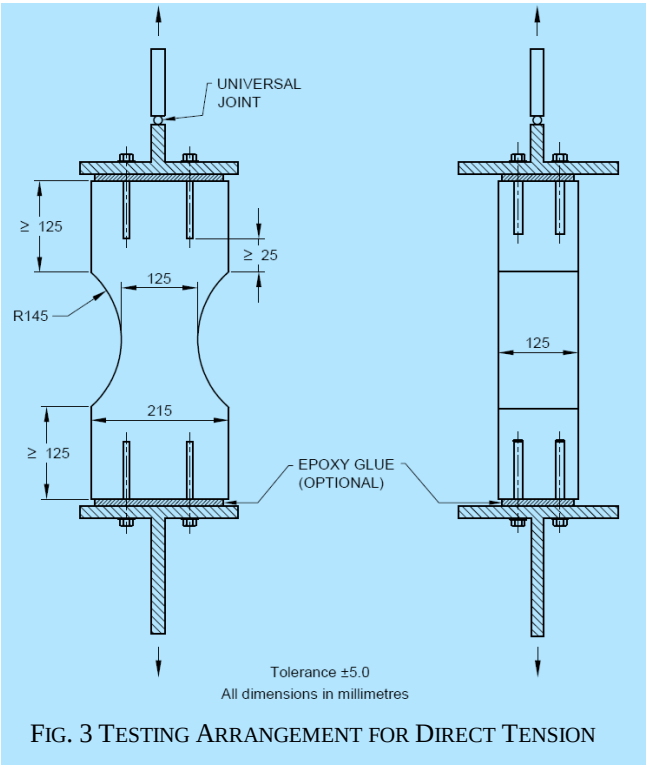

FIG. 3 TESTING ARRANGEMENT FOR DIRECT TENSION

NOTE — There is considerable discussion in the literature on whether the ends of the specimens should be fixed at each end, pinned at each end or fixed at one end and pinned at the other. The adoption of one pinned and one fixed end is a pragmatic decision between the ideal condition (fixed at each end) and limitations in manufacturing of specimens that are perfectly square at their ends for alignment within the testing machine. With both ends fixed any misalignment induces a moment in the specimen during gripping. This moment can induce a tensile stress on one side of the specimen, weakening the specimen. In some situations, the moment induced may be sufficient to crack the concrete. The inclusion of the universal joint at one end eliminates residual tensions that may result from the gripping procedure.

D-3.3.8 *Residual Flexural Tensile Strength*

The residual flexural tensile strength (F_R) shall be determined from 3 point notched bending tests on 150 mm square section prisms. The notch depth shall be 25 mm and the test conducted in accordance with **D-3.3.6**. The force F shall be plotted against the crack mouth opening displacement (CMOD), as shown in Fig. 4 and the residual flexural stress calculated as:

$$F_R = \frac{3F_{RJ}L}{2bh_{sp}^2}$$

where

F_{RJ} = load recorded at CMOD$_j$ (*see* Fig. 4).

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L = span, in mm;

b = width, in mm of the specimen; and

h_{sp} = distance, in mm between tip of the notch and top of cross section.

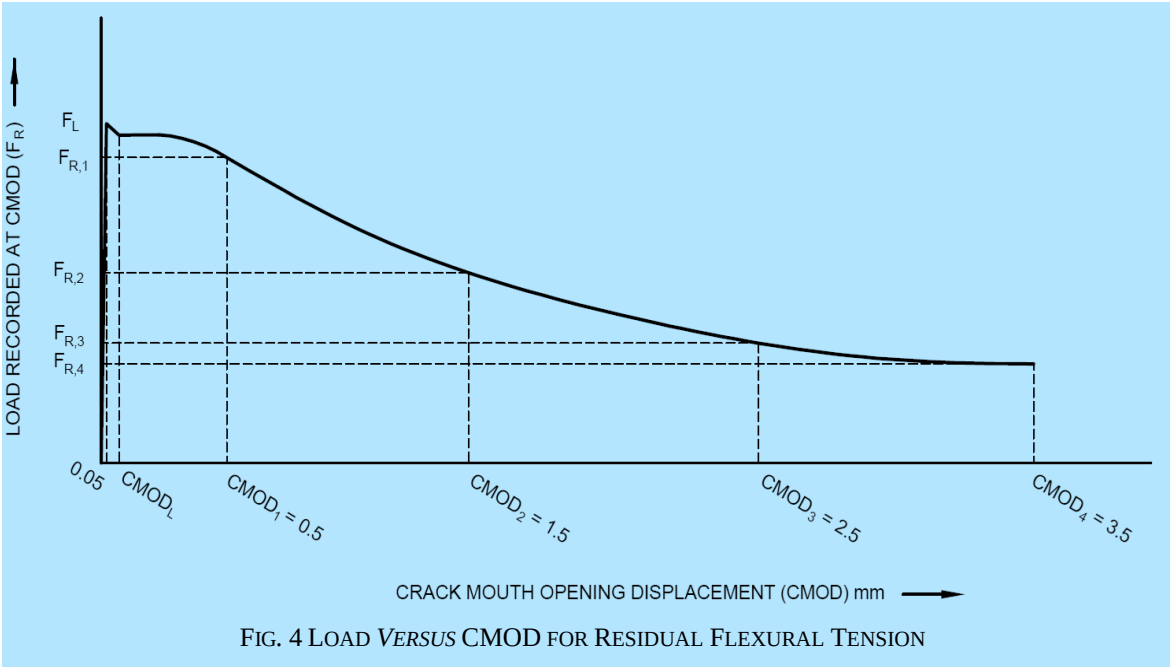

FIG. 4 LOAD *VERSUS* CMOD FOR RESIDUAL FLEXURAL TENSION

D-3.4 Modulus of Elasticity

The mean modulus of elasticity at the appropriate age (E_{cj}) shall be determined in accordance with accepted standard [C-5(38)].

NOTE — The addition of steel fibres to concrete in conventional dosages (100 kg/m^3 and less) does not change the elastic modulus of the concrete.

D-4 DESIGN OF REINFORCED SFRC MEMBERS

D-4.1 General

This clause applied to reinforced and prestressed beams subjected to any combination of shear force, bending moment and axial force. This clause does not apply when torsion acts in conjunction with shear or to non-flexural members.

D-4.2 Strength of Beams in Bending and Combined Bending and Axial Force

Calculations for strength of cross sections in bending shall incorporate equilibrium and strain-compatibility considerations and be consistent with the following assumptions:

a) Plane sections normal to the axis shall remain plane after bending;

b) Stress in the fibre reinforced concrete in that part of the cross section in tension shall be taken to be $f'_{1.5}$, where $f'_{1.5}$ is the characteristic residual tensile stress determined in accordance with **D-3.3.3**; and

c) Distribution of compressive stress shall be determined from a stress-strain relationship for the concrete.

The strength of a section in bending, or in combined bending and axial force, shall be determined using rectangular stress blocks for the concrete in compression and concrete in tension, as shown in Fig. 5.

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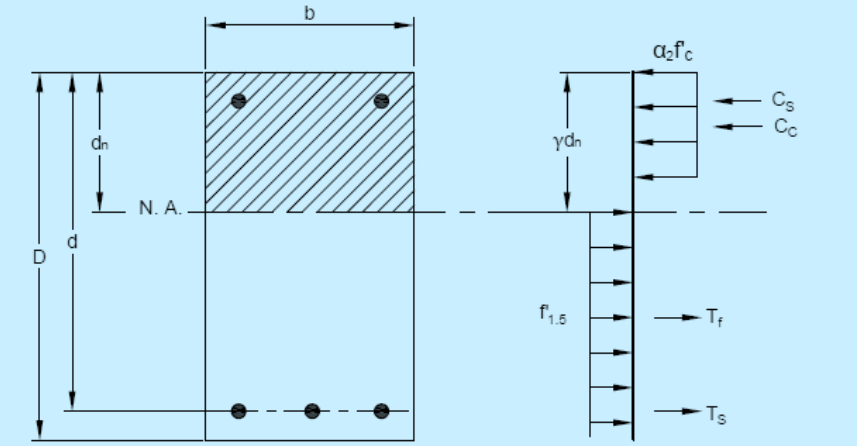
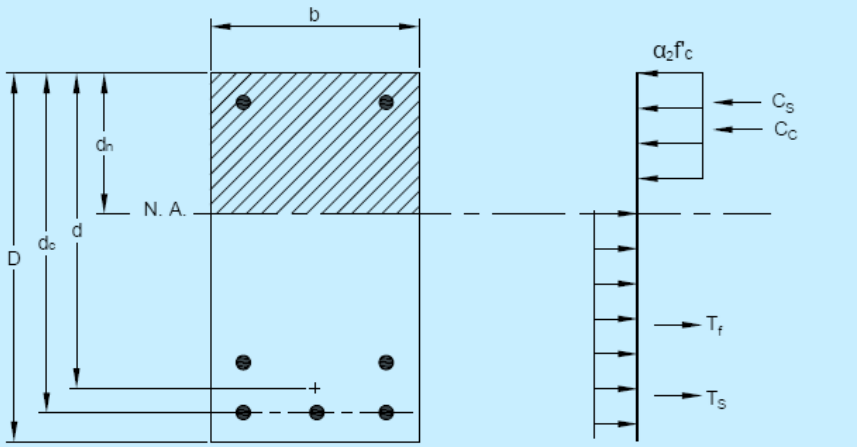


5A SINGLE TENSILE REINFORCEMENT LAYER



5B MULTIPLE TENSILE REINFORCEMENT LAYERS

FIG. 5 STRESS BLOCKS AND FORCES ON REINFORCED SFRC SECTION

D-4.3 Minimum Reinforcement Requirements for Bending

The minimum tensile reinforcement shall be not less than that determined using the principles given in this annex, excluding fibres.

D-4.4 Strength of Beams in Shear

D-4.4.1 *Design Shear Strength of a Beam*

The design shear strength of a beam shall be taken as V_u

where

$$V_u = V_{uc} + V_{uf} + V_{us}$$

where V_{uc} and V_{us} are determined from **39** of accepted standard [C-5(38)] and V_{uf} is determined from **D-4.4.2**.

Notwithstanding the above equations, the fibres component to the shear strength of a beam, V_{uf}, shall not exceed the greater of 0.3 V_u and that determined by **D-4.4.3** with V_{us} taken as zero.

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D-4.4.2 *Contribution to Shear Strength by Steel Fibres*

The contribution of the fibres to the ultimate shear strength (V_{uf}) of an SFRC beam shall be calculated from the following equation:

$$V_{uf} = 0.7 k_\theta b_v d_0 f'_{1.5}$$

where

k_θ = $\cot \theta_v \leq 1.28$

b_v = The effective width of the web (the vertical part of the beam).

d_0 = The effective depth of the beam.

$f'_{1.5}$ = characteristic residual tensile strength of concrete (at 28 days) at a Crack Opening Displacement (COD) of 0.5 mm.

θ_v = the angle between the axis of the concrete compression strut and the longitudinal axis of the member and shall be taken as not less than 38°.

D-4.4.3 *Minimum Shear Reinforcement*

The minimum contribution from the total of the transverse steel reinforcement and fibres shall satisfy the following:

$$(V_{us} + V_{uf})_{min} \geq 0.1 b_v d_0 \sqrt{f'_c} \quad and \quad \geq 0.6 b_v d_0$$

D-4.5 Design for Serviceability Limit States

D-4.5.1 *General*

When an SRFC cross sectional is uncracked, the full cross section shall be assumed to be active and both concrete and steel are assumed to be elastic in tension as well as in compression.

When an SRFC cross section is cracked, the SFRC shall be assumed to be elastic in compression, and capable of sustaining a tensile stress equal to 1.1 $f'_{1.5}$.

NOTE — Prior to cracking, linear elastic behaviour is assumed for the SFRC, reinforcement and the bonded tendons in both tension and compression. After cracking, the SFRC in tension is assumed to carry a tensile stress of 1.1 $f'_{1.5}$, while the SFRC in compression, the reinforcement and the bonded tendons are assumed to be linear elastic.

D-4.5.2 *Stress Limits*

D-4.5.2.1 *Concrete*

The maximum compressive stress in the concrete at the serviceability limit states shall not exceed $0.6 f_{cm}(t)$. Under permanent effect loading, the maximum compressive stress in the concrete shall not exceed $0.4 f_{cm}(t)$.

NOTES

1 The satisfaction of limits on the concrete tensile stress at the serviceability limit state is not necessary if the member performance is satisfactory at the ultimate limit state.

2 Compressive stresses are limited to avoid large time-dependent deformations and to ensure creep behaviour remains linear with respect to stress. Restrictions on the tensile stress in SFRC are not necessary provided the member satisfies all other requirements at the strength and serviceability limit states.

D-4.5.2.2 *Reinforcing steel*

To avoid any inelastic deformation that could lead to large, permanently open cracks, tensile stresses in the reinforcement at the serviceability limit states shall not exceed $0.8 f_{sy}$.

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D-4.5.3 *Minimum Reinforcement for Crack Control*

The minimum amount of longitudinal reinforcement required to obtain controlled crack formation shall be:

$$A_{\text{st,min}} = k_1 k_c k_p f_{\text{ct,ef}} - 1.1 f'_{1.5} \frac{A_{\text{ct}}}{f_{\text{s,max}}} \geq 0.0 \text{ (mm}^2\text{)}$$

where

$A_{\text{st,min}}$ = minimum area of reinforcement required within the tensile zone (mm^2). If $A_{\text{st,min}}$ is zero only steel fibres are necessary to control cracking;

k_1 = coefficient which allows for the effect of non-uniform self-equilibrating stresses due to non-linear shrinkage or temperature profiles through the member depth. In the absence of more detailed analysis, k_1 may be taken as 0.8;

k_c = coefficient that takes account of the nature of the stress distribution within the section immediately prior to cracking. The relevant stress distribution is that resulting from the combined effects of loading and restrained imposed deformations:

= 1.0 for pure tension ($e = M/N = 0$)

= 0.6 for pure bending

k_p = a coefficient that takes account of the level of prestress and is given by:

$$k_p = 1 - \frac{\sigma_{\text{cp}}}{k k_c f_{ct,ef}} \left(1 - k_c + 2.4 \frac{e}{D} - 6 k_c \frac{e}{D}\right)$$

$f_{\text{ct,ef}}$ = tensile strength of the concrete effective at the time when the cracks may first be expected to occur (MPa). Values of $f_{\text{ct,ef}}$ shall be obtained from $0.6\sqrt{f_{\text{cm}}}$ but not less than 3.0 MPa and f_{cm} is the mean concrete compressive strength at the time cracking is expected to occur;

A_{ct} = area of concrete within the tensile zone (mm^2). The tensile zone is that part of the cross section calculated to be in tension just before formation of the first crack;

$f_{\text{s,max}}$ = maximum stress permitted in the reinforcement immediately after formation of the crack;

e/D = ratio of the eccentricity of the prestressing force on the cross section, e, measured from the centroidal axis of the uncracked section to the overall depth of the cross section, D; and

σ_{cp} = average intensity of the effective prestress (P_e/A_g).

D-4.5.4 *Deflection Control*

D-4.5.4.1 *General*

The deflection of an SFRC member shall be calculated using the procedures outlined in **D-4.5.4.2** and **D-4.5.4.3**. Allowance shall be made for the expected load history, the expected construction procedure and any anticipated deflections resulting from deformation of forms or settlement of props.

D-4.5.4.2 *Short-term deflection*

The short-term deflections due to external loads and prestressing, which occur immediately on their application, shall be calculated using the value of E_{cj} determined in accordance with this Annex and the value of the effective second moment of area of the member (I_{ef}). The value of I_{ef} may be determined from the values of I_{ef} at nominated cross sections as follows:

a) For a simply supported span, the value at mid-span;

b) In a continuous beam:

   1) For an interior span, half the mid-span value plus one quarter of each support value; or

   2) For an end span, half the mid-span value plus half the value at the continuous support.

c) For a cantilever, the value at the support.

For the purpose of the above determinations, the value of I_{ef} at each of the cross sections nominated in items (a) to (c) is obtained from the instantaneous curvature $k_i = M_s^*/(E_{\text{cj}} I_{\text{ef}})$ calculated as the slope of the strain diagram in

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Fig. 6B and obtained by satisfying the requirements for rotational and horizontal equilibrium of the stress distribution in Fig. 6C.

NOTE — The procedure for calculating short-term deflection is consistent with the approach in this Annex. Short-term deflection of a cracked section is calculated based on the assumed stress and strain distribution shown in Fig. 6. The assumption of a uniform tensile stress in the cracked region of the SFRC is considered to be reasonable for the determination of the neutral axis location and curvature and, hence, for the determination of the effective second moment of area (I_{ef}).

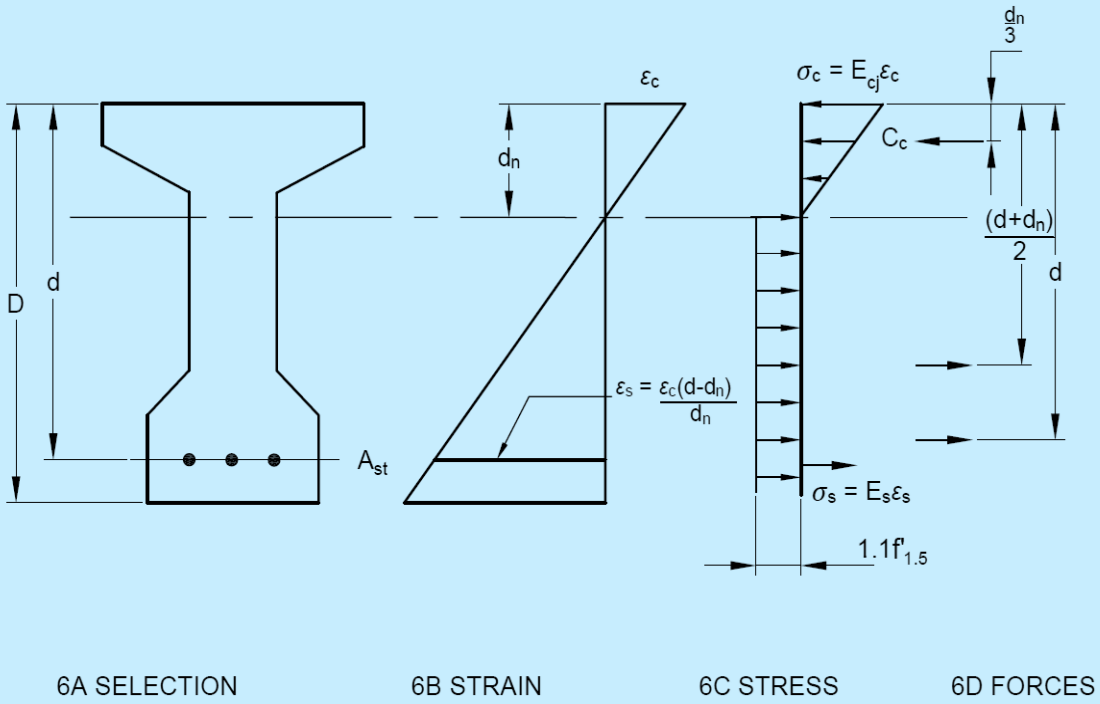

FIG. 6 STRESS AND STRAIN DISTRIBUTION ON A CRACKED SECTION SUBJECTED TO APPLIED IN-SERVICE BENDING MOMENT (M_s^*)

D-4.5.4.3 *Long-term deflection*

For reinforced and prestressed SFRC flexural members, that part of the deflection that occurs after the short term deflection shall be calculated as the sum of:

a) The shrinkage component of the long-term deflection, determined from the design shrinkage strain of concrete (ε_{cs}) and the principles of mechanics; and

b) The additional long-term creep deflections, determined from the design creep coefficient of concrete (φ_{cc}) and the principles of mechanics.

In the absence of more accurate calculations, the additional long-term deflection of a reinforced SFRC beam due to creep and shrinkage may be calculated by multiplying the short-term deflection caused by the sustained loads by a multiplier, k_{cs}, as follows:

$$k_{cs} = \left[2 - 1.2\left(\frac{A_{sc}}{A_{st}}\right)\right] \geq 0.8$$

NOTES

1 For the calculation of long-term deflection, two alternative approaches are specified. For reinforced and prestressed beams, the shrinkage and creep components of the long-term deflection may be calculated separately using the material data specified in the Annex and the principles of mechanics. The shrinkage component of the long-term deflection may be readily determined from the shrinkage-induced curvatures calculated at the critical sections along the span of the beam using the design shrinkage strain, (εcs). The creep component of long-term deflection may be obtained from the creep-induced curvatures at the critical sections under the sustained service loads using the creep coefficient for concrete, (φcc).

2 Alternatively, for reinforced concrete beams and slabs, the additional long-term deflection caused by creep and shrinkage may be approximated by multiplying the short-term or immediate deflection caused by the sustained load by the deflection multiplier $k$cs. The deflection multiplier depends on the ratio Asc/Ast, where Asc is the area of the longitudinal reinforcement in the compressive zone of

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the cracked section (that is, between the neutral axis and the extreme compressive fibre of the cracked cross section) and *A*st is the area of the tensile reinforcement.

D-5 DURABILITY

The minimum concrete grade and cover for SFRC in respective exposure classification shall be as for concrete without fibres and shall apply to the steel reinforcement only. SFRC shall not be used in exposure classification C1 or C2.

NOTES

1 Steel fibres do not require concrete cover as specified for steel reinforcement.

2 SFRC may not be suitable in some exposure classification U environments.

3 Fibres close to the surface are subject to corrosion and may result in surface staining. This will not impact durability.

D-6 FIRE

The structural performance of SFRC for fire shall be determined in accordance with this annex. The material properties for SFRC shall be as specified for concrete in this Annex except that to characteristic residual tensile stress of SFRC at elevated temperatures ($f'_{1.520}$) shall be either:

a) Taken as ($f'_{1.520}$) = $k_{\theta 1} \times f'_{1.5}$ where $k_{\theta 1}$ is given in Table 4; or

b) Determined statistically from tests.

Table 4 Elevated Temperature Coefficient for Residual Tensile Stress of SFRC

(*Clause* D-6)

Sl No.	Temperature of SFRC °C	$k_{\theta 1}$
(1)	(2)	(3)
i)	0	1.0
ii)	100	1.0
iii)	500	0.6
iv)	700	0.1
v)	1200	0.0

NOTE — Linear interpolation between the values is permitted.

D-7 PRODUCTION OF SFRC

D-7.1 Fibres

Steel fibres shall comply with accepted standard. The 'Certificate of Conformity' shall be supplied on request to the relevant authority.

NOTE — Till the availability of a separate Indian Standard specification for steel fibres, reference may be made to ASTM A820 : 2022 'Standard Specification for steel fibers for fiber-reinforced concrete'.

D-7.2 Mixing of Fibres

Fibres of the type and quantity specified shall be added in a controlled process ensuring that they are dispersed uniformly through the concrete mix. If added after the main mixing process, the concrete shall be remixed until the fibres have been completely dispersed throughout the batch.

NOTES

1 Balling of fibres should be avoided.

2 A record of fibre content should be recorded for each batch.

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D-7.3 Pre-Construction Testing of Materials

Pre-construction testing in the form of initial type tests shall be undertaken in accordance with Table 5. New initial type tests shall be undertaken when any of the following occur:

a) The concrete composition changes;

b) At least one source material of the concrete changes;

c) The results of compressive tests undertaken as per this Annex do not meet the specification; and

d) At least once in a period of 12 months.

The pre-construction tests shall be performed with the same personnel, materials and equipment that will be used during production.

Table 5 Pre-Construction Tests

(*Clause* D-7.3)

Sl No.	Material	Inspection/Test	Purpose
(1)	(2)	(3)	(4)
i)	Steel fibres	a) Check delivery note	Verify that the delivery is in accordance with the order, and is shipped from the correct source
		b) Check conformity label	Verify that the fibres have the correct conformity label which matches the corresponding certificate of conformity
ii)	Steel fibre content in the fresh concrete	Testing according to **D-7.5** on the basis of 9 samples	Conformity with the target dosage. Verify homogeneous distribution of the steel fibres in the mix
iii)	Steel fibre concrete performance	Check limit of proportionality, and post-crack flexural strength in accordance with **D-3.3.6** on a minimum of 12 beams	Verify, if the performance is in accordance to the specification. The performance level serves as the reference for continuous production control

NOTE — The strength of the population may be treated as normally distributed and the characteristic strength determined in accordance with this annex. A confidence level of 75 percent shall be used such that 95 percent of the population exceeds the characteristic value. For a sample of 12 specimens, the characteristic strength may be calculated from the mean strength using characteristic strength = mean strength × (1 – 1.84 × COV). The coefficient of variation (COV as a percentage) shall not exceed 25 percent.

D-7.4 Factory Production Control

Factory production control is accordance with Table 6 shall be undertaken to establish a production process for the steel fibre reinforced concrete, and shall include the following:

a) Checking the correct concrete constituents are being used in production;

b) Steel fibres are checked against specified requirements;

c) Compressive tests undertaken in accordance with good practice [C-5 (40)]; and

d) Fibre content and distribution.

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Table 6 Routine Production Control

(*Clause* D-7.4)

Sl No.	Subject	Inspection/Test	Purpose	Frequency
(1)	(2)	(3)	(4)	(5)
i)	*Equipment inspection*			
	a) Automatic dosing equipment for steel fibres	1) Visual inspection	Assure correct functioning of dosing device	Once per production day
		2) Control of accuracy	Avoid improper fibre dosage	On installation periodically in case of doubt
ii)	*Materials inspection*			
	a) Steel fibres	1) Check delivery note	Verify that the delivery is in accordance with the order, and is shipped from the correct source	Each delivery
		2) Check conformity requirements	Verify that the fibres matches the corresponding Certificate of Conformity	Each delivery
		3) Visual control, measure fibre dimensions	Compare the fibre geometry with the fibres used for ITT	Each delivery
iii)	*Production process inspection*			
	a) Fibres content-record	1) Record the quantity added	Check the content	Every batch
	b) Fibre content in the fresh concrete	2) Testing according to **D-7.5**	Conformity with the target dosage Verify homogenous distribution of the steel fibres in the mix	Beginning of each day and every/ 50 m^3 (manual dosing) and/150 m^3 (auto dosing)
	c) Concrete mix	3) Visual Check	Correct mixing with correct fibre type and even fibre distribution without balling	Daily
iv)	*Finished product inspection*			
	a) Steel fibre concrete performance	1) Check limit of proportionality, and post-crack flexural strength in accordance with **D-3.3.6**	Check performance level of the specification	2 beams every other day of production

D-7.5 Determining the Steel Fibre Content

Steel fibre content shall be measured from samples taken from the production concrete, by volume of concrete.

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In addition:

a) A sample shall be taken from the batch of concrete at unloading from the first third, middle third and final third of the batch;

b) Each sample shall be a minimum of 10 litre;

c) The sample container shall be filled in one continuous pour and where possible directly from the discharge chute; and

d) Wash out or magnetic separation only shall be used.

The conformity of the correct steel fibre dosage is proven if the criteria in Table 7 are met.

Table 7 Criteria of Acceptance for Steel Fibre Dosage

(*Clause* D-7.5)

Sl No.	Test Control	Test Control	Criteria
(1)	(2)	(3)	(4)
i)	Each sample	Each partial test	≥ 0.80 of the specified target dosage
ii)	Average of 3 samples from the batch	Each test	≥ 0.85 of the specified target dosage
iii)	Continuous control: average of > 3 tests	Continuous control average of > 3 tests	≥ 0.90 of the specified target dosage

D-7.6 Sampling, Testing and Assessment for Compliance of Hardened SFRC

When concrete is specified by parameters other than strength grade, the method of production control and, if required, project control shall be specified together with the relevant compliance criteria.

Methods of control and assessment shall provide a reliable operating characteristic curve so that:

a) Concrete with a proportion defective of 0.05 has a probability of acceptance of at least 50 percent; and

b) Concrete with a proportion defective of 0.30 has a probability of rejection of at least 98 percent.

ANNEX E

(*Foreword and Clause* 5)

CRITERIA FOR STRUCTURAL SAFETY OF TALL CONCRETE BUILDINGS

E-1 SCOPE

E-1.1 This Annex covers the following design aspects of reinforced concrete (RC) buildings of height greater than 50 m, but less than or equal to 250 m. Generally, these requirements are based on prescriptive approach and covers the following design aspects of tall buildings:

a) Selection of appropriate structural system;

b) Geometric proportioning of the building;

c) Integrity of structural system;

d) Resistance to wind and earthquake effects; and

e) Other special considerations related to tall buildings.

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E-1.2 This Annex is not applicable for tall buildings located in the near-field of seismogenic faults. For the purposes of this standard, near-field is taken as 10 km (shortest distance) from a seismogenic fault.

E-1.3 This Annex is applicable only for buildings that house 20 000 or fewer persons.

E-1.4 This Annex may also be used for design of buildings of height equal to or less than 50 m. The provisions of this Annex will add value to the design of such buildings. However, this Annex is not applicable for design of buildings of height more than 250 m.

E-1.5 This Annex addresses the following typical structural systems of tall concrete buildings:

a) Structural wall systems;

b) Moment frame systems;

c) Moment frame — Structural wall systems;

d) Structural wall — Flat slab floor systems with perimeter moment frame;

e) Structural wall — Framed tube systems;

f) Framed-tube system;

g) Tube-in-tube system;

h) Multiple tube system;

j) Hybrid system; and

k) Any of the above with additional framing systems, for example, outrigger trusses, belt trusses and braced frames.

E-1.6 This Annex shall be read along with all Indian Standards relevant to design and construction of buildings and structures. In case of conflict, clauses given in this Annex shall be applicable.

E-1.7 For buildings that do not conform to the prescriptive requirements of this Annex, a more rigorous procedure is necessary for design and review.

E-2 -

NOTE — (This clause is intentionally left blank.)

E-3 TERMINOLOGY

For the purpose of this Annex, the following definitions shall apply.

E-3.1 Building Height — The difference in levels between the base level of a building, at which inertia forces generated in the structure are transferred to the foundation, which then transfers these forces to the ground, and the highest level of a building, which is the roof of the topmost level which is structurally integral and continuous with the floor below (*see also* **E-4**).

E-3.2 Connecting Structure — A structure that links two or more towers, except at the podium levels.

NOTE — There are two distinctive types of connecting structures with each one requiring a different structural treatment, namely, (a) the connecting structure having the role of transferring gravity loads only and with no participation in the role of resisting the lateral loads on either of the towers; and (b) the connecting structure participating in the lateral load resisting structural system, where the two towers act together when one or both towers is subjected to lateral loads due to the interaction provided by the connecting tower.

E-3.3 Core — An assembly of structural walls coupled with link elements and offering lateral stiffness in two orthogonal directions.

E-3.4 Coupled Structural Wall Building — A building comprising of structural walls linked by in-plane beam elements, wherein the vertical and lateral loads are resisted by these structural walls, through axial load, in-plane bending moment and shear force, and the coupling action created by push-pull in the wall elements by framing action provided by link beams.

E-3.5 Frame Building — A building comprising beams and columns, wherein the vertical and lateral loads are resisted by these elements through moment-frame action.

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E-3.6 Gravity Columns — Vertical members that are intended to carry primarily the vertical loads arising out of the dead and imposed loads on the building, and do not participate in the lateral load resisting system. Such columns shall be capable primarily of carrying the gravity loads and additional induced loads while complying with the relative lateral deformation effects imposed on them during lateral earthquake shaking.

E-3.7 Key Elements — The element such that its failure would cause the collapse of more than a limited area close to it. The limited area may be taken equal to 70 m^2 or 15 percent of the area of the storey, whichever is lesser. If key elements exist, it is preferable to modify the layout so that the key element is avoided.

E-3.8 Linking Beams (Also called coupling beams) — Horizontal members spanning two (vertical) structural walls.

E-3.9 Mixed Building or Hybrid Building — A building in which select primary elements resisting vertical and/or lateral loads comprise more than one material; composite elements comprising structural steel and reinforced concrete are common examples.

E-3.10 Multi-Tower Structures Linked by Podium — Two or more towers above a common podium structure that are not separated by a movement joint at the podium level.

E-3.11 Natural Period — The time taken (in seconds) by the structure to complete one cycle of oscillation in its natural mode of oscillation.

E-3.12 Structural Systems and Sub-Systems

E-3.12.1 *Structural Wall System* — A structural system comprising of inter-connected structural walls, wherein the vertical and lateral loads are resisted by the walls through axial load, in-plane bending moment and shear force. The wall elements are configured by a combination of structural wall elements connected integrally to a floor diaphragm. The wall elements form the primary lateral load resisting structural system for the building, and resist the loads imposed on them through axial, shear and flexural actions, and through coupling actions offered by the connecting link elements if provided.

E-3.12.2 *Moment Frame System* — A structural system comprising (beam-column) frames and resisting the vertical and lateral loads.

E-3.12.3 *Moment Frame-Structural Wall System* — A structural system comprising (beam-column) frames and structural walls resisting the vertical and lateral loads. The relative share of the lateral load resisted between these systems is dependent on their relative lateral stiffnesses.

E-3.12.4 *Structural Wall System with Flat Slab Floor System* — A structural system comprising structural walls, a beam-less floor system, and columns resisting the vertical and lateral loads. The relative share of the lateral load resisted between these systems is dependent on their relative lateral stiffnesses.

E-3.12.5 *Core and Outrigger Structural System* — A structural system comprising wall element(s) and perimeter columns, resisting the vertical and lateral loads. Essentially, the perimeter columns are for resisting gravity loads only. The core structure is connected to select perimeter column element(s) (often termed outrigger columns) by beam elements, known as outriggers, at discrete floor locations along the height of the building. This type of structure is an extension of the core structure, to enhance the lateral stiffness for taller structures, which mobilizes the perimeter columns, and offers increased leverage for push pull-action through the framing action offered by the deep beam(s) connecting the core to the outrigger columns. The global lateral stiffness is sensitive to: flexural stiffness of the core element, the flexural stiffness of the outrigger element(s) and the axial stiffness of the outrigger column(s).

E-3.12.6 *Core, Outrigger and Belt Wall System* — A structural system, which is an extension to the core and outrigger structure to enhance the lateral stiffness further, where the outrigger column(s) is linked to the adjacent columns by deep beam elements (often known as belt truss), typically at the same level as the outrigger elements. The sharing of loads between multiple columns has the dual function of enhancing the axial stiffness and mobilizing greater number of gravity columns to counteract the induced tension loads generated by the overall lateral loads.

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E-3.12.7 *Framed-Tube System* — A structural system comprising closely spaced columns and deep beams in the perimeter frame for efficient tube action. The internal vertical elements, comprising core or columns are primarily utilized to resist gravity loads only. Effective utilization of the perimeter of the building maximizes the overall stiffness for a given building plan shape; this system is effective for very tall buildings.

E-3.12.8 *Tube-in-Tube System* — A structural system, which is an extension of the tube structure, where there is an internal tube, often a core element, supplementing the external perimeter described as the tube structure above, to enhance the overall lateral global stiffness. Just as the tube structure, typically, even this system is suited for very tall buildings.

E-3.12.9 *Multiple-Tube System* — A structural system, which is an extension of the tube structure and/or the tube-in-tube structure, where the architectural plan of the tower facilitates multiple tubes to be connected together, to enhance the lateral stiffness of the structure. Just as the tube structure and the tube-in-tube structure, typically, this suited is reserved for very tall buildings.

E-3.13 Super Tall Building — A building of height greater than 250 m.

E-3.14 Tall Building — A building of height greater than 50 m, but less than or equal to 250 m.

E-3.15 Transfer Structure — A structure, comprising of horizontal deep beams, trusses or thick slabs that transfers load actions and supports vertical elements above to vertical elements below that are not aligned with each other, through flexural and shear actions. Alternatively, it can be a trussed structure that fulfils the task through axial actions in the truss members.

E-4 SYMBOLS

For the purpose of this Annex, the following letter symbols shall have the meaning indicated against each; where other symbols are used, they are explained at appropriate places. All dimensions are in metres (m), loads in Newton (N), stresses in mega Pascals (MPa) and time in seconds (s), unless otherwise specified.

a — Major axis of elliptic plan shape of the building

A_{cr} — Cracked area of section

A_g — Gross area of section

b — Minor axis of elliptic plan shape of the building

h_i — Inter-storey height of i^{th} floor in the building

l_s — Clear span of coupling beam

B — Smaller plan dimension of the building at its base

B_t — Width of tower above podium level

D — Dimension of the building in plan along the considered direction of earthquake shaking

d — Overall depth of coupling beam

H — Building height from its base to roof level:

a) excluding the height of basement storeys, if basement walls are connected with the ground floor slab or basement walls are fitted between the building columns, but;

b) including the height of basement storeys, if basement walls are not connected with the ground floor slab and basement walls are not fitted between the building columns

H_t — Height of tower above podium level

h_w — Unsupported height of structural wall

I_{eff} — Effective or cracked moment of inertia of section

I_g — Gross moment of inertia of section

L — Larger plan dimension of the building at its base

L_t — Length of the tower above podium level

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M_u — Factored bending moment at a cross-section in a vertical member of the building

P_u — Factored axial load at a cross-section in the vertical member of the building

P_{uz} — Factored pure axial load capacity (at $M_u = 0$) of a vertical member of the building

T_a — Approximate fundamental translational lateral natural period

Δ_{max} — Maximum relative lateral displacement within the storey

E-5 GENERAL REQUIREMENTS

E-5.1 Elevation

E-5.1.1 *Height Limit for Structural Systems*

The maximum building height (in m) shall not exceed values given in Table 8 for buildings with different structural systems.

Table 8 Maximum values of Height *H* above Top of Base Level of Buildings with Different Structural Systems, in Metre

(*Clause* E-5.1.1)

Sl No.	Seismic Zones	Structural System				
		Moment Frame	Structural Wall Well Distributed[1)]	Structural Wall + Moment Frame	Structural Wall + Perimeter Frame	Structural Wall + Framed Tube*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	V	NA	120	150	150	180
ii)	IV	NA	150	200	200	225
Iii)	III	60	200	225	225	250
iv)	II	80	250	250	250	250

[1)] Structural walls are considered to be well-distributed when structural walls that are outside of the core are capable of carrying at least 25 percent of the lateral loads.

* This includes systems covered under **E-1.5 (g)**, **E-1.5 (h)** and **E-1.5 (k)**.

E-5.1.2 *Slenderness Ratio*

The maximum values of the ratio of height to minimum base width *B* shall not exceed values given in Table 9.

Table 9 Maximum Slenderness Ratio (H_t/B_t)

(*Clause* E-5.1.2)

Sl No.	Seismic Zones	Structural System				
		Moment Frame	Structural Wall Well Distributed	Structural Wall + Moment Frame	Structural Wall + Perimeter Frame	Structural Wall + Framed Tube*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	V	NA	8	8	9	9

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Table 9 (*Concluded*)

(*Clause* E-5.1.2)

Sl No.	Seismic Zones	Structural System				
		Moment Frame	Structural Wall Well Distributed	Structural Wall + Moment Frame	Structural Wall + Perimeter Frame	Structural Wall + Framed Tube*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
ii)	IV	NA	8	8	9	9
iii)	III	4	8	8	9	10
iv)	II	5	9	9	10	10

* This includes systems covered under **E-1.5(g)**, **E-1.5(h)** and **E-1.5(k)**.

E-5.1.3 *Aerodynamic Effects*

Elevation profile, façade features of the building, and plan shape of the building shall be such as to attract minimum wind drag effects. Effects of features such as sharp corners, projected balconies, etc, shall be considered in design.

E-5.2 Plan

E-5.2.1 *Plan Geometry*

E-5.2.1.1 The plan shall preferably be rectangular (including square) or elliptical (including circular). In buildings with said plan geometries, structural members participate efficiently in resisting lateral loads without causing additional effects arising out of re-entrant corners and others.

E-5.2.2 *Plan Aspect Ratio*

The maximum plan aspect ratio (L_t/B_t) of the overall building shall not exceed 5.0. In case of an L-shaped building, L_t and B_t shall refer to the respective length and width of each leg of the building.

E-5.3 Storey Stiffness and Strength

Parameters influencing stiffness and strength of the building should be so proportioned, that the following are maintained:

a) Lateral stiffness of any storey shall not be less than 70 percent of that of the storey above; and

b) Lateral strength of any storey shall not be less than 90 percent that of the storey above.

E-5.4 Deformations

E-5.4.1 *Lateral Drift*

When design lateral forces are applied on the building, the maximum inter-storey lateral drift ratio ($\Delta max/h_i$) under serviceability loads (including wind load with return period of 20 years), which is estimated based on the sectional properties for serviceability loads mentioned in **E-7.2**, shall be limited to 1/500. Total drift at the topmost usable floor shall be limited to $H/500$. For a single storey the drift limit may be relaxed to $h_i/400$.

E-5.5 Natural Modes of Vibration

E-5.5.1 The natural period of fundamental torsional mode of vibration shall not exceed 0.9 times the smaller of the natural periods of the fundamental translational modes of vibration in each of the orthogonal directions in plan.

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E-5.5.2 The fundamental translational lateral natural period in any of the two horizontal plan directions, shall not exceed 8 s, considering sectional properties as per **E-7.2(c)** corresponding to serviceability loads.

E-5.6 Floor Systems

E-5.6.1 *Material*

All floor slabs shall be cast in-situ. For precast floor systems a minimum structural topping of 75 mm concrete with reinforcing mesh shall be used in Seismic Zones III and above, but can be reduced to 50 mm in Seismic Zone II.

E-5.6.2 *Openings*

E-5.6.2.1 Openings in floor diaphragm shall not be permitted along any floor diaphragm edge, unless perimeter members are shown to have stability and adequate strength.

E-5.6.2.2 The maximum area of openings in any floor diaphragm shall not exceed 30 percent of the plan area of diaphragm. Transfer of lateral forces from diaphragm to lateral load resisting vertical elements shall be ensured using collector elements, if required.

E-5.6.2.3 At any storey, the minimum width of floor slab along any section after deduction of openings shall not be less than 5 m, if there is no perimeter beam. Further, the cumulative width of the slab at any location shall preferably not be less than 50 percent of the floor width.

E-5.6.3 *Natural Frequency of Floor System*

The natural vertical vibration frequency of any floor system shall not be less than 3 Hz without demonstration of acceptability using rational procedures.

E-5.7 Materials

E-5.7.1 *Concrete*

E-5.7.1.1 The minimum grade of concrete shall be M 30.

E-5.7.1.2 The maximum grade of concrete shall be M 70. When higher grades are required, the designer shall ensure through experimentation that such concretes shall have at least a minimum crushing strain in compression of 0.002.

E-5.7.2 *Reinforcing Steel*

E-5.7.2.1 Reinforcing seel shall conform to provisions of good practice [C-5(5)].

E-5.7.2.3 Lapping of longitudinal bars shall be avoided in RC columns and walls that form a part of the lateral load resisting system; when diameter of bars is 20 mm or higher; mechanical couplers as per accepted standard [C-5(33)] shall be used to extend bars.

E-5.8 Progressive Collapse

Following are general guidelines to avoid progressive collapse of structure.

E-5.8.1 Possibilities of progressive collapse shall be precluded by:

a) Selecting structural systems that are appropriate for ensuring structural integrity;

b) Adopting rigorous structural investigations that verify acceptable structural behaviour, even when select critical members do not play their intended role; and

c) Providing adequate redundancy and integrity to the structure.

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E-5.8.2 *Requirements of Key Elements*

E-5.8.2.1 Key elements are members, joints or other components, whose failure would result in a disproportionate deterioration of the building and whose presence is vital to ensure ductile behaviour of the building. Vertical and lateral resistance of key elements shall be improved in many ways, including by the use of higher partial safety factors for loads and materials, to ensure that they do not yield before the designated ductile elements.

E-5.8.2.2 Elements adjoining key elements and capable of providing an alternative load transfer path, shall be suitably designed and detailed.

E-6 LOADS AND LOAD COMBINATIONS

E-6.1 The loads and load combinations specified in Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS, the good practice [C-5(2)] and good practice [C-5(5)]shall be applicable for tall buildings also. In addition, requirements given in subsequent clauses shall be applicable.

E-6.2 Wind Effects

E-6.2.1 For buildings (a) with height greater than 150 m, or (b) with complexities in plan or elevation geometry, or (c) sited on complex topography with group effect or interference effect (existing and future potential),or (d) whose natural period is greater than 5 s, wind effects shall be determined by site-specific wind tunnel studies.

E-6.2.2 *Site-Specific Wind Tunnel Studies*

E-6.2.2.1 When wind tunnels studies result in higher storey shears and overturning moments than those calculated at based on **4** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS, the results of wind tunnel studies may be used in design.

E-6.2.2.2 When wind tunnel studies result in lower story shears and moments than those calculated based on **4** of Part C/Section 1 of this NBCS,

a) The minimum design wind base shear shall be at least 70 percent of that derived based on **4** of Part C/ Section 1 of this NBCS; and

b) The relative distribution of storey shears shall be as obtained from wind tunnel studies.

E-6.2.2.3 When wind tunnel studies indicate torsional motion, structural system of the building should be modified suitably to mitigate the torsional effects, so as to bring the torsional velocity below 0.003 rad/s for 10 year return period.

E-6.2.2.4 The damping ratio considered shall not be greater than 2 percent of critical for concrete buildings.

E-6.2.3 *Lateral Acceleration*

From serviceability considerations, under standard wind loads with return period of 10 years, the maximum structural peak combined horizontal acceleration a_{max} in the building for along and across wind actions at any floor level shall not exceed values given in Table 10, without or with the use of wind dampers in the building.

Table 10 Permissible Peak Combined Acceleration

(*Clause* E-6.2.3)

Sl No.	Building Use	Maximum Peak Combined Acceleration, a_{max} m/s^2
(1)	(2)	(3)
i)	Residential	0.18
ii)	Mercantile	0.25

E-6.3 Seismic Effects

E-6.3.1 Vertical shaking shall be considered simultaneously with horizontal shaking for tall buildings in Seismic Zone V.

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E-6.3.2 For buildings in Seismic Zone V, deterministic site-specific design spectra may preferably be estimated and used in design. When site-specific investigations result in higher hazard estimation, site-specific investigation results shall be used.

E-6.3.3 Design base shear coefficient of a building under design lateral forces, shall not be taken less than that given in Table 11.

Table 11 Minimum Design Base Shear Coefficient (Percent of Seismic Weight)

(*Clause* E-6.3.3)

Sl No.	Building Height, *H*	Seismic Zone			
		II	III	IV	V
(1)	(2)	(3)	(4)	(5)	(6)
i)	$H \leq 120$ m	0.7	1.1	1.6	2.4
ii)	$H \geq 200$ m	0.5	0.75	1.25	1.75

NOTE — For buildings of intermediate height in the range 120 m to 200 m, linear interpolation shall be used.

E-6.3.4 *Approximate Fundamental Natural Period*

For buildings of height 50 m and more, the fundamental period T (in s) for a structure shall be determined by accounting for all structural properties and inherent stiffness of the building through rigorously validated structural analysis procedures. The fundamental period shall however not exceed the value obtained from the approximate fundamental translational natural period T_a (in s) of oscillation, estimated by following expression:

$$T_a = 0.0644H^{0.9}$$ for concrete moment resisting frame systems

and

$$T_a = 0.0672H^{0.75}$$ for all other concrete structural systems

E-7 STRUCTURAL ANALYSIS

E-7.1 Software

Structural analysis shall be carried out using standard 3-D computer model using well-established structural analysis software.

E-7.2 Considerations

Computer modelling shall consider the following:

a) Rigid end offsets of linear members in the joint region, when centerline modelling is adopted;

b) Floor diaphragm flexibility, as applicable;

c) Cracked cross sectional area properties as per Table 12; and

d) The load combination for the initial analysis considering *P-Δ* effects shall be taken as: 1.2 DL + 0.5 IL ± 1.5 EL/WL.

E-7.3 Modelling

E-7.3.1 Modelling of buildings shall follow a simple approach, which reflects the distribution of mass and stiffness properties to properly account for all significant inertial forces under seismic actions and deformation shapes.

E-7.3.2 Analytical model of a building shall reflect the true behaviour of its members as well of the whole structure. One can adopt lumped modelling that is frame element modelling, distributed modelling that is finite element modelling or a combination of the two.

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Table 12 Cracked Section Properties of RC Members

(*Clauses* E-7.2 *and* E-7.3.6)

Sl No.	Structural Element	Serviceability Design		Strength Design	
		Cross-Sectional Area	Moment of Inertia	Cross-Sectional Area	Moment of Inertia
(1)	(2)	(3)	(4)	(5)	(6)
i)	Slabs	1.0 A_g	0.35I_g	1.00 A_g	0.25 I_g
ii)	Beams	1.0 A_g	0.7 I_g	1.00 A_g	0.35 I_g
iii)	Columns	1.0 A_g	0.9 I_g	1.00 A_g	0.70 I_g
iv)	Walls	1.0 A_g	0.9 I_g	1.00 A_g	0.70 I_g

E-7.3.3 In-plane stiffness of floor slabs shall be modelled, unless it is demonstrated that it is extremely stiff and sufficiently strong to remain elastic under seismic actions. *See* **5** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS to identify when a floor slab may be considered to be extremely stiff in its own plane.

E-7.3.4 In moment frame buildings, when buildings with unreinforced masonry infill panels contribute to storey lateral stiffness, their effect shall be modelled as equivalent diagonal struts as per relevant provisions of **5** of Part C/Section 1 of this NBCS.

E-7.3.5 The analytical model for performing dynamic analysis of buildings with irregular configuration shall adequately represent irregularities in the configuration of the building.

E-7.3.6 Cracked sectional properties shall be used when representing concrete elements as per Table 12.

E-7.3.7 In reinforced concrete buildings, lateral deflections resulting from unfactored lateral loads shall be estimated using section properties intended for use with unfactored lateral loads, and lateral deflections resulting from factored lateral loads using section properties intended for use with factored lateral loads.

E-7.3.8 Buildings may be considered to be fixed at their bases for determining lateral effects on buildings. When modelling flexibility of foundations, reference shall be made to **E-8** and **E-9**.

When foundation flexibility is included in linear analysis, load-deformation characteristics of foundation-soil system shall be accounted for by equivalent linear stiffness, using soil properties that are consistent with soil strain levels associated with the design forces. A 50 percent increase and decrease in stiffness shall be incorporated in dynamic analysis, unless smaller variation can be justified; the largest value of response shall be used in the raft design.

E-7.3.9 Second order deformation effects (P-Δ effects) shall be considered.

E-7.3.10 The flexibility of the building shall be such that the inter-storey drift stability coefficient θ satisfies

$$\theta = \frac{P_i \Delta_i}{V_i h_{i-1} R} \leq 0.2$$

where

P_i = total design vertical load at level i;

Δ_i = design storey drift at level i;

V_i = design shear force at level i;

h_{i-1} = storey height below level i; and

R = response reduction factor.

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E-7.3.11 Stiffness of frames with flat slabs as horizontal members (that is, slab-column frames) shall be ignored in lateral load resistance, in all Seismic Zones. These shall be designed as gravity elements for deformation compatibility.

E-7.3.12 The model used in structural analysis of solid, coupled, perforated or punched structural walls shall represent stiffness, strength and deformation capacities of structural wall, structural wall segments and coupling beams or spandrel connections between structural walls. Stiffness of coupling beams and spandrel connections shall capture aspect ratio of these coupling beam and spandrel connections, extent of cracking anticipated, and reinforcement provided in them.

E-7.3.13 Effects shall be considered of construction sequence in buildings taller than 150 m.

E-7.3.14 Tower(s) connected by a podium shall be modelled separately and integrally.

E-7.4 Building Movements

For buildings taller than 150 m, and for buildings taller than 100 m with mass asymmetry, analysis shall be carried out for both vertical and horizontal long-term building movements.

E-7.4.1 Measures shall be taken in concrete and composite buildings to minimize adverse effects of shrinkage, creep, temperature variation and foundation settlement during the design life of the building (not less than 30 years).

E-7.4.2 Non-structural elements, such as curtain walls, cladding, partitions and finishes and service installations (for example, elevators, vertical pipes, ducts and cables), shall be required to withstand long-term movements of the building and associated differential effects.

E-7.4.3 Details of connections of non-structural elements with the structural elements of the building shall be planned, such that their relative movements are allowed without causing distress to both structural and non-structural elements.

E-7.4.4 Appropriate vertical compensation and sway compensation shall be accounted for during construction to minimize long term building movements for concrete and composite structures.

E-7.4.5 In gravity load analysis, internal forces shall be considered, which are developed due to differential vertical movement of vertical structural elements, due to shrinkage, creep, temperature, foundation settlement and construction compensation. In any case the total shrinkage strain of concrete shall not exceed 0.04 percent.

E-7.4.6 To calculate the effect of creep and shrinkage, selected strain prediction models of concrete shall be based on established principles of mechanics elaborated in specialist literature.

E-8 STRUCTURAL DESIGN

E-8.1 General Requirements

E-8.1.1 *Method of Design*

Limit state design method (as given in [C-5(2)]) shall be used in the design of RC members.

E-8.1.2 *Staircase*

Staircases built integrally with the structural system of the building and not confined by structural walls, shall be included in the three-dimensional structural model, and its elements designed as per forces induced in them under various load combinations. Alternately a sliding joint may be provided.

E-8.1.3 *Multiple Tall Buildings connected with a Common Podium*

E-8.1.3.1 This section deals with requirements for the following tall buildings with podium:

a) Tall building with single tower and podium (*see* Fig. 7A); and

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b) Tall building with multiple towers and common podium (*see* Fig. 7B).

E-8.1.3.2 *Modelling*

E-8.1.3.2.1 *Sensitivity analyses*

a) As part of collapse prevention evaluation, two sets of backstay sensitivity analyses shall be carried out using upper-bound and lower-bound cracked section properties of floor diaphragms and the stiffness parameters for those diaphragms and perimeter walls of podium and below the level of the backstay are given in Table 13. These analyses shall be in addition to those required to be carried out using other cracked section properties described in **E-7.2(c)**.

Table 13 Stiffness Parameters

[*Clauses* E-8.1.3.2.1(a) *and* E-8.1.3.3.3(d)(2)]

Sl No.	Stiffness Parameter	Values to be Adopted	
		Upper-Bound	Lower-Bound
(1)	(2)	(3)	(4)
i)	I_{eff} / I_g	0.5	0.15
ii)	A_{cr} / A_g	0.5	0.15

b) Besides that of the floor diaphragms, flexibility of following structural elements in the structural analysis with appropriate modification to their stiffness:

1) Perimeter walls and their foundation supports; and

2) Foundation supports under the tower lateral load resisting system.

E-8.1.3.3 *Buildings with multiple towers*

E-8.1.3.3.1 *Backstay*

Backstay transfers the forces from lateral load-resisting elements in the tower to additional structural elements provided within the podium and the basement, typically through one or more floor diaphragms. Lateral load resistance in the podium levels with assured force transfer path through floor diaphragms at these levels, helps the tall building to resist lateral overturning forces. This component of overturning resistance, referred as the backstay effect (also called shear reversal), is critical because shear force changes direction within the podium levels, and the same lateral load-resisting element helps resist the changing shear force.

The following shall be considered:

a) In estimating backstay effects, two lateral load paths shall be considered (*see* Fig. 8), namely:

1) *Direct load path* — where overturning resistance is provided by the tower core elements and foundation directly beneath the tower; and

2) *Backstay load path* — where overturning resistance provided by in-plane forces in the backstay elements (lower floor diaphragm and perimeter walls).

In some tall buildings, backstay effects may not be considered. These include the following configurations:

i) Buildings without below grade levels or buildings without significantly increased lateral load resisting systems at the base;

ii) Buildings that extend below grade, but have structural separations between the superstructure and the podium portions of the building, and that accommodate lateral load deformations without transfer of lateral forces from superstructure portion to podium portion; and

iii) Buildings with perimeter basement walls, where the walls are located directly below the lateral load resisting elements of the superstructure above. Here, there may be a marked change in

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lateral strength and stiffness at this level, but lateral forces will not be transferred through the floor diaphragms.

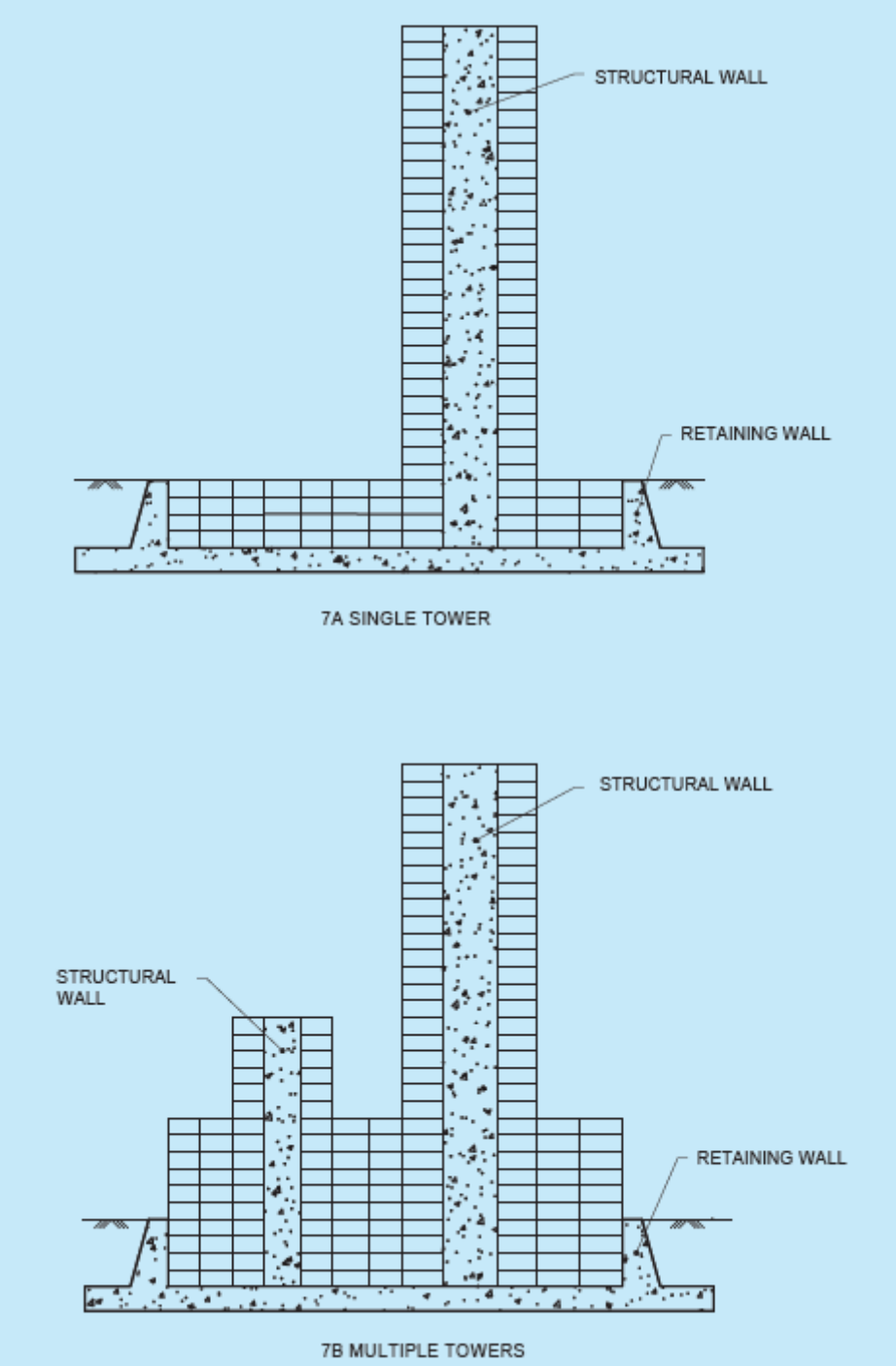

FIG.7 TALL BUILDINGS WITH PODIUMS, CORE STRUCTURAL WALL AND BELOW-GRADE PERIMETER RETAINING WALL

b) Backstay floor diaphragms shall be modelled considering their in-plane and out-of-plane floor flexibility. Any large discontinuity present in the slab shall be modelled;

c) In direct load path case, vertical stiffness shall be considered of the piles, foundation and supporting soil;

d) In backstay load path case, relative stiffness shall be considered of floor diaphragms and perimeter walls, along with vertical in-plane rocking stiffness of soil below the walls. Also, horizontal pressure imposed

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by soil on retaining walls shall be considered. Axial stiffness of elements (representing backstay) along load path shall be reduced to account for cracking, bond slip, interface slip and other such effects;

e) In tall buildings with backstay diaphragms, collector elements shall be provided (*see* Fig. 8), which are capable of transferring forces from the lateral load-resisting system of the tower to the additional elements providing the resistance and in turn to those forces of the podium. Such collector elements shall be provided for transferring lateral forces originating in the other portions of the structure;

f) The backstay diaphragms shall be designed in accordance with the following:

1) They shall be designed for the maximum of forces derived from sensitivity analysis;

2) When the lateral force resisting system of a tall building has plan irregularity as per **5** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS of Type I (torsion irregularity), Type II (re-entrant corners) and Type IV (out of plane offsets), and vertical irregularity as per **5** of Part C/Section 1 of this NBCS of Type IV (in-plane discontinuity of vertical elements resisting lateral forces), seismic forces shall be amplified by a factor of 1.5 in the design of:

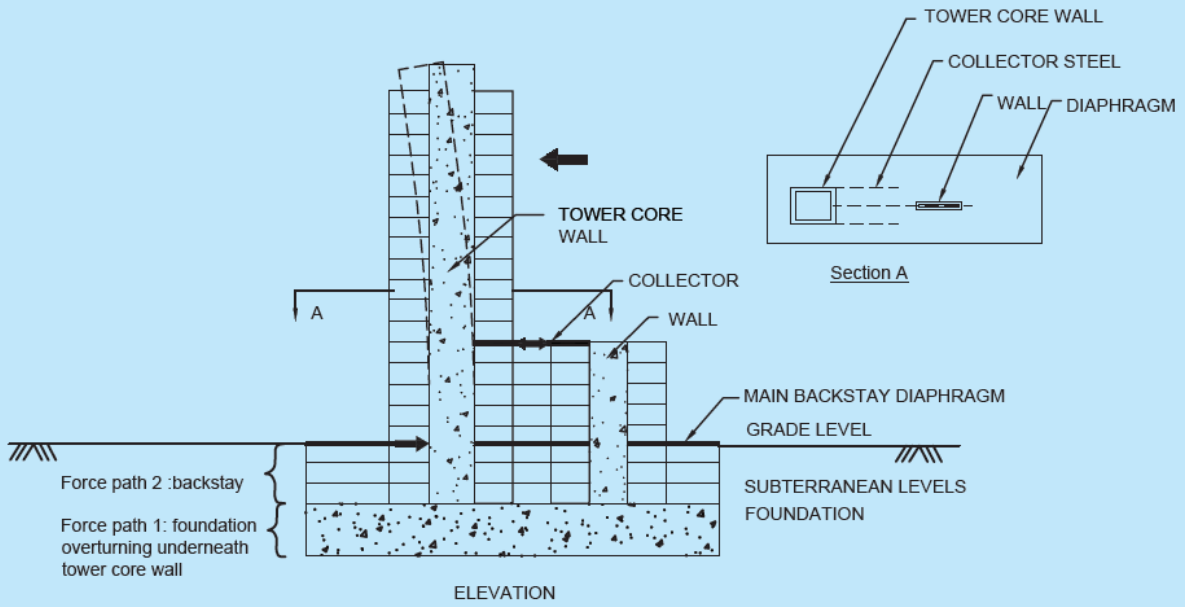
i) Connections of diaphragms to vertical elements and to collectors; and

ii) Collectors and their connections, including connections to vertical elements, of the seismic forces resisting system.

3) When the lateral force resisting system of a tall building has plan irregularity of Type III (diaphragm discontinuity) as per **5** of Part C/Section 1 of this NBCS at the backstay diaphragm levels, collector elements and their connections to vertical elements shall be designed to resist seismic forces amplified by an over-strength factor of 2.5;

4) Backstay diaphragm floor shall be at least 150 mm thick, with double mesh reinforcement not less than 0.25 percent of cross section area in each direction (including both layers of reinforcement steel); and

5) Adequate measures shall be taken to prevent shear sliding failure at connections of diaphragm to structural walls; here, the inclination of the strut shall be taken as 45°. Additional reinforcements shall be provided to resist the shear force at the interface between diaphragm and structural walls.



a) **Path 1**: *Direct Load Path* through Tower Elements, and

b) **Path 2**: *Indirect Load Path* through Backstay Elements

FIG. 8 LOAD PATHS IN LATERAL OVERTURNING RESISTANCE OF TALL BUILDINGS WITH PODIUMS

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E-8.1.3.3.2 *Structural walls*

a) Structural walls shown in Fig. 8 can sustain plastic hinges at the level of the backstay diaphragm also. Such walls shall also be designed and detailed for plastic hinge development at that level; and

b) All peripheral columns of the tower (irrespective of whether they are gravity columns or not) shall be provided with confinement reinforcements throughout the storeys adjoining (above and below) the backstay diaphragm level, as per the requirements of good practice [C-5(5)].

E-8.1.3.3.3 *Towers connected by common podium*

When buildings have two or more towers, they shall be designed considering the following:

a) Such buildings shall be modelled as separate towers as well as integral towers. The podium shall be designed based on the worst of the two results;

b) The estimation of natural period (for calculation of base shear) shall be based on individual building model;

c) In the integral tower modelling:

   1) directional effects for all worst possibilities (that is, tower shaking in the same and in the opposite directions) should be considered in the design load combinations; and

   2) equivalent static seismic forces can be used, provided they are scaled to match base overturning moments obtained from response spectrum analysis.

d) Where significant changes occur to mass or stiffness between the floors, the floor diaphragms of upper and lower levels shall be modelled to capture:

   1) diaphragm forces. Equivalent beam approach, finite element approach or strut and tie approach may be adopted to model the diaphragms; and

   2) potential cracking in the diaphragm by considering an upper bound and a lower bound axial stiffness. The lower and upper bound values of axial and flexural stiffnesses given in Table 13 for sensitivity analysis, may be considered for the cracked section properties, to arrive at the design level earthquake demand on the RC diaphragms.

e) Plan irregularities of Type I (torsion irregularity) and Type III (diaphragm discontinuity) as per **5** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS, shall not be present in the first topmost connected floor and the first floor of the tower above the connected floor;

f) Vertical irregularities of Type I (soft story) as per **5** of Part C/Sec 1 of this NBCS shall not be present in the first connected level and the level above it;

g) Topmost slab between the towers of connected podium shall be at least 150 mm thick with double mesh reinforcements not less than 0.25 percent of cross-sectional area in each direction;

h) Peripheral columns of the tower shall be provided with confinement reinforcements as per good practice [C-5(5)] at the first connected level and a level above;

j) Structural walls of the tower shall be provided with boundary elements as per good practice [C-5(5)] at the uppermost connected level and a level above; and

k) A transfer structure shall not be provided at the first connected floor.

E-8.2 Ductility

Notwithstanding any of the clauses of this standard, the designer shall take all measures to ensure that the building has:

a) sufficient ductility capacity;

b) acceptable energy dissipation mechanism; and

c) desirable sequence of initiation of ductile behaviour in members.

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E-8.3 Frame Buildings

Frame structure for seismic design shall have at least two planar frames with minimum 3 bays in the direction where the lateral load resistance is provided by the moment frames.

E-8.4 Moment Frame — Structural Wall Systems

E-8.4.1 Frame structural wall systems shall be designed as dual systems as per 5 of Part C/Section 1 of this NBCS.

E-8.4.2 In a moment frame–structural wall system, the moment frame shall comply with the requirements of **E-8.3**, and the structural wall with the requirements of **E-8.5**. In addition, the moment frames and structural walls shall comply with the requirements of good practice [C-5(5)].

E-8.4.3 *Special Requirements for Seismic Zone* IV *and above*

Special moment frame and shear walls shall not be discontinued in lower storeys and supported on less stiff and brittle elements.

E-8.5 Structural Wall Systems

E-8.5.1 The thickness of structural wall shall not be less than 160 mm or $h_i/20$, whichever is larger.

E-8.5.2 Opening in structural walls and the associated coupling beams shall meet the following requirements:

a) For walls, when the opening size is less than 800 mm or one-third length of wall, whichever is lesser, in height or length, the influence of the opening may not be taken into account in the overall stiffness of the building;

b) For beams, the opening size shall be less than one-third of the height or length of beam and location of the opening shall be such that the top and bottom one-third of height of the coupling beam is not disturbed; and

c) In either case, all four sides of the opening shall be strengthened with additional reinforcements, and shall comply with requirements of good practice [C-5(5)]. Diameter of reinforcement bars used in this reinforcement shall not be less than 12 mm.

E-8.5.3 Gravity columns in structural wall buildings shall be designed as per requirements of deformation compatibility on non-seismic members given in **5** of Part C/Section 1 of this NBCS.

E-8.5.4 Beams carrying predominant vertical load shall not be supported on coupling beams. Also, columns or structural walls carrying predominant vertical load shall not be supported on coupling beam.

E-8.5.5 In a structural wall system, the structural wall shall comply with the requirements of good practice [C-5(5)].

E-8.5.6 Concentrated gravity loads applied on the wall above the design flexural section shall be assumed to be distributed over a width equal to the bearing width, plus a width on each side that increases at a slope of 2 vertical to 1 horizontal down to the design section, but:

a) not greater than the spacing of the concentrated loads; and

b) not extending beyond the edges of the wall panel.

E-8.5.7 Design of coupling beam shall comply with requirements of good practice [C-5(5)], unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the vertical load carrying ability of the structure, the egress from the structure, or the integrity of non-structural components and their connections to the structure.

E-8.5.8 The nominal design shear stress shall be limited to $\tau_{c,max}$ in structural walls that do not sustain tensile axial stress under any load combination, while the nominal design shear stress shall be limited to $0.5\tau_{c,max}$ in

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structural walls under tension in any load combination and coupling beams in structural walls under factored design loads, where $\tau_{c,max}$ is as per Table 20 in the good practice [C-5(2)].

E-8.5.9 The amount and distribution of the minimum reinforcement in structural walls shall be as per good practice [C-5(5)].

E-8.5.10 At locations where yielding of longitudinal reinforcements is likely to occur as a result of lateral displacement, development length of longitudinal reinforcement shall be 1.25 times the values calculated for the bar yielded in tension, that is at a stress level of f_y.

E-8.5.11 The maximum longitudinal reinforcement ratio in coupling beam shall be as given in Table 14.

Table 14 Maximum Longitudinal Reinforcement in Coupling Beams (Each at Top and Bottom Surface)

(*Clause* E-8.5.11)

Sl No.	Span to Depth Ratio	Maximum Reinforcement, Percent
(1)	(2)	(3)
i)	$L_s/D \leq 1.0$	0.6
ii)	$1.0 \leq L_s/D < 2.0$	1.2
iii)	$2.0 \leq L_s/D$	1.5

E-8.5.12 *Requirements for Each Storey Resisting more than 35 percent of Design Base Shear*

Structural wall with a height-to-length ratio greater than 1.0 may be removed in any storey above, if it accounts for less than one-third of the storey shear strength. Further, the resulting structural system must not have any torsional irregularity as per **5** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS .

E-8.5.13 *Special Requirements for Seismic Zones IV and Above*

a) Structural walls shall be continuous to the base without being transferred in plane or out of plane at any level;

b) The minimum longitudinal reinforcement in structural walls shall not be less than 0.4 percent of gross cross-sectional area;

c) The minimum transverse reinforcement in structural walls shall not be less than 0.25 percent of gross cross-sectional area;

d) The reinforcements shall be distributed in two curtains in each direction;

e) Structural walls shall be fully embedded and anchored at their base in adequate basements or foundations, so that the wall does not rock. In this respect, walls supported by slabs or beams are not permitted; and

f) All openings in structural walls shall preferably be aligned vertically. Random openings, arranged irregularly, shall not be permitted in coupled walls, unless their influence is insignificant.

E-8.6 Flat Slab — Structural Wall Systems

In structures with flat slab system as gravity load carrying system, structural walls shall carry all lateral loads on the building, and column strips of the flat slab system shall not be included in the lateral load resisting system. All requirements related to structures with flat slabs mentioned in **5** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS shall be met with.

E-8.7 Framed Tube System, Tube-in-Tube System and Multiple Tube System

E-8.7.1 The plan shape of a tube-in-tube system shall be regular with a length to width ratio not more than 2 and, the inner tube shall be centered with the outer tube.

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E-8.7.2 Reentrant corners and sharp changes to tubular form should be avoided.

E-8.7.3 Column spacing of framed tube shall preferably be not more than 5 m.

E-8.7.4 In a framed-tube system:

a) Area of corner column shall be 1 to 2 times that of internal column; and

b) Height to width ratio of the opening shall be similar to ratio of storey height to column spacing.

E-8.7.5 Due consideration shall be given to shear lag effects in the design of tube structures.

E-8.7.6 In Seismic Zones III and above:

a) Single span frame shall not be adopted; and

b) Axial compression ratio of columns shall be as per good practice [C-5(5)].

E-8.7.7 Beams carrying predominantly gravity load shall be directly supported on columns or on walls and not on frame beams.

E-8.7.8 The minimum requirements for reinforcement bar diameters in beams of moment frames of framed-tube structures are given in Table 15.

Table 15 Reinforcement Requirements in Beams

(*Clause* E-8.7.8)

All dimensions in millimetres.

Sl No.	Reinforcement Type	Seismic Zone	
		II	III and above
(1)	(2)	(3)	(4)
i)	Stirrup diameter	≥ 8	≥ 10
ii)	Stirrup spacing	≤ 150	≤ 100
iii)	Main (longitudinal) reinforcements	≥ 16	≥ 16

E-9 FOUNDATIONS

E-9.1 Load paths and mechanisms shall be ensured explicitly for transferring vertical and lateral loads between structure and soil system underneath.

E-9.2 A factor of safety of 1.5 shall be provided against overturning and sliding under, (a) unfactored design wind load, gravity load and other permanent imposed load; and (b) 2.5 times design earthquake load, unfactored gravity load and other permanent imposed load .

E-9.3 Geotechnical Investigations

E-9.3.1 All geotechnical investigations needed to establish the safety of the building shall be conducted including for liquefaction potential analysis, and estimation of soil spring constants and modulus of subgrade reaction.

E-9.3.2 For geotechnical investigations, the good practice [C-5(39)] shall be referred.

E-9.4 Depth of Foundation

The embedded depth of the building shall be at least 1/15 of height of building for raft foundation and 1/20 of the height of building for pile and piled raft foundation (excluding pile length). But, this requirement may be relaxed:

a) when the foundation rests on hard rock; or

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b) when there is no uplift under any portion of the raft in any service load combination, and provided the minimum competent founding strata requirement is fulfilled.

E-9.5 Podium and basement roof slab shall be capable of transferring in-plane shear from the tower to the foundation.

E-9.6 Expansion joints shall preferably be avoided in basements of tall buildings. If provided such joints shall be suitably detailed for water tightness.

E-9.7 Modelling of Soil

E-9.7.1 While modelling raft foundations through spring constant or modulus of sub-grade reaction, zoned spring constants or zoned modulus of sub-grade reaction shall be utilized for design, at least for the case of (dead load + live load) condition. For design of rafts for buildings taller than 150 m, a soil-structure interaction study shall be conducted, using actual column loads and column locations to obtain the zoned spring constants.

E-9.7.2 For piled raft foundations designed with settlement reducing piles, soil-structure interaction study shall be conducted with actual column loads and column locations. This analysis shall be conducted at a minimum for combinations of following loads:

a) Dead;

b) Imposed;

c) Wind in X-direction;

d) Wind in Y-direction;

e) Seismic in X-direction; and

f) Seismic in Y-direction.

where X and Y are orthogonal axes in plan.

E-9.8 Settlements of Foundations

Maximum vertical settlement of raft or piled raft foundations under gravity loads shall comply with requirements of good practice [C-5(40)] and good practice [C-5(41)]. The maximum vertical settlement is limited to 125 mm in raft or raft-pile foundation subject to maximum angular distortion of raft not exceeding 1/500 and 50 mm in rock.

E-10 RECOMMENDATIONS FOR NON-STRUCTURAL ELEMENTS

The non-structural elements (NSEs) of tall buildings shall comply with all relevant existing national standards and guidelines as laid down by the various statutory and non-statutory bodies as well as the client/owner of the building.

E-11 MONITORING DEFORMATIONS IN BUILDINGS

E-11.1 Earthquake Shaking

All tall buildings in Seismic Zone V and tall buildings exceeding 150 m in Seismic Zone III and IV shall be instrumented with tri-axial accelerometers to capture translational and twisting behavior of buildings during strong earthquake shaking.

E-11.2 Wind Oscillations

Buildings over 150 m in height may be instrumented with anemometers and accelerometers to measure wind speed, acceleration and direction on top of the buildings.

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E-11.3 Foundation Settlement and Pressure Measurement

E-11.3.1 Permanent settlement markers (at corners and center) shall be provided at raft top level and referenced to a permanent benchmark. Records of settlement shall be maintained till completion of the building and preferably even after completion.

E-11.3.2 Raft or Piled-raft shall be instrumented for monitoring long-term pressure imposed by soil on the raft, at appropriate number (at least 5) of pressure pads below the raft. Alternatively, piles can be instrumented with strain gauges at their top to measure the load on them.

LIST OF STANDARDS

The following list records those standards which are acceptable as 'good practice' and 'accepted standards' in the fulfilment of the requirements of this NBCS. The latest version of a standard shall be adopted at the time of enforcement of this NBCS. The standards listed may be used as a guide in conformance with the requirements of the referred clauses in this NBCS.

In the following list, the number appearing in the first column within parentheses indicates the number of the reference in this Part C/Section 5.

Sl No.	*IS No.*	*Title*
(1)	IS 4845 : 1968	Definitions and terminology relating to hydraulic cement
	IS 6461	Cement concrete — Glossary of terms:
	(Part 1) : 2024	Concrete aggregates (*first revision*)
	(Part 2) : 2024	Materials (other than cement and aggregate) (*first revision*)
	(Part 3) : 2024	Concrete reinforcement (*first revision*)
	(Part 4) : 2024	Types of concrete (*first revision*)
	(Part 5) : 2024	Formwork for concrete (*first revision*)
	(Part 6) : 2024	Equipment, tool and plant (*first revision*)
	(Part 7) : 2024	Mixing, laying, compaction, curing and other construction aspect (*first revision*)
	(Part 8) : 2024	Properties of concrete (*first revision*)
	(Part 9) : 2024	Structural aspects (*first revision*)
	(Part 10) : 2024	Tests and testing apparatus (*first revision*)
	(Part 11) : 2024	Prestressed concrete (*first revision*)
	(Part 12) : 2024	Miscellaneous terms (*first revision*)
(2)	IS 456 : 2000	Plain and reinforced concrete — Code of practice (*fourth revision*)
(3)	IS 1343 : 2012	Prestressed concrete — Code of practice (*second revision*)
(4)	IS 16700 : 2023	Criteria for structural safety of tall concrete buildings (*first revision*)
(5)	IS 13920 : 2016	Ductile design and detailing of reinforced concrete structures subjected to seismic forces — Code of Practice (*first revision*)
(6)	IS 383 : 2016	Coarse and fine aggregate for concrete — Specification (*third revision*)
(7)	IS 9142 (Part 2) : 2018	Artificial lightweight aggregate for concrete — Specification: Part 2 Sintered fly ash coarse aggregate (*first revision*)
(8)	IS 269 : 2015	Ordinary Portland cement — Specification (*sixth revision*)
(9)	IS 455 : 2015	Portland slag cement — Specification (*fifth revision*)
(10)	IS 1489	Portland pozzolana cement — Specification
	Part 1 : 2015	Part 1 Fly ash based (*fourth revision*)

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Sl No.	*IS No.*	*Title*
	Part 2 : 2015	Part 2 Calcined clay based (*fourth revision*)
(11)	IS 16415 : 2015	Composite cement — Specification
(12)	IS 18189 : 2023	Portland calcined clay limestone cement — Specification
(13)	IS 12330 : 1988	Specification for sulphate resisting Portland cement
(14)	IS 16993 : 2018	Microfine ordinary Portland cement — Specification
(15)	IS 8043 : 1991	Hydrophobic Portland cement — Specification (*second revision*)
(16)	IS 12600 : 1989	Portland cement, low heat — Specification
(17)	IS 8041 : 1990	Rapid hardening Portland cement — Specification (*second revision*)
(18)	IS 6452 : 1989	High alumina cement for structural use — Specification (*first revision*)
(19)	IS 6909 : 1990	Supersulphated cement — Specification (*first revision*)
(20)	IS 8042 : 2015	White Portland cement — Specification (*third revision*)
(21)	IS 10500 : 2012	Drinking water— Specification (*second revision*)
(22)	IS 3025 (Part 18) : 2022	Methods of sampling and test (physical and chemical) for water and wastewater: Part 18 Volatile and fixed solids (total, filterable and non-filterable at 550 C (*second revision*)
(23)	IS 3025 (Part 24/Sec 1) : 2022	Methods of sampling and test (physical and chemical) for water and wastewater: Part 24 Sulphates, Section 1 Gravimetric and turbidity methods (*second revision*)
(24)	IS 3025 (Part 32) :1988	Methods of sampling and test (physical and chemical) for water and wastewater: Part 32 Chloride (*first revision*)
(25)	IS 3025 (Part 17) : 2022	Methods of sampling and test (physical and chemical) for water and wastewater Part 17 Non- filterable residue total suspended solids at 103 °C to 105 ºC (*second revision*)
(26)	IS 3812 (Part 1) : 2013	Pulverized fuel ash — Specification: Part 1 For use as pozzolana in cement, cement mortar and concrete (*third revision*)
(27)	IS 15388 : 2003	Silica fume — Specification
(28)	IS 16354 : 2015	Metakaolin for use in cement, cement mortar and concrete — Specification
(29)	IS 16714 : 2018	Ground granulated blast furnace slag for use in cement, mortar and concrete — Specification
(30)	IS 16715 : 2018	Ultrafine ground granulated blast furnace slag — Specification
(31)	IS 19058 : 2024	Ultrafine fly ash — Specification
(32)	IS 9103 : 1999	Concrete admixtures — Specification (*first revision*)
(33)	IS 16172 : 2023	Reinforcement couplers for mechanical splices of steel bars in concrete — Specification (*first revision*)
(34)	IS 18256 : 2023	Solid round glass fibre reinforced polymer (GFRP) bars for concrete reinforcement — Specification
(35)	IS 1566 : 1982	Specification for hard-drawn steel wire fabric for concrete reinforcement (*second revision*)
(36)	IS 1199 (Part 6) : 2018	Fresh concrete — methods of sampling, testing and analysis: Part 6 Tests on fresh self compacting concrete (*first revision*)
(37)	IS 10262 : 2019	Concrete mix proportioning — Guidelines (*second revision*)
(38)	IS 4031 (Part 5) : 1988	Methods of physical tests for hydraulic cement: Part 5 Determination of initial and final setting times (*first revision*)

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Sl No.	*IS No.*	*Title*
(39)	IS 1892 : 2021	Subsurface investigation for foundations — Code of practice (*second revision*)
(40)	IS 1904 : 2021	General requirements for design and construction of foundations in soils — Code of practice (*fourth revision*)
(41)	IS 12070 : 1987	Code of practice for design and construction of shallow foundations on rocks

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 6 STEEL AND COMPOSITE CONSTRUCTION

SUBSECTION 6(A) STEEL

BUREAU OF INDIAN STANDARDS

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National Building Code Sectional Committee, CED 46

FOREWORD

This NBCS (Part C/Subsection 6A) covers the structural design aspects of steel structures in buildings.

This Section covers the use of hot-rolled structural steel sections and steel tubes in buildings. It covers the design by limit state method and plastic theory.

This Section was first published in 1970 and revised in 1983, 2005 and 2016 as a chapter of the National Building Code of India. In the second revision of 2005, the detailed design provisions were not included as IS 800 : 1984 'Code of practice for general construction in steel (*second revision*)' was under revision at that time. This Section, therefore, then referred to the 1984 version of IS 800 with the remarks that the latest version would prevail, as and when published. It, however, covered the philosophical aspects of limit state design which was being introduced in the revision of IS 800. IS 800 was subsequently revised and published as IS 800 : 2007 and hence, became the prevailing version for use under this Section. In the 2016 version of the Code, this Section was brought it in line with the revised version of IS 800 : 2007 read with its Amendment No. 1 and to take care of further improvements and incorporations required.

In the current revision, this publication has been renamed as National Building Construction Standards (NBCS).

This revision of the Section is intended to bring it in line with the revised version of IS 800 : 2026 (*under publication*) to take care of further improvements and incorporations required. In this revision, therefore, the following major modifications have been made:

a) The working stress design has been discontinued due to the increasing familiarity of designers with the limit state design of steel structures;

b) The table on tensile properties of structural steel products has been updated by including the latest Indian Standards relating to structural steels and aligning the corresponding provisions;

c) Fire-resistant steel as per IS 15103 : 2002 'Fire resistant steel — Specification', structural weather-resistant steels as per IS 11587 : 1986 'Structural weather resistant steels — Specification', and structural steels for buildings and structures with improved seismic resistance as per IS 15962 : 2012 'Structural steels for buildings and structures with improved seismic resistance — Specification' have been included in the table of tensile properties of structural steel products;

d) Provisions related to the requirement of temperature stress analysis have been updated;

e) Provisions related to the requirement of fire load have been updated;

f) Table 2 Limiting width thickness ratio of elements has been updated;

g) Provisions related to the consideration of second order effects have been updated;

h) Provisions related to design for earthquake loads relating to load and load combinations, various types of frames and connections in the critical zones of frames have been updated;

j) Provisions related to bracing connections have been updated;

k) Provisions related to fabrication procedures have been updated;

m) Certain terminologies, symbols and figures have been updated based on the changes incorporated;

n) Buckling class of cross section table has been updated;

p) Imperfection parameter for lateral torsional buckling of beam has been added;

q) A new clause on sustainable steel construction has been added. This provision covers the following:

   1) Requirements from Engineers, contractors and supply chain members;

   2) Sustainable design process by minimizing the materials, fabrication; and avoidance of maintenance; and

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3) Design for adaptability and deconstruction, etc.

r) The detailed provision relating to design of corrugated web girder have been introduced in Annex G.

The revised Section will enhance the confidence of designers, engineers, contractors, technical institutions, professional bodies and the industry and will open a new era in safe and economic construction in steel.

The provisions relating to using steel tubes in general building construction are based on IS 806 : 1968 ‘Code of practice for the use of steel tubes in general building construction’, which is complementary to IS 800. The design using tubular steel in structural applications may be carried out using the limit state design provisions in this Section until IS 806 is revised as per the limit state design method.

The design of cold-formed steel members is based on the provisions of IS 801 : 1975 ‘Code of practice for use of cold formed light gauge steel structural members in general building construction’. The standard is currently under revision to be based on the limit state design practice. Until the revised IS 801 is published, the cold formed steel members may be designed using the current IS 801, based on the working stress method.

All standards, whether given herein above or cross-referred to in the main text of this section, are subject to revision. The parties to agreement based on this Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 2022 ‘Rules for rounding of numerical values (*second revision*)’. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

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Important Explanatory Note for Users of the NBCS

In any Part/Section of this NBCS, where reference is made to **'good practice'** in relation to design, constructional procedures or other related information, and where reference is made to **'accepted standard'** in relation to material specification, testing, or other related information, the Indian Standards listed at the end of the Part/Section shall be used as a guide to the interpretation.

At the time of publication, the editions indicated in the standards were valid. All standards are subject to revision and parties to agreements based on any Part/ Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

In the list of standards given at the end of a Part/Section, the number appearing within parentheses in the first column indicates the number of the reference of the standard in the Part/Section. For example:

a) Good practices [C-6A(1)] refers to the Indian Standard(s) give at serial number (1) of the list of standards given at the end of this Part C/Subsection 6A, that is, IS 786 : 1967 'Conversion factors and conversion tables (*first revision*)'.

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 6 STEEL AND COMPOSITE CONSTRUCTION

SUBSECTION 6(A) STEEL

1 SCOPE

1.1 This NBCS (Part C/Subsection 6A) covers the structural design aspects of steel structures in buildings. This Section applies to general construction using hot rolled steel sections and steel tubes joined using riveting, bolting, and welding. Cold formed light gauge steel sections are covered in separate standards. This Section covers the design by limit state method and plastic theory.

1.2 This Section gives only general guidance as regards to the various loads to be considered in the design. For the actual loads, such as dead, imposed, snow, wind and earthquake loads and load combinations to be used, reference may be made to Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS.

1.3 Fabrication and erection requirements covered in this Section are general and the minimum necessary quality of material and workmanship consistent with assumptions in the design rules. The actual requirements may be further developed as per other standards or the project specification, the type of structure and the method of construction.

1.4 For seismic design, recommendations pertaining to steel frames only are covered in this section. For more detailed information on seismic design of other structural and non-structural components, reference be made to **5** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS and other special publications on the subject.

2 –

NOTE — This clause is intentionally left blank.

3 TERMINOLOGY

For the purpose of this standard, the following definitions shall apply.

3.1 Accidental Loads — Loads due to explosion, impact of vehicles, or other rare loads for which the structure is susceptible as per user.

3.2 Accompanying Load — Imposed load acting along with leading imposed load but Causing action/deflection lesser than the leading load.

3.3 Action Effect or Load Effect — Internal response of a structural element, including axial force, shear force, bending moment, and torsional moment, resulting from applied external loads, temperature variations, and other environmental influences such as seismic activity, settlement effects, and residual stresses.

3.4 Action — The primary cause for stress or deformations in a structure such as dead, imposed, wind, seismic or temperature loads.

3.5 Actual Length — The distance measured from centre to centre of intersection points with supporting members, applicable to beams, trusses, or other structural elements. In the case of a cantilevered member, it refers to the full projection from the fixed support.

3.6 Beam — A member subjected predominantly to bending.

3.7 Bearing Type Connection — A connection made using bolts in 'snug-tight' condition, or rivets where the load is transferred by bearing of bolts or rivets against plate inside the bolt hole.

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3.8 Braced Member — A member in which the relative transverse displacement is effectively prevented by bracing.

3.9 Brittle Cladding — Claddings, such as cement fibre boards which get damaged before undergoing considerable deformation.

3.10 Buckling Load — The load at which an element, a member or a structure as a whole, either collapses in service or buckles in a load test and develops excessive lateral (out of plane) deformation or instability.

3.11 Buckling Strength or Resistance — Force or moment, which a member can withstand without buckling.

3.12 Built-up Section — A member fabricated by interconnecting more than one element to form a compound section acting as a single member.

3.13 Camber — Intentionally introduced pre-curving (usually upwards) in a system, member or any portion of a member with respect to the straight line joining the end points in place. Frequently, camber is introduced to compensate for deflections at a specific level of loads.

3.14 Characteristic Load (Action) — The value of specified load (action), above which not more than a specified percentage (usually 5 percent) of samples of corresponding load are expected to be encountered.

3.15 Characteristic Yield/Ultimate Stress — The minimum value of stress, below which not more than a specified percentage (usually 5 percent) of corresponding stresses of samples tested are expected to occur.

3.16 Column — A member in upright (vertical) position which supports a roof or floor system and predominantly subjected to compression.

3.17 Compact Section — A cross section, which can develop plastic moment, but has inadequate plastic rotation capacity needed for formation of a plastic collapse mechanism of the member or structure.

3.18 Constant Stress Range — The amplitude between which the stress ranges under cyclic loading is assumed to be constant during the life of the structure or a structural element.

3.19 Corrosion — An electrochemical process over the surface of steel, leading to oxidation of the metal.

3.20 Crane Load — The horizontal and vertical forces exerted by a crane on a structure, including dynamic effects due to crane movement, acceleration, and braking caused by its due to its self-weight and load lifted.

3.21 Cumulative Fatigue — Total damage due to fatigue loading of varying stress ranges.

3.22 Cut-off Limit — The stress range, corresponding to the particular detail, below which cyclic loading need not be considered in cumulative fatigue damage evaluation (corresponds to 10^8 number of cycles in most cases).

3.23 Dead Loads — The self-weights of all permanent constructions and installations including the self-weight of all walls, partitions, floors, roofs, and other permanent fixtures acting on a member.

3.24 Deflection — The deviation from the standard position of a member or structure.

3.25 Design Life — Time period for which a structure or a structural element is required to perform its function without damage.

3.26 Design Load/Factored Load — A load value obtained by multiplying the characteristic load with the partial safety factor for loads.

3.27 Design Spectrum — Frequency distribution of the stress ranges from all the nominal loading events during the design life (stress spectrum).

3.28 Detail Category — Designation given to a particular detail to indicate the S-N curve to be used in fatigue assessment.

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3.29 Discontinuity — A sudden change in cross section of a loaded member, causing a stress concentration at the location.

3.30 Ductility — The property of the material or a structure indicating the extent to which it can deform beyond the limit of yield deformation before failure or fracture. The ratio of ultimate to yield deformation is usually termed as ductility.

3.31 Durability — The ability of a material to resist deterioration over long periods of time.

3.32 Earthquake effects — The inertia forces and relative deformations induced in a structure due to the ground shaking during an earthquake.

3.33 Edge Distance — Distance from the centre of a fastener hole to the nearest edge of an element measured perpendicular to the direction of load transfer.

3.34 Effective Lateral Restraint — Restraint that produces sufficient resistance to prevent deformation in the lateral direction.

3.35 Effective Length — Actual length of a member between points of effective restraint or effective restraint and free end, multiplied by a factor to take account of the end conditions in buckling strength calculations.

3.36 Elastic Cladding — Claddings, such as metal sheets that can undergo considerable deformation without damage.

3.37 Elastic Critical Moment — The elastic moment, which initiates lateral-torsional buckling of a laterally unsupported beam.

3.38 Elastic Design — Design, which assumes elastic behaviour of materials throughout the service load range.

3.39 Elastic Limit — The stress below which the material regains its original size and shape when the load is removed. In steel design, it is taken as the yield stress.

3.40 End Distance — Distance from the centre of a fastener hole to the edge of an element measured parallel to the direction of load transfer.

3.41 Erection Loads — The actions (loads and deformations) experienced by the structure exclusively during erection.

3.42 Erection Tolerance — Amount of deviation related to the plumbness, alignment, and level of the element as a whole in the erected position. The deviations are determined by considering the locations of the ends of the element.

3.43 Exposed Surface Area to Mass Ratio — The ratio of the surface area exposed to the fire (in mm^2) to the mass of steel (in kg).

NOTE — In the case of members with fire protection material applied, the exposed surface area is to be taken as the internal surface area of the fire protection material.

3.44 Fabrication Tolerance — Amount of deviation allowed in the nominal dimensions and geometry in fabrication activities, such as cutting to length, finishing of ends, cutting of bevel angles, etc.

3.45 Factor of Safety — The factor by which the yield stress of the material of a member is divided to arrive at the permissible stress in the material.

3.46 Fatigue — Damage caused by repeated fluctuations of stress, leading to progressive cracking of a structural element.

3.47 Fatigue Loading — Set of nominal loading events, cyclic in nature, described by the distribution of the loads, their magnitudes and the numbers of applications in each nominal loading event.

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3.48 Fatigue Strength — The stress range for a category of detail, depending upon the number of cycles it is required to withstand during design life.

3.49 Fire Exposure Condition

3.49.1 *Three-Sided Fire Exposure Condition* — Steel member incorporated in or in contact with a concrete or masonry floor or wall (at least against on surface); and

NOTES

1 Three-sided fire exposure condition is to be considered separately unless otherwise specified (*see* **22.9**).

2 Members with more than one face in contact with a concrete or masonry floor or wall may be treated as three-sided fire exposure.

3.49.2 *Four-Sided Fire Exposure Condition* — Steel member, which may be exposed to fire on all sides.

3.50 Fire Protection System — The fire protection material and its method of attachment to the steel member.

3.51 Fire Resistance — The ability of an element, component or structure, to fulfil for a stated period of time, the required stability, integrity, thermal insulation and/or other expected performance specified in a standard fire test.

3.52 Fire Resistance Level — The fire resistance grading period for a structural element or system, in minutes, which is required to be attained in the standard fire test.

3.53 Flexural Stiffness — Stiffness of a member against rotation as evaluated by the value of bending deformation moment required to cause a unit rotation while all other degrees of freedom of the joints of the member except the rotated one are assumed to be restrained.

3.54 Friction Type Connection — Connection effected by using pre-tensioned high strength bolts where shear force transfer is due to mobilization of friction between the connected plates due to clamping force developed at the interface of connected plates by the bolt pre-tension.

3.55 Gauge — The spacing between adjacent parallel lines of fasteners, transverse to the direction of load/stress.

3.56 Gravity Load — Loads arising due to gravitational effects.

3.57 Gusset Plate — The plate to which the members intersecting at a joint are connected.

3.58 High Shear — High shear condition is caused when the actual shear due to factored load is greater than a certain fraction of design shear resistance (*see* **16.2.2**).

3.59 Imposed (Live) Load — The load assumed to be produced by the intended use or occupancy including distributed, concentrated, impact, vibration and snow loads but excluding, wind, earthquake and temperature effects.

3.60 Instability — The phenomenon which disables an element, member or a structure to carry further load due to excessive deflection lateral to the direction of loading and vanishing stiffness.

3.61 Lateral Restraint for a Beam — *See* **3.34**.

3.62 Leading Imposed Load — Imposed load causing higher action and/or deflection.

3.63 Limit State — Any limiting condition beyond which the structure ceases to fulfill its intended function (*see* also **3.86**).

3.64 Live Load — *See* **3.59**.

3.65 Load — An externally applied force or action (*see also* **3.4**).

3.66 Main Member — A structural member, which is primarily responsible for carrying and distributing the applied load or action.

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3.67 Mill Tolerance — Amount of variation allowed from the nominal dimensions and geometry, with respect to cross sectional area, non-parallelism of flanges, and out of straightness such as sweep or camber, in a product, as manufactured in a steel mill.

3.68 Normal Stress — Stress component acting normal to the face, plane or section.

3.69 Partial Safety Factor — The factor normally greater than unity by which either the loads (actions) are multiplied or the resistances are divided to obtain the design values.

3.70 Period of Structural Adequacy under Fire — The time (t), in minutes, for the member to reach the limit state of structural inadequacy in a standard fire test.

3.71 Permissible Stress — When a structure is being designed by the working stress method, the maximum stress that is permitted to be experienced in elements, members or structures under the nominal/service load (action).

3.72 Pitch — The centre to centre distance between individual fasteners in a line, in the direction of load/stress.

3.73 Plastic Collapse — The failure stage at which sufficient number of plastic hinges have formed due to the loads (actions) in a structure leading to a failure mechanism.

3.74 Plastic Design — Design against the limit state of plastic collapse.

3.75 Plastic Hinge — A yielding zone with significant inelastic rotation, which forms in a member, when the plastic moment is reached at a section.

3.76 Plastic Moment — Moment capacity of a cross section when the entire cross section has yielded due to bending moment.

3.77 Plastic Section — Cross section, which can develop a plastic hinge and sustain plastic moment over sufficient plastic rotation required for formation of plastic failure mechanism of the member or structure.

3.78 Poisson's Ratio — The absolute value of the ratio of lateral strain to longitudinal strain under uniaxial loading.

3.79 Proof Stress — The stress to which high strength friction grip (HSFG) bolts are pre-tensioned.

3.80 Proof Testing — The application of test loads to a structure, sub-structure, member or connection to ascertain the structural characteristics of only that specific unit.

3.81 Prototype Testing — Testing of structure, sub-structure, members or connections to ascertain the structural characteristics of that class of structures, sub-structures, members or connections that are nominally identical (full scale) to the units tested.

3.82 Prying Force — Additional tensile force developed in a bolt as a result of the flexing of a connection component such as a beam end plate or leg of an angle.

3.83 Rotation — The change in angle at a joint between the original orientation of two linear member and their final position under loading.

3.84 Secondary Member — Member which is provided for overall stability and or for restraining the main members from buckling or similar modes of failure.

3.85 Semi-compact Section — Cross section, which can attain the yield moment, but not the plastic moment before failure by plate buckling.

3.86 Serviceability Limit State — A limit state of acceptable service condition exceedance of which causes serviceability failure.

3.87 Shear Force — The in-plane force at any transverse cross section of a straight member of a column or beam.

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3.88 Shear lag — The in plane shear deformation effect by which concentrated forces tangential to the surface of a plate gets distributed over the entire section perpendicular to the load over a finite length of the plate along the direction of the load.

3.89 Shear Stress — The stress component acting parallel to a face, plane or cross section.

3.90 Slender Section — Cross section in which the elements buckle locally before reaching yield moment.

3.91 Slenderness Ratio — The ratio of the effective length of a member to the radius of gyration of the cross section about the axis under consideration.

3.92 Slip Resistance — Limit shear that can be applied in a friction grip connection before slip occurs.

3.93 S-N Curve — The curve defining the relationship between the number of stress cycles to failure (N_{sc}) at a constant stress range (S_c), during fatigue loading of a structure.

3.94 Snow Load — Load on a structure due to the accumulation of snow and ice on surfaces such as roof.

3.95 Snug Tight — The tightness of a bolt achieved by a few impacts of an impact wrench or by the full effort of a person using a standard spanner.

3.96 Stability Limit State — A limit state corresponding to the loss of static equilibrium of a structure by excessive deflection transverse to the direction of predominant loads.

3.97 Stickability — The ability of the fire protection system to remain in place as the member deflects under load during a fire test.

3.98 Stiffener — An element used to retain or prevent the out-of-plane deformations of plates.

3.99 Strain — Deformation per unit length or unit angle.

3.100 Strain Hardening — The phenomenon of increase in stress with increase in strain beyond yielding.

3.101 Strength — Resistance to failure by yielding or buckling.

3.102 Strength Limit State — A limit state of collapse or loss of structural integrity.

3.103 Stress — The internal force per unit area of the original cross section.

3.104 Stress Analysis — The analysis of the internal force and stress condition in an element, member or structure.

3.105 Stress Cycle Counting — Sum of individual stress cycles from stress history arrived at using any rational method.

3.106 Stress Range — Algebraic difference between two extremes of stresses in a cycle of loading.

3.107 Stress Spectrum — Histogram of stress cycles produced by a nominal loading event design spectrum, during design life.

3.108 Structural Adequacy for Fire — The ability of the member to carry the test load exposed to the standard fire test.

3.109 Structural Analysis — The analysis of stress, strain, and deflection characteristics of a structure.

3.110 Strut — A compression member, which may be oriented in any direction.

3.111 Sway — The lateral deflection of a frame.

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3.112 Sway Member — A member in which the transverse displacement of one end, relative to the other is not effectively prevented.

3.113 Tensile Stress — The characteristic stress corresponding to rupture in tension, specified for the grade of steel in the appropriate Indian Standard, as listed in Table 1.

3.114 Test Load — The factored load, equivalent to a specified load combination appropriate for the type of test being performed.

3.115 Transverse — Direction along the stronger axes of the cross section of the member.

3.116 Ultimate Limit State — The state which, if exceeded can cause collapse of a part or the whole of the structure.

3.117 Ultimate Stress — *See* **3.113**.

3.118 Wind Loads — Load experienced by member or structure due to wind pressure acting on the surfaces.

3.119 Yield Stress — The characteristic stress of the material in tension before the elastic limit of the material is exceeded, as specified in the appropriate Indian Standard, as listed in Table 1.

3.120 Terms Relating to Sustainable Steel Design

3.120.1 *Embodied Carbon* — Life-cycle greenhouse gas emissions (expressed as carbon dioxide equivalents – CO2e) that occur during the manufacture and transport of construction materials and components, as well as the construction process itself and end-of-life aspects of the building.

NOTE — In recent years, the term embodied carbon of construction materials and products has become synonymous with the term carbon footprint.

3.120.2 *Deconstruction* (*or Disassembly, or Demounting*) — Deconstruction is the process of taking a building apart into its components in such a way that they can be more readily reused; it minimizes the destructive aspects of the process of removing buildings, preserving components and materials not wasting them, and treating them as a resource.

3.120.3 *Design for Deconstruction* (*DfD*) — Designing for deconstruction is considering, at the design stage, how a building can be taken apart.

3.120.4 *Environmental Product Declaration* (*EPD*) — An EPD communicates verifiable, accurate, non-misleading environmental information for products and their applications; EPD information is expressed in information modules, which allow easy organization and expression of data packages throughout the life cycle of the product.

3.120.5 *Material Efficiency* — Optimizing the use of the steel by avoiding waste during fabrication and avoiding over specification of steel in the design.

3.120.6 *Recycling* — Steel involves re-melting of scrap to form new semi-finished products.

3.120.7 *Responsible Sourcing* — Management of sustainable development in the provision or procurement of a product.

3.120.8 *Reuse* — Use of previously used components with little or no reprocessing, largely in their original form; they may be reused for the original function (a conventional reuse scenario), or repurposed.

3.120.9 *Sustainable Development* — An enduring, balanced approach to economic activity, environmental responsibility and social progress.

3.120.10 *Traceability* — Ability to trace the history, application or location of an object.

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4 SYMBOLS

For the purpose of this Subsection, the symbols (other than those used for load categorization) given in Annex A are applicable.

5 UNITS

For the purpose of design calculations the following units are recommended:

a) forces and loads, in kN, kN/m, kN/m^2;

b) unit mass, in kg/m^3;

c) unit weight, in kN/m^3;

d) stresses and strengths, in N/mm^2 (= MN/m^2 or MPa); and

e) moments (bending, etc), in kNm.

For conversion of one system of units to another system, [C-6A(1)] may be referred.

6 STANDARD DIMENSIONS, FORM AND WEIGHT

The dimensions, form, weight, tolerances of all rolled shapes, all rivets, bolts, nuts, studs, and welds and other members used in any steel structure shall conform to accepted standards [C-6A(2)], wherever applicable.

7 PLANS AND DRAWINGS

7.1 Plans, drawings and stress sheet shall be prepared according to accepted standards [C-6A(3)].

7.1.1 *Plans*

The plans (design drawings) shall show the sizes, sections, and the relative locations of the various members. Floor levels, column centres, and offsets shall be dimensioned. Plans shall be drawn to a scale large enough to convey the information adequately. Plans shall indicate the type of construction to be employed; and shall be supplemented by such data on the assumed loads, shears, moments and axial forces to be resisted by all members and their connections, as may be required for the proper preparation of shop drawings. Any special precaution to be taken in the erection of structure, from the design consideration shall also be indicated in the drawing.

7.1.2 *Shop Drawings*

Shop drawings, giving complete information necessary for the fabrication of the component parts of the structure including the location, type, size, length and detail of all welds and fasteners shall be prepared in advance of the actual fabrication. They shall clearly distinguish between shop and field rivets, bolts and welds. For additional information to be included on drawings for designs based on the use of welding, reference shall be made to appropriate Indian Standards. Shop drawings shall be made in conformity with accepted standard [C-6A(4)]. A marking diagram allotting distinct identification marks to each separate part of steel work shall be prepared. The diagram shall be sufficient to ensure convenient assembly and erection at site.

7.2 Symbols used for welding on plans and shop drawings shall be according to good practice [C-6A(5)].

8 CONVENTION FOR MEMBER AXES

Unless otherwise specified convention used for member axes is as follows (Fig. 1):

a) *x-x* along the member;

b) *y-y* an axis of the cross section:

   1) perpendicular to the flanges; and

   2) perpendicular to the smaller leg in an angle section.

c) *z-z* an axis of the cross-section:

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1) axis parallel to flanges; and

2) axis parallel to smaller leg in angle section.

d) *u-u* major axis (when it does not coincide with *z-z* axis); and

e) *v-v* minor axis (when it does not coincide with *y-y* axis).

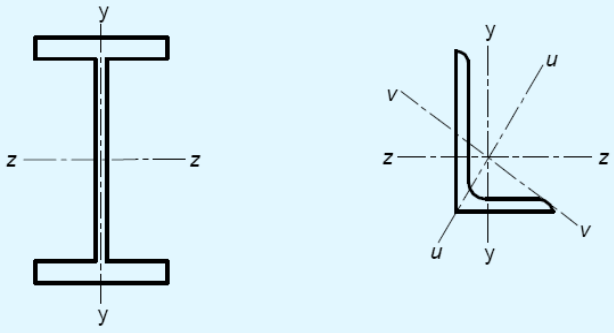

FIG. 1 AXES OF MEMBERS

9 MATERIALS

9.1 General

The material properties given in this clause are nominal values, to be accepted as characteristic values in design calculations.

9.2 Structural Steel

9.2.1 The provisions in this Section are applicable to the steels commonly used in steel construction namely, structural mild steel and high tensile structural steel.

9.2.2 All the structural steel used in general construction, coming under the purview of this standard shall before fabrication conform to accepted standard [C-6A(6)].

9.2.3 Structural steel other than those specified in 9.2.2 may also be used provided that the permissible stresses and other design provisions are suitably modified and the steel is also suitable for the type of fabrication adopted.

9.2.3.1 Steel that is not supported by mill test result may be used only in unimportant members and details, where their properties such as ductility and weldability would not affect the performance requirements of the members and the structure as a whole.

However, such steels may be used in structural system after confirming their quality by carrying out appropriate tests in accordance with the method specified in accepted standard [C-6A(7)].

9.2.4 *Properties*

The properties of structural steel for use in design may be taken as given in 9.2.4.1 and 9.2.4.2.

9.2.4.1 Physical properties of structural steel irrespective of its grade may be taken as:

a) unit mass of steel, ρ = 7 850 kg/m^3;

b) modulus of elasticity, $E = 2.0 \times 10^5$ N/mm^2 (MPa);

c) Poisson's ratio, $\mu = 0.3$;

d) modulus of rigidity, $G = 0.769 \times 10^5$ N/mm^2 (MPa); and

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e) coefficient of thermal expansion $\alpha_t = 12 \times 10^{-6}$ /°C

9.2.4.2 *Mechanical properties of structural steel*

The principal mechanical properties of the structural steel important in design are the yield stress, f_y; the tensile or ultimate stress, f_u; the maximum percent elongation on a standard gauge length and notch toughness. Except for notch toughness, the other properties are determined by conducting tensile tests on samples cut from the plates, sections, etc, in accordance with accepted standard [C-6A(7)]. Commonly used properties for the common steel products of different specifications are summarized in Table 1.

9.2.4.2.1 *Steel tubes* — Steel tubes used in building construction shall be hot finished tubes conforming to the requirements specified in accepted standard [C-6A(8)].

9.2.4.2.2 Tubes made by other than hot finishing processes, or which have been subjected to cold working, shall be regarded as hot finished if they have subsequently been heat-treated and are supplied in the normalized conditions.

9.2.4.3 *Electrodes* — The electrodes used for welding steel tubes shall conform to the requirements of accepted standard [C-6A(9)] or [C-6A(10)], as appropriate.

9.3 Rivets

9.3.1 Rivets shall be manufactured from steel conforming to accepted standard [C-6A(11)]. They may also be manufactured from steel conforming to accepted standard [C-6A(6)] provided that the steel meets the requirements given in accepted standard [C-6A(12)].

9.3.2 Rivets shall conform to accepted standards [C-6A(13)] as appropriate.

9.3.3 *High Tensile Steel Rivets*

High tensile steel rivets, shall be manufactured from steel conforming to accepted standard [C-6A(12)].

9.4 Bolts, Nuts and Washers

Bolts, nuts and washers shall conform as appropriate to accepted standards [C-6A(14)]. The recommendations in good practice [C-6A(15)] shall be followed.

9.5 Steel Casting

Steel casting shall conform to accepted standard [C-6A(16)] or accepted standard [C-6A(17)].

9.6 Welding Consumable

9.6.1 Covered electrodes shall conform to accepted standard [C-6A(9)] or [C-6A(10)], as appropriate.

9.6.2 Filler rods and wires for gas welding shall conform to accepted standard [C-6A(18)].

9.6.3 The supply of solid filler wires for submerged arc welding of structural steels shall conform to accepted standard [C-6A(19)].

9.6.4 The bare wire electrodes for submerged arc welding and the combination of wire and flux shall satisfy the requirements of accepted standard [C-6A(20)].

9.6.5 Filler rods and bare electrodes for gas shielded metal arc welding shall conform to accepted standards [C-6A(21)], as appropriate.

9.7 Other Materials

Other materials used in association with structural steel work shall conform to appropriate Indian standards.

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Table 1 Tensile Properties of Structural Steel Products

(*Clauses* 3.113, 3.119 *and* 9.2.4.2)

Sl No.	Accepted Standard	Grade/ Classification	Properties											
			Yield Stress, MPa *Min*				Ultimate Tensile Stress, MPa *Min*	Elongation, Percent *Min*						
(1)	(2)	(3)	(4)				(5)	(6)						
i)	[C-6A(22)]							Gauge Length						
								80 mm				50 mm		
		CR1	280				410	27				28		
		CR2	240				370	30				31		
		CR3	220				350	34				35		
		CR4	210				350	36				37		
		CR5	190				350	38				40		
			thickness, *t*, mm					Gauge Length-50mm and thickness, *t*, mm						
			< 0.40	≥ 0.40 and < 0.80	≥ 0.80 and < 1.00	≥ 1.00		< 0.40	≥ 0.40 and < 0.60	≥ 0.60 and <0.80	≥ 0.80 and < 1.00	≥ 1.00 and < 1.20	≥ 1.20 and < 1.60	≥ 1.60 and < 2.00 or ≥ 2.00 and < 2.50 or > 2.50
		ISC590LA	—	430 to 580	420 to 570	410 to 560	590	—	—	17 to 32	17 to 32	18 to 33	18 to 33	≥18
		ISC490W	—	305 to 410	295 to 400	295 to 400	490	—	—	23 to 35	23 to 35	24 to 36	24 to 36	≥ 24
		ISC540W	—	335 to 440	325 to 430	325 to 430	540	—	—	20 to 32	20 to 32	21 to 33	21 to 33	≥ 21
		ISC590W	—	365 to 470	355 to 460	355 to 460	590	—	—	17 to 29	17 to 29	18 to 30	18 to 30	≥ 18
		ISC450Y	—	270 to 360	260 to 370	250 to 340	450	—	—	26 to 42	27 to 43	28 to 44	29 to 45	≥ 29
		ISC490Y	—	310 to 410	300 to 400	290 to 390	490	—	—	22 to 38	23 to 39	24 to 40	25 to 41	≥ 25
		ISC540Y	—	340 to 450	330 to 440	320 to 430	540	—	—	18 to 35	19 to 36	20 to 37	21 to 38	≥ 21

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Table 1 (*Continued*)

Sl No.	Accepted Standard	Grade / Classification	Properties		
			Yield Stress, MPa *Min*	Ultimate Tensile Stress, MPa *Min*	Elongation, Percent *Min*
(1)	(2)	(3)	(4)	(5)	(6)
ii)	[C-6A(9)]				
		Ex40xx	330	410 to 540	16
		Ex41xx	330	410 to 540	20
		Ex42xx	330	410 to 540	22
		Ex43xx	330	410 to 540	24
		Ex44xx	330	410 to 540	24
		Ex50xx	360	510 to 610	16
		Ex51xx	360	510 to 610	18
		Ex52xx	360	510 to 610	18
		Ex53xx	360	510 to 610	20
		Ex54xx	360	510 to 610	20
		Ex55xx	360	510 to 610	20
		Ex56xx	360	510 to 610	20

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Table 1 (*Continued*)

<table>
<tr><th rowspan="3">Sl No.</th><th rowspan="3">Accepted Standard</th><th rowspan="3">Grade/ Classification</th><th colspan="11">Properties</th></tr>
<tr><th colspan="4">Yield Stress, MPa
Min</th><th rowspan="2">Ultimate Tensile Stress, MPa
Min</th><th colspan="6">Elongation, Percent Min</th></tr>
<tr><td>(1)</td><td>(2)</td><td>(3)</td><td colspan="4">(4)</td><td>(5)</td><td colspan="6">(6)</td></tr>
<tr><td rowspan="7">iii)</td><td rowspan="7">[C-6A(23)]</td><td rowspan="4">HR1
HR2
HR3
HR4</td><td colspan="4" rowspan="4">—
—
—
—</td><td rowspan="4">440
420
400
380</td><td colspan="2">t ≤ 3</td><td colspan="4">t > 3</td></tr>
<tr><td>GL
$L_0 = 80$ mm</td><td>GL
$L_0 = 50$ mm</td><td colspan="2">GL
$L_0 = 5.65\sqrt{S_0}$</td><td colspan="2">GL
$L_0 = 50$ mm</td></tr>
<tr><td>23
25
28
31</td><td>24
26
29
32</td><td colspan="2">28
30
33
36</td><td colspan="2">29
31
34
37</td></tr>
<tr><td rowspan="3">ISH270C
ISH270D
ISH270E</td><td>t < 2</td><td colspan="2">2 ≤ t < 3.2</td><td>$t \geq 3.2$</td><td rowspan="3">270 to 420
270 to 400
270 to 380</td><td>t < 2</td><td>2 ≤ t < 3.2</td><td>$t \geq 3.2$</td><td>t < 3</td><td>t > 3</td></tr>
<tr><td colspan="4"></td><td colspan="3">GL
Lo = 50 mm</td><td>GL
Lo = 80 mm</td><td>GL
$L0 = 5.65\sqrt{S_0}$</td></tr>
<tr><td colspan="4">170, Min</td><td>26</td><td>26</td><td>31</td><td>25</td><td>30</td></tr>
<tr><td colspan="3">170, Min</td><td>165, Min</td><td>29</td><td>29</td><td>34</td><td>28</td><td>33</td></tr>
<tr><td>165, Min</td><td>155, Min</td><td>145, Min</td><td>32</td><td>32</td><td>37</td><td>31</td><td>36</td></tr>
</table>

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Table 1 (*Continued*)

Sl No.	Accepted Standard	Grade/Classification	Properties		
			Yield Stress, MPa *Min*	Ultimate Tensile Stress MPa *Min*	Elongation, Percent *Min*
(1)	(2)	(3)	(4)	(5)	(6)
iv)	[C-6A(24)]	3.6	180	330	25
		4.6	240	400	22
		4.8	320	420	—
		5.6	300	500	20
		5.8	400	520	—
		6.8	480	600	—
		8.8 ($d \leq 16$ mm)	640 [1)]	800	12
		8.8 ($d > 16$ mm)	660 [1)]	830	12
		9.8	720 [1)]	900	10
		10.9	940 [1)]	1 040	9
		12.9	1 100 [1)]	1 220	8
v)	[C-6A(25)]	1	200	370	26
		1A	220	410	25
		2	230	430	24
		2A	250	460	22
		3	270	490	21
		3A	280	540	20
		4	320	620	15
		5	350	710	13
		6	370	740	10

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Table 1 (*Continued*)

Sl No.	Accepted Standard	Grade/Classification	Properties								
			Yield Stress, MPa *Min*					Ultimate Tensile Stress, MPa *Min*			Elongation, Percent *Min*
(1)	(2)	(3)	(4)					(5)			(6)
vi)	[C-6A(26)]		*d* or *t*								
			≤ 20		> 20						
		S_t 37	220		200			360 to 440			26
		S_t 42	250		240			410 to 500			23
vii)	[C-6A(27)]		*t*					*t*			*t*
			≤ 16	> 16 and ≤ 40	> 40 and ≤ 60	> 60 and ≤ 100	>100 and ≤ 350	≤60	> 60 and ≤ 100	> 100 and ≤ 350	≤ 60	> 60 and ≤ 350
		1	235	225	215	200	185	360 to 480	360 to 480	350 to 480	24	23
		2	265	255	245	215	200	410 to 530	410 to 530	400 to 530	22	21
		3	290	285	280	255	230	460 to 580	450 to 570	440 to 570	21	20

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Table 1 (*Continued*)

Sl No.	Accepted Standard	Grade/Classification	Properties		
			Yield Stress, MPa *Min*	Ultimate Tensile Stress, MPa *Min*	Elongation Percent, *Min*
(1)	(2)	(3)	(4)	(5)	(6)
viii)	[C-6A(6)]		*d* or *t*		
			<20 / 20 to 40 / > 40		
		E250 (A, BR, B0, C)	250 / 240 / 230	410	23
		E275 (A, BR, B0, C)	275 / 265 / 255	430	22
		E300 (A, BR, B0, C)	300 / 290 / 280	440	22
		E350 (A, BR, B0, C)	350 / 330 / 320	490	22
		E410 (A, BR, B0, C)	410 / 390 / 380	540	20
		E450 (A, BR)	450 / 430 / 420	570	20
ix)	[C-6A(28)]				$L_o = 5.65\sqrt{S_o}$
		A	235	400 to 520[2)]	22[3)]
		B	235	400 to 520[2)]	22[3)]
		D	235	400 to 520[2)]	22[3)]
		E	235	400 to 520[2)]	22[3)]
x)	[C-6A(29)]	—	240	350 to 450	25

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Table 1 (*Continued*)

Sl No.	Accepted Standard	Grade/Classification	Properties		
			Yield Stress, MPa *Min*	Ultimate Tensile Stress, MPa *Min*	Elongation Percent, *Min*
(1)	(2)	(3)	(4)	(5)	(6)
xi)	[C-6A(11)]	Annealed Condition	160	330 to 410	30
		As-Drawn Condition	190	410 to 490	20
xii)	[C-6A(30)]	HFC 210/CDS 210/ERW210	210	330	20
		HFC 240/CDS 240/ERW240	240	410	18
		HFC 310/CDS 310/ERW310	310	450	15
xiii)	[C-6A(31)]	1	170	290	30
		2	210	330	28
		3	240	410	25
		4	275	430	20
		5	310	490	15
xiv)	[C-6A(32)]		< 12 / 12 to 25 / 25 to 40 / > 40		
		WR-Fe 480A	345 / 325 / 325 / –	345	21
		WR-Fe 480B	345 / 345 / 345 / 340	345	21
		WR-Fe 500	355 / – / – / –	355	20

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Table 1 (*Concluded*)

Sl No.	Accepted Standard	Grade/Classification	Properties		
			Yield Stress, MPa *Min*	Ultimate Tensile Stress, MPa *Min*	Elongation Percent, *Min*
(1)	(2)	(3)	(4)	(5)	(6)
xv)	[C-6A(33)]		<12 12 to 25 25 to 40 > 40		
		Grade A (FR-Fe 410)	300 280 260 240	410	21
		Grade B (FR-Fe 490)	350 330 310 290	490	18
xvi)	[C-6A(34)]		< 20 20 to 40 > 40		
		E250S	250 240 230	410	23
		E300S	300 290 280	440	22
		E350S	350 330 320	490	22
		E450S	450 430 420	570	20

NOTE — Percentage of elongation shall be taken over the gauge length of $5.65\sqrt{S_0}$, where S_0 = original cross-sectional area of the test specimen.

[1)] Stress at 0.2 percent non-proportional elongation, Min.

[2)] For Grade A sections and bars, the upper limit of the tensile strength may be 550 MPa or as agreed between the purchaser and the manufacturer/the supplier.

[3)] For full thickness flat test pieces with width 25 mm and gauge length 200 mm, the minimum elongation (percent) may be reduced as per specified in good practice [C-6A(**29**)].

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10 GENERAL DESIGN REQUIREMENTS

10.1 Basis for Design

10.1.1 *Design Objective*

The objective of design is the achievement of an acceptable probability that structures will perform satisfactorily for the intended purpose during the design life. With an appropriate degree of safety, they should sustain all the loads and deformations, during construction and use and have adequate resistance to certain expected accidental loads and fire. Structure should be stable and have alternate load paths to prevent disproportionate overall collapse under accidental loading.

10.1.2 *Methods of Design*

Structure and its elements shall normally, be designed by the limit state method. Account should be taken of accepted theories, experimental information and experience and the need to design for durability. Calculations alone may not produce safe, serviceable and durable structures. Suitable materials, quality control, adequate detailing and good supervision are equally important.

10.1.3 *Design Process*

Structural design, including design for durability, construction and use should be considered as a whole. The realization of design objectives requires compliance with clearly defined standards for materials, fabrication, erection and in-service maintenance.

10.2 Loads and Forces

10.2.1 For the purpose of designing any element, member or a structure, the following loads (actions) and their effects shall be taken into account, where applicable, with partial safety factors for loads and their combinations given in Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS (*see* **12.3.3**).

a) Dead loads;

b) Imposed loads (imposed load, crane load, snow load, dust load, wave load, earth pressures, etc);

c) Wind loads;

d) Earthquake effects;

e) Fire Load, forces induced due to restraint against expansion during a fire;

f) Erection loads;

g) Accidental loads such as those due to blasts, impact of vehicles, etc; and

h) Secondary effects due to contraction or expansion resulting from temperature changes, differential settlements of the structure as a whole or of its components, eccentric connections, rigidity of joints differing from design assumptions.

10.2.1.1 Dead loads should be assumed in design as specified in **2** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS.

10.2.1.2 Imposed loads for different types of occupancy and function of structures shall be taken as recommended in **3** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS. Imposed loads arising from equipment, such as cranes and machines should be assumed in design as per manufacturers/suppliers data (*see* **10.6.4**). Snow load shall be taken as per in **6** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS.

10.2.1.3 Wind loads on structures shall be taken as per the recommendations of **4** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS.

10.2.1.4 Earthquake effects shall be assumed as per the recommendations of **5** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS.

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10.2.1.5 The fire loads, erection loads and temperature effects shall be considered as specified in **10.3**, **10.4** and **10.5**, respectively.

10.3 Fire Loads

During a fire, the structural members around the fire experience higher temperatures. This causes expansion in members that is restrained by rest of the structure, inducing forces in the members. Simultaneously, due to high temperature both strength and stiffness of the members experiencing the higher temperature degrade. The forces in members during fire including that due to dead load and imposed load are checked against the reduced capacity of the members. As it is an extreme event, the design is done for loads without extra partial safety factors. The imposed load is taken as 100 percent of the characteristic Imposed loads in case of storage areas and otherwise taken as 50 percent of the characteristic Imposed load value, during fire.

Due to proper fire protection, the temperature felt by the members may be less than the ambient temperature, and this may be taken into consideration in evaluating temperature effects due to fire.

10.4 Erection Loads

All loads required to be carried by the structure or any part of it due to storage or positioning of construction material and erection equipment, including all loads due to operation of such equipment shall be considered as erection loads. Proper provision shall be made, including temporary bracings, to take care of all stresses developed during erection. Dead load, wind load and also such parts of the imposed load as would be imposed on the structure during the period of erection shall be taken as acting together with the erection loads. The structure as a whole and all parts of the structure in conjunction with the temporary bracings shall be capable of sustaining these loads during erection.

10.5 Temperature Effects

10.5.1 Expansion and contraction due to changes in temperature of the members and elements of a structure shall be considered and adequate provision made for such effect.

10.5.2 The temperature range varies for different localities and under different diurnal and seasonal conditions. The absolute maximum and minimum temperatures, which may be expected at the site, may be obtained from the Indian Metrological Department and used in assessing the maximum variations of temperature for which provision for expansion and contraction has to be made in the structure.

10.5.3 The range of variation in temperature of the building materials may be appreciably greater or lesser than the variation of air temperature and is influenced by the condition of exposure and the rate at which the materials composing the structure absorb or radiate heat. This difference in temperature variations of the material and air shall be given due consideration. The effect of differential temperature within an element or member, due to part exposure to direct sunlight shall also be considered.

10.5.4 The coefficient of thermal expansion for steel is as given in **9.2.4.1**(e).

10.6 Load Combinations

10.6.1 Load combinations for design purposes shall be those that produce maximum forces and effects and, consequently maximum stresses and deformations. The combination of loads with appropriate partial safety factors may be as per Part C/Section 1 of this NBCS.

NOTE — In the case of structures supporting cranes, imposed loads shall include the crane effects as given in **10.6.4**.

10.6.2 Wind load and earthquake effects shall not be assumed to act simultaneously. The effect of each shall be considered separately.

10.6.3 The effect of cranes to be considered under imposed loads shall include the vertical loads, eccentricity effects induced by the vertical loads, impact factors, lateral (surge) and longitudinal (horizontal) thrusts, not acting simultaneously across and along the crane rail, respectively [*see* **3** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS].

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10.6.4 The crane loads and their combinations to be considered shall be as indicated by the customer. In the absence of any specific indications, the load combinations shall be in accordance with the provisions in **3** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS or as given below:

a) Vertical loads with full impact from one loaded crane or two cranes in case of tandem operation, together with vertical loads without impact from as many loaded cranes as may be positioned for maximum effect, along with maximum horizontal thrust from one crane only or two in case of tandem operation;

b) Loads as specified in **10.6.4**(a), subject to cranes in maximum of any two bays of the building cross section shall be considered for multi-bay multi-crane gantries;

c) The longitudinal thrust on a crane track rail shall be considered for a maximum of two loaded cranes on the track; and

d) Lateral thrust (surge) and longitudinal thrust acting across and along the crane rail, respectively, shall be assumed not to act simultaneously. The effect of each force, shall however be investigated separately.

10.6.5 While investigating the effect of earthquake, the resulting effect from dead loads of all cranes parked in each bay, positioned to cause maximum effect shall be considered.

10.6.6 The crane runway girders supporting bumpers shall be checked for bumper impact loads also, as specified by the manufacturers.

10.6.7 Stresses developed due to secondary effects such as handling; erection, temperature and settlement of foundations, if any, shall be appropriately added to the stresses calculated from the combination of loads stated in **10.6.1**, with appropriate partial safety factors.

10.7 Geometrical Properties

10.7.1 *General*

The geometrical properties of the gross and the effective cross sections of a member or part thereof, shall be calculated on the following basis:

a) the properties of the gross cross section shall be calculated from the specified size of the member or part thereof or read from appropriate table; and

b) the properties of the effective cross section shall be calculated by deducting from the area of the gross cross section, in the following:

1) The sectional area in excess of effective plate width, in case of slender sections [*see* **10.8.2 (d)**]; and

2) The sectional areas of all holes in the section except for parts in compression. In case of punched holes, hole size 2 mm in excess of the actual diameter may be deducted.

10.8 Classification of Cross Sections

10.8.1 Plate elements of a cross section may buckle locally due to compressive stresses. The local buckling can be avoided before the limit state is achieved by limiting the width to thickness ratio of each element of a cross section subjected to compression due to axial force, moment or shear.

10.8.1.1 When plastic analysis is used, the members shall be capable of forming plastic hinges with sufficient rotation capacity (ductility) without local buckling, to enable the redistribution of bending moment required before formation of the failure mechanism.

10.8.1.2 When elastic analysis is used, the member shall be capable of developing the yield stress under compression without local buckling.

10.8.2 On basis of the above, four classes of sections are defined as follows:

a) *Class* 1 (*Plastic*) — Cross sections, which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of plastic mechanism. The width to thickness ratio of plate elements shall be less than that specified under class 1 (Plastic), in Table 2;

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b) *Class* 2 (*Compact*) — Cross sections, which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local buckling. The width to thickness ratio of plate elements shall be less than that specified under class 2 (compact), but greater than that specified under class 1 (plastic), in Table 2;

c) *Class* 3 (*Semi-compact*) — Cross sections, in which the extreme fiber in compression can reach yield stress, but cannot develop the plastic moment of resistance, due to local buckling. The width to thickness ratio of plate elements shall be less than that specified under class 3 (semi-compact), but greater than that specified under class 2 (compact), in Table 2; and

d) *Class* 4 (*Slender*) — Cross sections in which the elements buckle locally even before reaching yield stress. The width to thickness ratio of plate elements shall be greater than that specified under class 3 (semi-compact), in Table 2. In such cases, the effective sections for design shall be calculated either by following the provisions of good practice [C-6A(35)] to account for the post-local-buckling strength or by conservatively deducting width of the compression plate element in excess of the semi-compact section limit [*see* Fig. 2(C)].

The design of slender web elements in flexural members may be done as given in **15.2.1.1** for flexure and **15.4.2.2** for shear.

When different elements of a cross section fall under different classes, the section shall be classified as governed by the most critical element.

The maximum value of limiting width to thickness ratios of elements for different classifications of sections are given in Fig. 2(A). The effective section in case of class 4 (slender elements) may be conservatively taken as shown in Fig. 2(C).

10.8.3 *Types of Elements*

a) *Internal elements* — These are elements attached along both longitudinal edges to other elements or to longitudinal stiffeners connected at suitable intervals to transverse stiffeners; for example, web of I-section and flanges and web of box section;

b) *Outside elements or outstands* — These are elements attached along only one of the longitudinal edges to an adjacent element, the other edge being free to displace out of plane, for example flange overhang of an I-section, stem of T-section and legs of an angle section; and

c) *Tapered elements* — These may be treated as flat elements having average thickness as defined in accepted standard [C-6A(36)].

The design of slender compression element (Class 4) considering the strength beyond elastic local buckling of element is outside the scope of this section. Reference may be made to in good practice [C-6A(35)] for such design provisions. The design of slender web elements may be made as given in **15.2.1.1** for flexure and **15.4.2.2** for shear.

10.8.4 *Compound Elements in Built-up Section* (*see* Fig. 2)

In case of compound elements consisting of two or more elements bolted or welded together, the limiting width to thickness ratios as in Table 2 should be considered on basis of the following:

a) Outstanding width of compound element (b_e) to its own thickness;

b) The internal width of each added plate between the lines of welds or fasteners connecting it to the original section to its own thickness; and

c) Any outstand of the added plates beyond the line of welds or fasteners connecting it to original section to its own thickness.

10.9 Maximum Effective Slenderness Ratio

The maximum effective slenderness ratio, KL/r, values of a beam, strut or tension member shall not exceed those given in Table 3. 'KL' is the effective length of the member and 'r' is appropriate radius of gyration based on the effective section as defined in **10.8**.

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Table 2 Limiting Width to Thickness Ratio

(*Foreword* and *Clauses* 10.8.2(a), 10.8.2(b), 10.8.2(c), 10.8.2(d),10.8.4, 11.5.2.2, 14.3.1 *and* 15.6.3.1)

Sl No.	Compression Element			Ratio	Class of Section		
					Class 1 Plastic	Class 2 Compact	Class 3 Semi-compact
(1)	(2)			(3)	(4)	(5)	(6)
i)	Outstanding element	Under uniform compression		b/t_f	8.7ε	9.7ε	13.6ε (a in Fig. 2C)
		Non uniform stress	Tip in compression	b/t_f	$\frac{8.7\,\varepsilon}{r_1}$	$\frac{9.7\,\varepsilon}{r_1}$	$21\varepsilon\sqrt{k_\sigma}$
			Tip in tension	b/t_f	$\frac{8.7\,\varepsilon}{r_1\sqrt{r_1}}$	$\frac{9.7\,\varepsilon}{r_1\sqrt{r_1}}$	$21\varepsilon\sqrt{k_\sigma}$
ii)	Web of I, Channel, H, box or RHS	Axial compression		d/t_w	27ε	33 ε	37ε (b in Fig. 2C)
		Non uniform stress	Neutral axis at mid-depth	d/t_w	70ε	80ε	117ε
			$r_1 > 0.5$ (or) $r_2 > -1$	d/t_w	$\frac{122\,\varepsilon}{5.5\,r_1 - 1}$	$\frac{182\,\varepsilon}{6.53\,r_1 - 1}$	$\frac{37\,\varepsilon}{0.608 + 0.343\,r_2 + 0.049\,r_2^2}$
			$r_1 \leq 0.5$ (or) $r_2 \leq -1$	d/t_w	$\frac{35\,\varepsilon}{r_1}$	$\frac{40\,\varepsilon}{r_1}$	$59\,\varepsilon\,(1 - r_2)$
iii)	Angle, compression due to bending (Both criteria should be satisfied)			b/t d/t	8.7 ε 8.7 ε	9.7 ε 9.7 ε	14.5 ε 14.5 ε
iv)	Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)			b/t d/t $(b+d)/t$	Not applicable		14.5 ε 14.5 ε 22.3 ε
v)	Outstanding leg of an angle in contact back-to-back in a double angle member or with another component			d/t	8.7ε	9.7ε	14.5 ε
vi)	Circular, elliptical hollow tube (*see* Note 6), including welded tube	a) bending moment		D_e/t	48 ε^2	68 ε^2	87 ε^2
		b) axial compression		D_e/t	Not applicable		87 ε^2

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NOTES

1 Elements which exceed semi-compact limits are to be taken as of slender cross section

2 $\varepsilon = \sqrt{(250 / fy)}$

3 Webs shall be checked for shear buckling in accordance with **15.4.2** when $d/t > 67\varepsilon$. Where, b is the width of the element may be taken as clear distance between lateral supports or between lateral support and free edge, as appropriate, t is the thickness of element, d is the depth of the web, D is the outer diameter of the element (*see* Fig 2, **10.8.3** and **10.8.4**).

4 Different elements of a cross section can be in different classes. In such cases the section is classified based on the least favourable classification.

5 The stress ratio r_1 and r_2 are defined as:

$$r_1 = \frac{\text{depth under compression}}{\text{Overall depth}}$$

$$r_2 = \frac{\text{Minimum compressive stress (tensile if negative)}}{\text{Maximum design compressive stress of web alone}}$$

6 (a) for circular sections $D_e = D$,

(b) Elliptical sections: (h = major axis, b = minor axis)

in compression: $D_e = h\left[1 + \left(1 - 2.3\left(\frac{t}{h}\right)^{0.6}\right)\left(\frac{h}{b} - 1\right)\right]$

conservatively $D_e = \frac{h^2}{b}$

In bending about the strong (minor) axis

For h/b ≤ 1.36: $D_e = \frac{b^2}{h}$

For h/b > 1.36: $D_e = 0.4\,\frac{h^2}{b}$

In compression and or bending about weak axis:

$D_e = \frac{h^2}{b}$

7 k_σ is the buckling factor given by:

Maximum compression at free edge	r_2	1	0	–1	–1 to – 3
	K_σ	0.43	0.57	0.85	$0.57 - 0.21k_2 + 0.07r_2$
Maximum Compression at supported edge	r_2	1	$0 < r_2 < 1$	0	$-1 < r_2 < 0$
	K_σ	0.43	$0.578/(r_2 + 0.34)$	1.70	$1.7 - 5\,r_2 + 17.1\,r_2^2$

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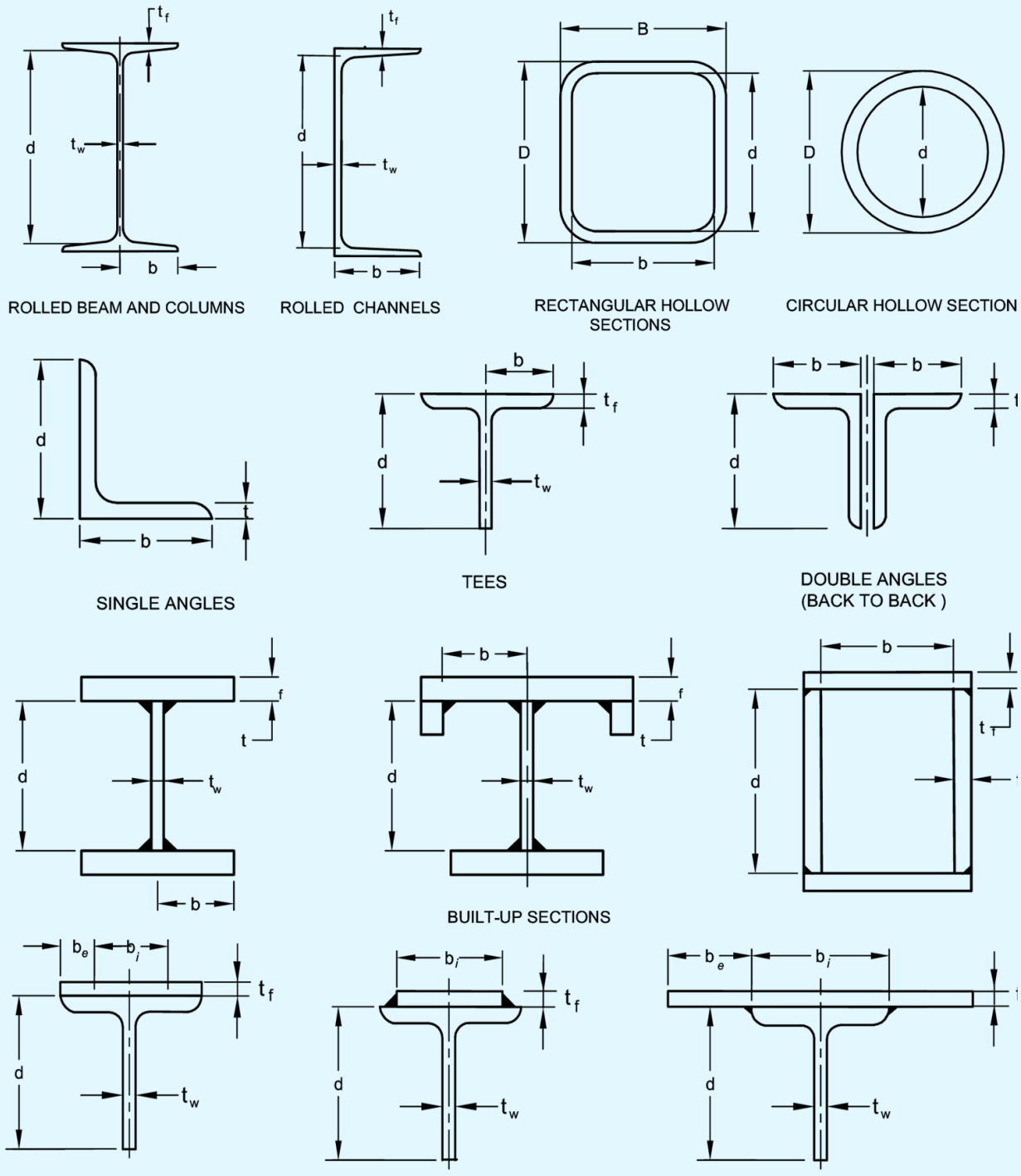

b_i – INTERNAL ELEMENT WIDTH
b_e – EXTERNAL ELEMENT WIDTH

FIG. 2(A) DIMENSIONS OF SECTIONS

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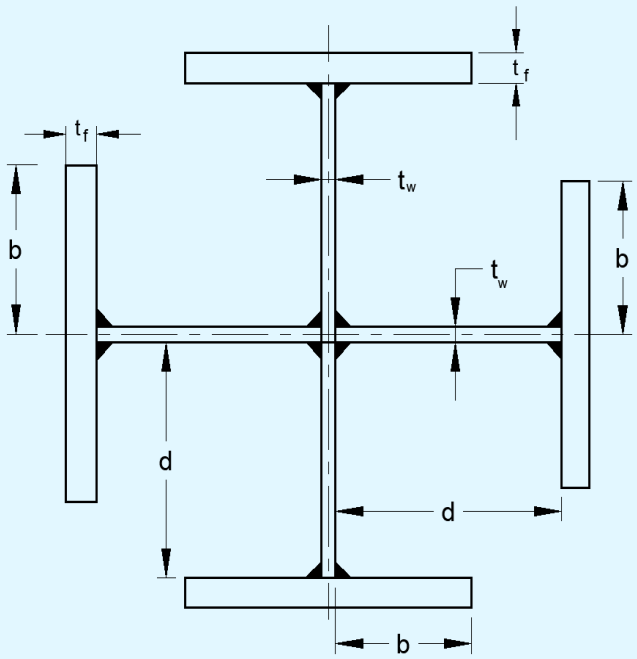
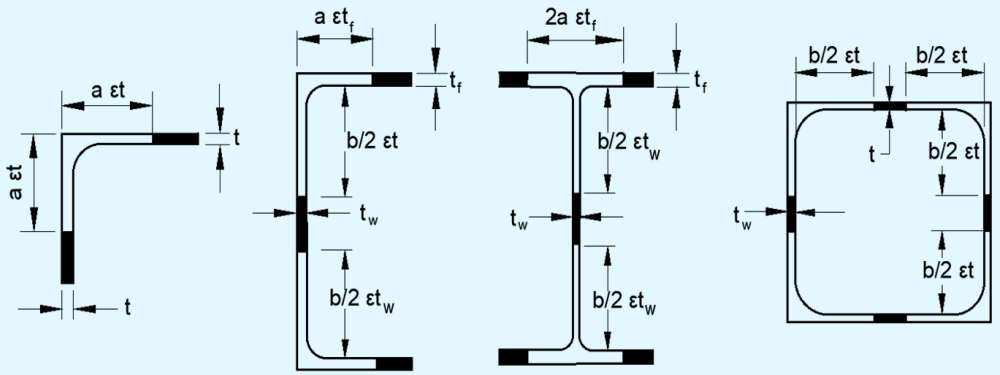


FIG. 2(B) BUILT-UP STEEL COLUMN CRUCIFORM, FLANGED



VALUES OF (a) AND (b) ARE TO BE TAKEN FROM TABLE 2.

FIG. 2(C) EFFECTIVE SECTION OF SLENDER ELEMENTS

FIG.2 CLASSIFICATION OF SECTIONS

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Table 3 Maximum Values of Effective Slenderness Ratios

(*Clause* 10.9)

Sl No.	Member	Maximum Effective Slenderness Ratio *KL/r*
(1)	(2)	(3)
i)	A member carrying compressive loads resulting from dead loads and imposed loads	180
ii)	A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces	180
iii)	A member subjected to compression forces resulting only from combination with wind/earthquake effects, provided the deformation of such member does not adversely affect the stress in any part of the structure	250
iv)	Compression flange of a beam against lateral torsional buckling	300
v)	A member normally acting as a tie in a roof truss or a bracing system not considered effective when subject to possible reversal of stress into compression resulting from the action of wind or earthquake effects[1)]	350
vi)	Members always under tension[1)] (other than pre-tensioned members)	400

[1)] Tension members, such as bracings, where sag is avoided, need not satisfy the maximum slenderness ratio limits.

10.10 Resistance to Horizontal Forces

10.10.1 In designing the steel frame work of a building, provision shall be made (by adequate moment connections or by a system of bracing) to effectively transmit to the foundations all the horizontal forces, giving due allowance for the stiffening effect of the walls and floors, where applicable.

10.10.2 When the walls, or walls and floors and/or roofs are capable of effectively transmitting all the horizontal forces directly to the foundations, the structural steel framework may be designed without considering the effects of wind or earthquake.

10.10.3 Wind and earthquake effects are reversible and therefore call for initial stiffness and strength under force reversal in both longitudinal and transverse directions. To resist torsional effects of wind and earthquake forces, bracings in plan should be provided and integrally connected with the longitudinal and transverse bracings, to impart adequate torsional resistance to the structure.

10.10.3.1 In shed type steel mill buildings, adequate bracings shall be provided to transfer the wind or earthquake loads from their points of action to the appropriate supporting members. Where the connections to the interior columns or frames are designed such that the wind or earthquake loads will not be transferred to the interior columns, the exterior columns or frames shall be designed to resist the total wind or earthquake loads. Where the connections to the interior columns and frames are designed such that the wind or earthquake effects are transferred to the interior columns also, and where adequate rigid diaphragm action can be mobilized as in the case of the cast-in place RC slab, both exterior and interior columns and frames may be designed on the assumption that the wind or earthquake load is divided among them in proportion to their relative stiffness. Columns also should be designed to withstand the net uplifting effect caused by excessive wind or earthquake shaking. Additional axial forces arising in adjacent columns due to the vertical component of bracings or due to frame action shall also be accounted for.

10.10.3.2 Earthquake forces are proportional to the seismic mass as defined in 5 of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS. Earthquake forces should be applied at the centre of

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mass of all such components and their transfer to the foundation should be ensured. Other construction details, stipulated in good practice [C-6A(37)] should also be followed.

10.10.3.3 In buildings where high-speed traveling cranes are supported or where a building or structure is otherwise subjected to vibration or sway, triangulated bracing or rigid portal systems shall be provided to reduce the vibration or sway to an acceptable minimum.

10.10.4 *Foundations*

The foundations of a building or other structures shall be designed to provide the rigidity and strength that has been assumed in the analysis and design of the superstructure.

10.10.5 *Eccentrically Placed Loads*

Where a wall, or other gravity load, is placed eccentrically upon the flange of a supporting steel beam, the beam and its connections shall be designed for torsion, unless the beam is restrained laterally in such a way as to prevent the twisting of the beam.

10.11 Expansion Joints

Expansion joints in case of low-rise buildings (up to three storeys) shall comply with **10.11.1** to **10.11.4**. For requirement of expansion joints in high-rise buildings, specialist literature shall be referred to.

10.11.1 In view of the large number of factors involved in deciding the location, spacing and nature of expansion joints, the decision regarding provision of expansion joints shall be left to the discretion of the designer.

10.11.2 Structures in which marked changes in plan dimensions take place abruptly, shall be provided with expansion joints at the section where such changes occur. Expansion joints shall be so provided that the necessary movement occurs with minimum resistance at the joint. The gap at the expansion joint should be such that:

a) it accommodates the expected expansion/contraction due to seasonal and diurnal variation of temperature; and

b) it avoids pounding of adjacent units under earthquake shaking. The structure adjacent to the joint should preferably be supported on separate columns but not necessarily on separate foundations.

10.11.3 The details as to the length of a structure where expansion joints have to be provided may be determined after taking into consideration various factors such as temperature, exposure to weather and structural design. The provisions in **10.11.3.1** to **10.11.3.3** are given as general guidance. The provision for expansion may be made by providing double framing at the joint or providing for expansion at the connection to the single framing in detailing.

10.11.3.1 If one bay of longitudinal bracing is provided at the centre of the building or building section, the length of the building section may be restricted to 180 m in case of covered buildings and 120 m in case of open gantries (*see* Fig. 3).

10.11.3.2 If more than one bay of longitudinal bracing is provided near the centre of the building/section, the maximum centre line distance between the two lines of bracing may be restricted to 50 m for covered buildings (and 30 m for open gantries) and the maximum distance between the centre of the bracing to the nearest expansion joint/end of building or section may be restricted to 90 m (60 m in case of open gantries). The maximum length of the building section thus may be restricted to 230 m for covered buildings (150 m for open gantries). Beyond this, suitable expansion joints shall be provided (*see* Fig.4).

10.11.3.3 The maximum width of the covered building section should preferably be restricted to 150 m beyond which suitable provisions for the expansion joint may be made.

10.11.4 When the provisions of these sections are met for a building or open structure made only of steel frame with elastic steel claddings, the stress analysis due to temperature is not required. However, for buildings, specialist literature may be referred to decide on thermal stress design.

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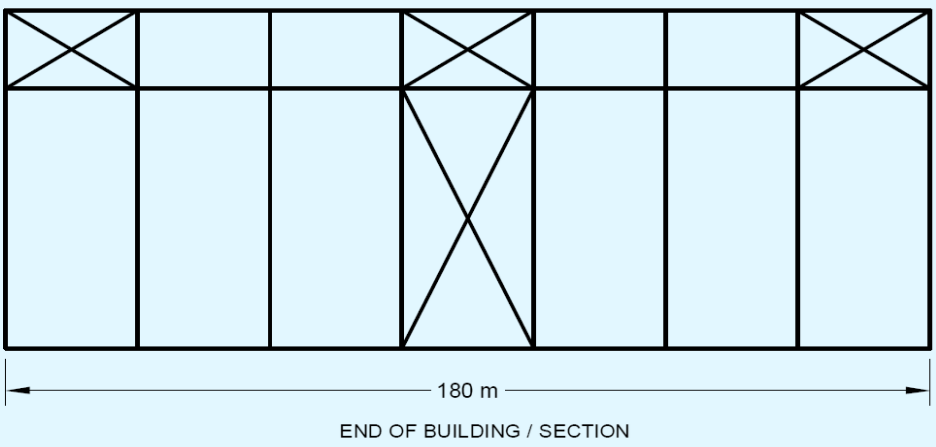

FIG. 3 MAXIMUM LENGTH OF BUILDING WITH ONE BAY OF BRACING

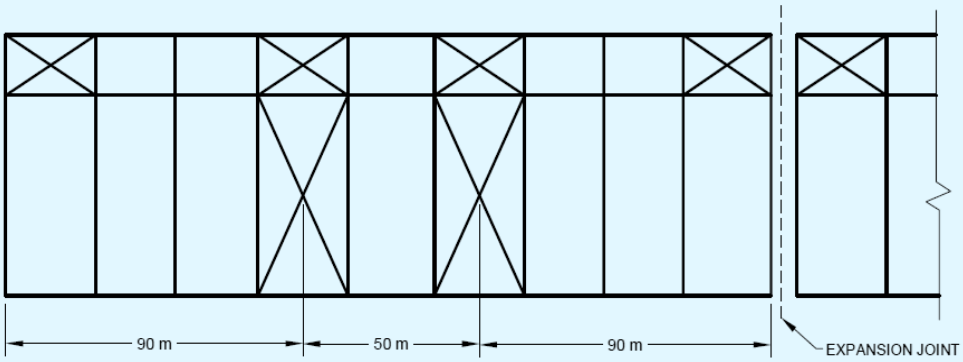

FIG. 4 MAXIMUM LENGTH OF COVERED BUILDING/SECTION WITH TWO BAYS OF BRACINGS

11 METHODS OF STRUCTURAL ANALYSIS

11.1 Methods of Determining Action Effects

11.1.1 *General*

For the purpose of complying with the requirements of the limit states of stability, strength and serviceability specified under Section **12** on the effects of design actions on a structure and its members and connections, shall be determined by structural analysis using the assumptions of **11.2** and **11.3** and one of the following methods of analysis:

a) elastic analysis in accordance with **11.4**;

b) plastic analysis in accordance with **11.5**;

c) advanced analysis in accordance with Annex B; or

d) dynamic analysis in accordance with **5** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS.

The effects of design earthquake shaking specified in Section **18** shall be estimated by elastic analysis. Plastic analysis shall not be employed to obtain member forces corresponding to design earthquake shaking specified in Section **18**. More information on analysis to resist earthquake effects is given in **5** of Part C 'Structural Design/ Section 1 Loads and Load Effects' of this NBCS.

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11.1.2 *Non-sway and Sway Frames*

For the purpose of analysis and design, the structural frames are classified as non-sway and sway frames as given below:

a) *Non-sway frame* — One in which the transverse displacement of one end of the member relative to the other end is effectively prevented. This applies to triangulated frames and trusses or to frames where in-plane stiffness is provided by bracings, or by shear walls, or by floor slabs and roof decks secured horizontally to walls or to bracing systems parallel to the plane of loading and bending of the frame;

b) *Sway frame* — One in which the transverse displacement of one end of the member relative to the other end is not effectively prevented. Such members and frames occur in structures which depend on flexural action of members to resist lateral loads and sway, as in moment resisting frames; and

c) A rigid jointed multi-storey frame may be considered as a non-sway frame if in every individual storey, the deflection δ, over a storey height h_s, due to the notional horizontal loading given in **11.3.6** satisfies the following criteria:

1) For clad frames, when the stiffening effect of the cladding is not taken into account in the deflection calculations:

$$\delta \leq \frac{h_s}{2\,000}$$

2) For unclad frame or clad frames, when the stiffening effect of the cladding is taken into account in the deflection calculations:

$$\delta \leq \frac{h_s}{4\,000}$$

3) A frame, which when analyzed considering all the lateral supporting system does not comply with the above criteria, should be classified as a sway frame, even if it is braced or otherwise laterally stiffened.

11.2 Forms of Construction Assumed for Structural Analysis

11.2.1 The effects of design action in the members and connections of a structure shall be determined by assuming singly or in combination of the following forms of construction (*see* **17.6.1**).

11.2.1.1 *Rigid construction*

In rigid construction, the connections between members (beam and column) at their junction shall be assumed to have sufficient rigidity to hold the original angles between the members connected at a joint unchanged under loading.

11.2.1.2 *Semi-rigid construction*

In semi-rigid construction, the connections between members (beam and column) at their junction may not have sufficient rigidity to hold the original angles between the members at a joint unchanged, but shall be assumed to have the capacity to furnish a dependable and known degree of flexural restraint. The relationship between the degree of flexural restraint and the level of the load effects shall be established by any rational method or based on test results (*see* Annex F).

11.2.1.3 *Simple construction*

In simple construction, the connections between members (beam and column) at their junction will not resist any appreciable moment and shall be assumed to be hinged.

11.2.2 *Design of connections*

The design of all connections shall be consistent with the form of construction, and the behaviour of the connections shall not adversely affect any other part of the structure beyond what is allowed for in design. Connections shall be designed in accordance with provisions under **17**.

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11.3 Assumptions in Analysis

11.3.1 The structure shall be analyzed in its entirety except as follows:

a) Regular building structures, with orthogonal frames in plan, may be analyzed as a series of parallel two-dimensional sub-structures (part of a structure), the analysis being carried out in each of the two directions, at right angles to each other, except when there is significant load redistribution between the sub-structures (part of a structure);

b) For vertical loading in a multi-storey building structure, provided with bracing or shear walls to resist all lateral forces, each level thereof, together with the columns immediately above and below, may be considered as a substructure, the columns being assumed fixed at the ends remote from the level under consideration; and

c) Where beams at a floor level in a multi-bay building structure are considered as a substructure (part of a structure), the bending moment at the support of the beam due to gravity loads may be determined based on the assumption that the beam is fixed at the far end support, one span away from the span under consideration, provided that the floor beam is continuous beyond that support point.

11.3.2 *Span Length*

The span length of a flexural member in a continuous frame system shall be taken as the centre-to-centre distance between the supports.

11.3.3 *Arrangements of Imposed Loads in Buildings*

For building structures, the various arrangements of imposed loads considered for the analysis, shall include at least the following:

a) Where the loading pattern is fixed, the arrangement concerned;

b) Where the imposed load is variable and not greater than three-quarters of the dead load, the imposed load may be taken to be acting on all spans; and

c) Where the imposed load is variable and exceeds three-quarters of the dead load, arrangements of imposed load acting on the floor under consideration shall include the following cases:

   1) imposed load on alternate spans;

   2) imposed load on two adjacent spans; and

   3) imposed load on all the spans.

11.3.4 *Base Stiffness*

In the analysis of all structures, the appropriate base stiffness about the axis under consideration shall be used. In the absence of the knowledge of the pedestal and foundation stiffness, the following may be assumed:

a) When the column is rigidly connected to a suitable foundation, the stiffness of the pedestal shall be taken as the stiffness of the column above base plate. However, in case of very stiff pedestals and foundations the column may be assumed as fixed at base;

b) When the column is nominally connected to the foundation, a pedestal stiffness of 10 percent of the column stiffness may be assumed;

c) When an actual pin or rocker is provided in the connection between the steel column and pedestal, the column is assumed as hinged at base and the pedestal and foundation may be appropriately designed for the reactions from the column; and

d) In case of (a) and (b), the bottom of the pedestal shall be assumed to have the following boundary condition in the absence of any detailed procedure based on theory or tests:

   1) When the foundation consists of a group of piles with a pile cap, raft foundation or an isolated footing resting on rock or very hard soil, the pedestal shall be assumed to be fixed at the level of the bottom of footing or at the top of the pile cap. Pedestal to be considered as fixed at top of pile cap, where

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the Single Piles are tied at Pile cap level by means of ground beams (in both orthogonal directions) with required stiffness;

2) When the foundation consists of an isolated footing resting on other soils, the pedestal shall be assumed to be hinged at the level of the bottom of the footing;

3) When the pedestal is supported by a single pile, which is laterally surrounded by soil providing passive resistance, the pile shall be assumed to be fixed at a depth of 5 times the diameter of the pile below the ground level in case of compact ground or the top level of compact soil in case of poor soil overlying compact soil. Single pile cap, connected flexurally to orthogonal beams having adequate stiffness, shall be assumed to be fixed at the top of pile cap; and

4) When the column is founded into rock, it may be assumed to be fixed at the interface of the column and rock.

11.3.5 *Simple Construction*

Bending members may be assumed to have their ends connected for shear only and to be free to rotate. In triangulated structures, axial forces may be determined by assuming that all members are pin-connected. The eccentricity for stanchion and column shall be assumed in accordance with **14.3.3**.

11.3.6 *Notional Horizontal Loads*

To analyze a frame subjected to gravity loads, considering the sway stability of the frame, notional horizontal forces should be applied. These notional horizontal forces account for practical imperfections and should be taken at each level as being equal to 0.5 percent of factored dead load plus vertical imposed loads applied at that level. The notional load should not be applied along with other lateral loads, such as wind and earthquake effects, in the analysis.

11.3.6.1 The notional forces should be applied on the whole structure, in both orthogonal directions, in one direction at a time, at roof and all floor levels or their equivalent. They should be taken as acting simultaneously with factored gravity loads.

11.3.6.2 The notional force should not be:

a) applied when considering overturning or overall instability;

b) combined with other horizontal (lateral) loads;

c) combined with temperature effects; and

d) taken to contribute to the net shear on the foundation.

11.3.6.3 The sway effect using notional load under gantry load case need not be considered if the ratio of height to lateral width of the building is less than unity.

11.4 Elastic Analysis

11.4.1 *Assumptions*

Individual members shall be assumed to remain elastic under the action of the factored design loads for all limit states.

The effect of haunching or any variation of the cross section along the axis of a member shall be considered, and where significant, shall be taken into account in the determination of the member stiffness.

11.4.2 *First-order Elastic Analysis*

In a first-order elastic analysis, the equilibrium of the frame in the undeformed geometry is considered, the changes in the geometry of the frame due to the loading are not accounted for, and changes in the effective stiffness of the members due to axial force are neglected. The conditions under which the first order elastic analysis results are adequate for design are given in Section **11.7**.

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11.5 Plastic Analysis

11.5.1 *Application*

The effects of design action throughout or on part of a structure may be determined by a plastic analysis, provided that the requirements of **11.5.2** are met. The distribution of design action effects shall satisfy equilibrium and the boundary conditions.

11.5.2 *Requirements*

When a plastic method of analysis is used, all of the following conditions shall be satisfied, unless adequate ductility of the structure and plastic rotation capacity of its members and connections are established for the design loading conditions by other means of evaluation:

a) The yield stress of the grade of the steel used shall not exceed 450 MPa;

b) The stress-strain characteristics of the steel shall not be significantly different from those obtained for steels complying with the accepted standard [C-6A(6)] or equivalent and shall be such as to ensure complete plastic moment redistribution. The stress-strain diagram shall have a plateau at the yield stress, extending for at least six times the yield strain. The ratio of the tensile strength to the yield stress specified for the grade of the steel shall not be less than 1.2. The elongation on a gauge length complying with the accepted standard [C-6A(6)] shall not be less than 15 percent; and the steel shall exhibit strain-hardening capability. Steels conforming to the accepted standard [C-6A(6)] shall be deemed to satisfy the above requirements;

c) The members used shall be hot-rolled or fabricated using hot-rolled plates and sections;

d) The cross section of members not containing plastic hinges should be at least that of compact section (*see* **10.8.2**), unless the members meet the strength requirements from elastic analysis;

e) Where plastic hinges occur in a member, the proportions of its cross section should not exceed the limiting values for plastic section given in **10.8.2**;

f) The cross section should be symmetrical about its axis perpendicular to the axis of the plastic hinge rotation; and

g) The members shall not be subject to impact loading, requiring fracture assessment or fluctuating loading, requiring a fatigue assessment [*see* **19**].

11.5.2.1 *Restraints*

If practicable, torsional restraint (against lateral buckling) should be provided at all plastic hinge locations. Where not feasible, the restraint should be provided within a distance of $D/2$ of the plastic hinge location, where D is the total depth of section.

The torsional restraint requirement at a section as above, need not be met at the last plastic hinge to form, provided it can be clearly identified.

Within a member containing a plastic hinge, the maximum distance L_m from the restraint at the plastic hinge to an adjacent restraint should be calculated by any rational method or the conservative method given below, so as to prevent lateral buckling.

Conservatively L_m (in mm) may be taken as:

$$L_m \leq \frac{38 r_y}{\left[\frac{f_c}{130} + \left(\frac{f_y}{250}\right)^2 \left(\frac{x_t}{40}\right)^2\right]^{1/2}}$$

where

A = area of cross section;

f_c = actual, in N/mm^2, compressive stress on the cross section due to axial load;

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f_y = yield, in N/mm^2, stress;

I_w, I_y, I_t = warping constant, second moment of the cross section about the minor axes and St. Venant's torsion constant, respectively;

r_y = radius, in mm, of gyration about the minor axis; and

x_t = torsional index, $x_t = 1.132(AI_w / I_y I_t)^{0.5}$.

Where the member has unequal flanges, r_y should be taken as the lesser of the values of the compression flange only or the whole section.

Where the cross section of the member varies within the length L_m, the maximum value of r_y and the minimum value of x_t should be used.

The spacing of restraints to member lengths not containing a plastic hinge should satisfy the recommendations of section on lateral buckling strength of beams (*see* **15.2.2**). Where the restraints are placed at the limiting distance L_m, no further checks are required.

11.5.2.2 *Stiffeners at plastic hinge locations*

Web stiffeners should be provided where a concentrated load, which exceeds 10 percent of the shear capacity of the member, is applied within *D/2* of a plastic hinge location, (*see* **15.2.1.2**). The stiffener should be provided within a distance of half the depth of the member on either side of the hinge location and be designed to carry the applied load in accordance with **15.7.4**. If the stiffeners are flat plates, the outstand width to the thickness ratio, *b*/*t*, should not exceed the values given in the plastic section (*see* **10.7** and Table 2). Where other sections are used, the ratio $\left(\frac{I_{so}}{I_t}\right)^{1/2}$, should not exceed the values given for plastic section (for simple outstand, as in **10.7**);

where

I_{so} = second moment of area of the stiffener about the face of the element perpendicular to the web; and

I_t = St. Venant's torsion constant of the stiffener.

11.5.2.3 The frame shall be adequately supported against sway and out-of-plane buckling, by bracings, moment resisting frame or an independent system such as shear wall.

11.5.2.4 *Fabrication restriction*

Within a length equal to the member depth, on either side of a plastic hinge location, the following restrictions should be applied to the tension flange and noted in the design drawings. Holes, if required, should be drilled or else punched 2 mm undersize and reamed. All sheared or hand flame cut edges should be finished smooth by grinding, chipping or planning.

11.5.3 *Assumptions in Analysis*

The design action effects shall be determined using a rigid-plastic analysis.

It shall be permissible to assume full strength or partial strength connections, provided the capacities of these are used in the analysis, and provided that:

a) In a full strength connection, the moment capacity of the connection shall be not less than that of the member being connected;

b) In a partial strength connection, the moment capacity of the connection may be less than that of the member being connected; and

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c) In both cases the behavior of the connection shall be such as to allow all plastic hinges necessary for the collapse mechanism to develop, and shall be such that the required plastic hinge rotation does not exceed the rotation capacity at any of the plastic hinges in the collapse mechanism.

In the case of building structures, it is not normally necessary to consider the effect of alternating plasticity.

11.6 Frame Buckling Analysis

11.6.1 The elastic buckling load factor (λ_{cr}) shall be the ratio of the elastic buckling load set of the frame to the design load set for the frame, and shall be determined in accordance with **11.6.2**.

NOTE — The value of λ_{cr} depends on the load set and has to be evaluated for each possible set of load combination.

11.6.2 *In-plane frame buckling*

The elastic buckling load factor (λ_{cr}) of a rigid-jointed frame shall be determined by using:

a) One of the approximate methods of **11.6.2.1** and **11.6.2.2**; or

b) A rational elastic buckling analysis of the whole frame.

11.6.2.1 *Regular non-sway frames* (*see* **11.1.2**)

In a rectangular non-sway frame with regular loading and negligible axial forces in the beams, the Euler buckling stress $f_{cr,}$ for each column shall be determined in accordance with **14.1.2.1**. The elastic buckling load factor (λ_{cr}) for the whole frame shall be taken as the lowest of the ratio of (f_{cr}/f_{cd}) for all the columns, where f_{cd} is the axial compressive stress in the column from the factored load analysis.

11.6.2.2 *Regular sway frames*

In a rectangular sway frame with regular loading and negligible axial forces in the beams, the buckling load, P_{cr}, for each column shall be determined as $P_{cr} = A\, f_{cr}$, where f_{cr} is the elastic buckling stress of the column in the plane of frame, obtained in accordance with **14.1.2.1**. The elastic buckling load factor $\lambda_{cr,}$ for the whole frame shall be taken as the lowest of all the ratios, λ_{scr}, calculated for each storey of the building, as given below:

$$\lambda_{scr} = \frac{\sum (P_{cr} / L)}{\sum (P / L)}$$

where

L = column length and the summation includes all columns in the plane frame within a storey; and

P = member axial force from the factored load analysis, with tension taken as negative.

11.7 Consideration of Second Order Effects

11.7.1 Any second-order effects of the loads acting on the structure in its deformed configuration may be neglected and first order elastic analysis or plastic analysis results may be used provided the following are satisfied:

a) for clad frames, provided with the stiffening effects of masonry infill wall panels or diaphragms of profiled wall panel is not taken into account, and where elastic buckling load factor, λ_{cr} (*see* **11.6**) satisfies $\lambda_{cr}/\lambda_P \geq 10$;

If $10 > \lambda_{cr}/\lambda_P \geq 4.6$ the second-order effects may be considered by amplifying the design load effects obtained from elastic/plastic analysis by a factor as given in **11.7.3.1**.

If $\lambda_{cr}/\lambda_P < 4.6$, second-order elastic analysis or second-order plastic analysis (*see* **11.7.3**) is to be carried out.

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b) for un-clad frames or for clad frames where the stiffening effects of masonry infill or diaphragms of profiled wall panel is taken into account, where elastic buckling load factor, λcr (see 11.6) satisfies $\lambda cr/ \lambda_P \geq 20$.

If $20 > \lambda_{cr}/ \lambda_P \geq 5.75$ the second-order effects may be considered by amplifying the design load effects obtained from plastic analysis by a factor as given in **11.7.3.2**.

If $\lambda_{cr}/\lambda_P < 5.75$, second-order elasto-plastic analysis or second-order elastic analysis (*see* **11.7.2**) shall be carried out.

where

λ_p = Ultimate load factor from elastic or plastic analysis.

11.7.2 *Second-order Elastic Analysis*

11.7.2.1 The analysis shall allow for the effects of the design loads acting on the structure and its members in their displaced and deformed configuration. These second-order effects shall be taken into account by using either:

a) a first-order elastic analysis with moment amplification in accordance with **11.7.3.1**, **11.7.3.2**, provided the moment amplification factors, K_y and K_z are not greater than 1.4; or

b) a second-order elastic analysis in accordance with Annex B.

11.7.3 Moment Amplification

11.7.3.1 *Moment amplification for members in non-sway frames*

For a member with zero axial compression or a member subject to axial tension, the design bending moment is that obtained from the first-order analysis for factored loads, without any amplification.

For a braced member with a design axial compressive force P_d as determined by the first-order analysis, the design bending moment shall be calculated considering moment amplification as in **16.3.2.2**.

11.7.3.2 *Moment amplification for members in sway frames*

The design bending moment shall be calculated as the product of moment amplification factor, [*see* **16.3.2.2** (K_y, K_z)] and the moment obtained from the first-order analysis of the sway frame, unless analysis considering second order effects is carried out (*see* **11.7.2**).

11.7.4 The calculated bending moments from the first order elastic analysis may be modified by redistribution up to 15 percent of the peak calculated moment of the member under factored load, provided that:

a) in which the moments are reduced shall belong to plastic or compact section classification (*see* **10.8**). the internal forces and moments in the members of the frame are in equilibrium with applied loads; and

b) all the members.

12 LIMIT STATE DESIGN

12.1 Basis for Design

12.1.1 In the limit state design method, the structure shall be designed to withstand safely all loads likely to act on it throughout its life. It shall not suffer total collapse under accidental loads such as from explosions or impact or due to consequences of human error to an extent beyond the local damages. The objective of the design is to achieve a structure that will remain fit for use during its life with acceptable target reliability. In other words, the probability of a limit state being reached during its lifetime should be very low. The acceptable limit for the safety and serviceability requirements before failure occurs is called a limit state. In general, the structure shall be designed on the basis of the most critical limit state and shall be checked for other limit states.

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12.1.2 Steel structures are to be designed and constructed to satisfy the design requirements with regard to stability, strength, serviceability, brittle fracture, fatigue, fire, and durability such that they meet the following:

a) Remain fit with adequate reliability and be able to sustain all actions (loads) and other influences experienced during construction and use;

b) Have adequate durability under normal maintenance; and

c) Do not suffer overall damage or collapse disproportionately under accidental events like explosions, vehicle impact or due to consequences of human error to an extent beyond local damage. The potential for catastrophic damage shall be limited or avoided by appropriate choice of one or more of the following:

   1) Avoiding, eliminating or reducing exposure to hazards, which the structure is likely to sustain;

   2) Choosing structural forms, layouts and details and designing such that;

      i) the structure has low sensitivity to hazardous conditions; and

      ii) the structure survives with only local damage even after serious damage to any one individual element by the hazard.

   3) Choosing suitable material, design and detailing procedure, construction specifications, and control procedures for shop fabrication and field construction as relevant to the particular structure.

12.1.3 The following conditions may be satisfied to avoid a disproportionate collapse:

a) The building should be effectively tied together at each principal floor level and each column should be effectively held in position by means of continuous ties (beams) nearly orthogonal, except where the steel work supports only cladding weighing not more than 0.7 kN/m^2 along with imposed and wind loads. These ties must be steel members such as beams, which may be designed for other purposes, steel bar reinforcement anchoring the steel frame to concrete floor or steel mesh reinforcement in composite slab with steel profiled sheeting directly connected to beam with shear connectors. These steel ties and their end connections should be capable of resisting factored tensile force not less than the factored dead and imposed loads acting on the floor area tributary to the tie nor less than 75 kN. Such connection of ties to edge column should also be capable of resisting 1 percent of the maximum axial compression in the column at the level due to factored dead and imposed loads. All column splices should be capable of resisting a tensile force equal to the largest of a factored dead and imposed load reaction from a single floor level located between that column splice and the next column splice below that splice. Lateral load system to resist notional horizontal loads prescribed in **11.3.6** should be distributed throughout the building in nearly orthogonal directions so that no substantial portions is connected at only one point to such a system. Precast concrete or other heavy floor or roof units should be effectively anchored in the direction of their span either to each other over the support or directly to the support.

b) Where the above conditions to tie the columns to the floor adequately are not satisfied each storey of the building should be checked to ensure that disproportionate collapse would not precipitate by the notional removal, one at a time, of each column.

c) Where each floor is not laterally supported by more than one system, check should be made at each storey by removing one such lateral support system at a time to ensure that disproportionate collapse would not occur. The collapse is considered disproportionate, if more than 15 percent of the floor or roof area of 70 m^2 collapse at that level and at one adjoining level either above or below it, under a load equal to 1.05 or 0.9 times the dead load, 0.33 times temporary or full imposed load of permanent nature (as in storage buildings) and 0.33 times wind load acting together.

12.1.4 Structures designed for unusual or special functions shall comply with any other relevant additional limit state considered appropriate to that structure.

12.2 Limit State Design

12.2.1 For achieving the design objectives, the design shall be based on characteristic values for material strengths and applied loads (actions), which take into account the probability of variations in the material strengths and in the loads to be supported. The characteristic values shall be based on statistical data, if available. Where such data is not available, these shall be based on experience. The design values are derived from the characteristic values through the use of partial safety factors, both for material strengths and for loads. In the absence of special

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considerations, these factors shall have the values given in this Section according to the material, the type of load and the limit state being considered. The reliability of design is ensured by satisfying the requirement:

$$\text{Design action} \leq \text{Design strength}$$

12.2.2 Limit states are the states beyond which the structure no longer satisfies the performance requirements specified. The limit states are classified as:

a) limit state of strength; and

b) limit state of serviceability.

12.2.2.1 The limit states of strength are those associated with failures (or imminent failure), under the action of probable and most unfavourable combination of loads on the structure using the appropriate partial safety factors, which may endanger the safety of life and property. The limit state of strength includes:

a) loss of equilibrium of the structure as a whole or any of its parts or components;

b) loss of stability of the structure (including the effect of sway where appropriate and overturning) or any of its parts including supports and foundations;

c) failure by excessive deformation, rupture of the structure or any of its parts or components;

d) fracture due to fatigue; and

e) brittle fracture.

12.2.2.2 The limit state of serviceability include:

a) deformation and deflections, which may adversely affect the appearance or effective use of the structure or may cause improper functioning of equipment or services or may cause damages to finishes and non-structural members;

b) vibrations in the structure or any of its components causing discomfort to people, damages to the structure, its contents or which may limit its functional effectiveness. Special consideration shall be given to systems susceptible to vibration, such as large open floor areas free of partitions to ensure that such vibrations are acceptable for the intended use and occupancy (*see* Annex C).;

c) repairable damage or crack due to fatigue;

d) corrosion, durability; and

e) fire.

12.3 Actions

The actions (loads) to be considered in design include direct actions (loads) experienced by the structure due to self-weight, external actions, etc, and imposed deformations such as that due to temperature and settlements.

12.3.1 *Classification of Actions*

Actions are classified by their variation with time as given below:

a) *Permanent actions* (Q_p) — Actions due to self-weight of structural and non-structural components, fittings, ancillaries, and fixed equipment, etc;

b) *Variable actions* (Q_v) — Actions due to construction and service stage loads such as imposed (live) loads (crane loads, snow loads, etc), wind loads, and earthquake effects; and

c) *Accidental actions* (Q_a) — Actions expected due to explosions, and impact of vehicles, etc.

12.3.2 *Characteristic Actions* (*Loads*)

12.3.2.1 The characteristic actions, Q_c, are the values of the different actions that are not expected to be exceeded with more than 5 percent probability, during the life of the structure and they are taken as:

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a) the self-weight, in most cases calculated on the basis of nominal dimensions and unit weights [*see* **2** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS];

b) the variable loads, values of which are specified in relevant standard [(*see* Part C 'Structural design/Section 1 Loads and Load Effects' of this NBCS];

c) the upper limit with a specified probability (usually 5 percent) not exceeding during some reference period (design life); and

d) specified by client, or by designer in consultation with client, provided they satisfy the minimum provisions of the relevant loading standard.

12.3.2.2 The characteristic values of accidental loads generally correspond to the value specified by relevant code, standard or client. The design for accidental load is generally not required in building unless it is required by client or approving authority in which case, generally recommendation in **12.1.2** (c) or specialist literature shall be followed.

12.3.3 *Design Actions*

The design action, $Q_{d,}$ is expressed as:

$$Q_d = \sum_k \gamma_{Lk} Q_{ck}$$

where

$_{Lk}$ = partial safety factor for different loads k, given in Table 4 to account for.

1) possibility of unfavourable deviation of the load from the characteristic value;

2) possibility of inaccurate assessment of the load;

3) uncertainty in the assessment of effects of the load; and

4) uncertainty in the assessment of the limit states being considered.

The loads, Load combinations and the relevant partial safety factors for loads in a combination shall be as given in Part C/Section 1 of this NBCS. The loads or load effects shall be multiplied by the relevant γ_L factors, given in Table 4, to get the design loads or design load effects, when such a value is not available in Part C/Section 1 of this NBCS.

12.4 Strength

The ultimate strength calculation may require consideration of the following:

a) Loss of equilibrium of the structure or any part of it, considered as a rigid body; and

b) Failure by excessive deformation, rupture or loss of stability of the structure or any part of it including support and foundation.

12.4.1 *Design Strength*

The design strength, $S_{d,}$ is obtained as given below from ultimate strength, S_u and partial safety factors for materials, γ_m given in Table 5.

$$S_d = S_u / \gamma_m$$

where partial safety factor for materials, γ_m account for:

a) possibility of unfavourable deviation of material strength from the characteristic value;

b) possibility of unfavourable variation of member sizes;

c) possibility of unfavourable reduction in member strength due to fabrication and tolerances; and

d) uncertainty in the calculation of strength of the members.

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Table 4 Partial Safety Factors for Loads, γ_L, for Limit States

(*Clauses* 10.6.1, 12.3.3, 12.6, 12.6.1, 18.2 *and* 22.5)

Sl No.	Combination	Limit State of Strength						Limit State of Serviceability			
		DL	IL [1]		WL/EL	FL	AL	DL	IL [1]		WL/EL/FL
			Leading	Accompanying					Leading	Accompanying	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
i)	DL + IL + CL	1.5	1.5	1.05	—	—	—	1.0	1.0	1.0	—
ii)	DL + IL + CL + WL	1.2 1.2	1.2 1.2	1.05 0.53	0.6 1.2	—	—	1.0	0.8	0.8	0.8
iii)	DL + WL	1.5 (0.9) [2]	—	—	1.5	—	—	1.0	—	—	1.0
iv)	DL + IL + CL + EL	1.2 1.2	1.2 1.2	1.05 0.53	1.0 1.0	—	—	1.0	0.8	0.8	1.0
v)	DL + EL	1.5 (0.9) [2]	—	—	1.0	—	—	1.0	—	—	1.0
vi)	DL + ER	1.2 (0.9) [2]	1.2	—	—		—	—	—	—	—
vii)	DL + IL[3] + FL	1.0	1.0/0.5	—	—	1.0	—	1.0	1.0/0.5	—	1.0
viii)	DL + IL + AL	1.0	0.35[4]	0.35[4]	—		1.0	—	—	—	—

Abbreviations:

DL= Dead load, IL= Imposed load, WL= Wind load, CL= Crane load (Vertical/Horizontal), AL=Accidental load, ER= Erection load, EL= Earthquake effects, FL = Forces due to restraints during fire.

NOTE — The effects of actions (loads) in terms of stresses or stress resultants may be obtained from an appropriate method of analysis as in 11.

[1] When action of different imposed loads is simultaneously considered, the leading imposed load shall be considered to be the one causing the higher load effects in the member/section.

[2] This value is to be considered when the dead load contributes to stability against overturning is critical or the dead load causes reduction in stress due to other loads.

[3] Imposed load shall be taken as the full characteristic specified values in case of storage loads and 50 percent of the specified characteristic values in other cases.

[4] 0.35 shall not be used when the imposed load is not permeant load as in earth pressure.

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Table 5 Partial Safety Factor for Materials, γ_m

(*Clauses* 12.4.1, 13.2, 13.3.1, 17.1.6 *and* 17.5.7.1.1)

Sl No.	Definition	Partial Safety Factor	
(1)	(2)	(3)	
i)	Resistance, governed by yielding, γ_{m0}	1.10	
ii)	Resistance of member to buckling, γ_{m0}	1.10	
iii)	Resistance, governed by ultimate stress, γ_{m1}	1.25	
iv)	Resistance of connection:	Shop Fabrications	Field Fabrications
	a) bolts-Friction Type, γ_{mf};	1.25	1.25
	b) bolts-Bearing Type, γ_{mb};	1.25	1.25
	c) rivets, γ_{mr}; and	1.25	1.25
	d) welds, γ_{mw}	1.25	1.50

12.5 Factors Governing the Ultimate Strength

12.5.1 *Stability*

Stability shall be ensured for the structure as a whole and for each of its elements. This should include, overall frame stability against overturning and sway, as given in **12.5.1.1** and **12.5.1.2**.

12.5.1.1 *Stability against overturning*

The structure as a whole or any part of it shall be designed to prevent instability due to overturning, uplift or sliding under factored load as given below:

a) The actions shall be divided into components aiding instability and components resisting instability;

b) The permanent and variable actions and their effects causing instability shall be combined using appropriate partial safety factors for loads as per the limit state requirements, to obtain maximum destabilizing effect;

c) The permanent actions (loads) and effects contributing to resistance shall be multiplied with a partial safety factor 0.9 and added together with design resistance (after multiplying with appropriate partial safety factor). Variable actions and their effects contributing to resistance shall be disregarded; and

d) The resistance effect shall be greater than or equal to the destabilizing effect. Combination of imposed and dead loads should be such as to cause most severe effect on overall stability.

12.5.1.2 *Sway stability*

The whole structure, including portions between expansion joints, shall be adequately stiff against sway. To ensure this, in addition to designing for applied horizontal loads, a separate check should be carried out for notional horizontal loads such as given in **11.3.6** to evaluate the sway under gravity loads.

12.5.2 *Fatigue*

Generally, fatigue need not be considered unless a structure or element is subjected to numerous significant fluctuations of stress. Stress changes due to fluctuations in wind loading normally need not be considered. Fatigue design shall be in accordance with the provisions under **19**. When designing for fatigue, the partial safety factor for load, $\gamma_{L,}$ equal to unity shall be used for the load causing stress fluctuation and stress range.

12.5.3 *Plastic Collapse*

Plastic analysis and design may be used, if the requirement specified under the plastic method of analysis (*see* **11.5**) are satisfied.

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12.6 Limit State of Serviceability

Serviceability limit state is related to the criteria governing normal use. Serviceability limit state is limit state beyond which the service criteria specified below, are no longer met:

a) deflection limit;

b) vibration limit;

c) durability consideration; and

d) fire resistance.

Unless specified otherwise (*see* Table 4), partial safety factor for loads, γ_L, of value equal to unity shall be used for all loads leading to serviceability limit states to check the adequacy of the structure under serviceability limit states.

12.6.1 *Deflection*

The deflection under serviceability loads of a building or a building component should not impair the strength of the structure or components or cause damage to finishing. Deflections are to be checked for the most adverse but realistic combination of service loads and their arrangement, by elastic analysis, using partial safety factors in Table 4 for loads, as required, in Table 6. Table 6 gives recommended limits of deflections for certain structural members and systems. In Table 6, imposed load should include all post construction loads including superimposed dead loads. Circumstances may arise where greater or lesser values would be more appropriate, depending upon the nature of material in element to be supported (vulnerable to cracking or not) and intended use of the structure, as required by client.

Where the deflection due to the combination of dead load and imposed load is likely to be excessive, consideration should be given to pre-camber the beams, trusses and girders. The value of desired camber shall be specified in design drawing. Generally, for spans greater than 25 m, a camber approximately equal to the deflection due to dead loads plus half the imposed load may be used. The deflection of a member shall be calculated without considering the impact factor or dynamic effect of the loads on deflection. Roofs, which are very flexible, shall be designed to withstand any additional load that is likely to occur as a result of ponding of water or accumulation of snow or ice.

12.6.2 *Vibration*

Suitable provisions in the design shall be made for the dynamic effects of imposed loads, impact loads and vibration due to machinery operating loads. In severe cases possibility of resonance, fatigue or unacceptable vibrations shall be investigated. Unusually flexible structures (generally the height to effective width of lateral load resistance system exceeding 5 : 1) shall be investigated for lateral vibration under dynamic wind loads. Structures subjected to large number of cycles of loading shall be designed against fatigue failure, as specified under **19**. Floor vibration effect shall be considered using specialist literature (*see* Annex C).

12.6.3 *Durability*

Factors that affect the durability of the buildings, under conditions relevant to their intended life, are listed below:

a) Environment;

b) Degree of exposure;

c) Shape of the member and the structural detail;

d) Protective measure; and

e) Ease of maintenance.

The durability of steel structures shall be ensured by following recommendations given under **21**. Specialist literature may be referred to for more detailed and additional information in design for durability.

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Table 6 Deflection Limits

(*Clause* 12.6.1)

Sl No.	Type of Building	Deflection	Design Load	Member	Supporting	Maximum Deflection
(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Industrial buildings	Vertical	Imposed load and wind load	Purlins and Girts	Elastic cladding	Span/150
					Brittle cladding	Span/180
			Imposed load	Simple span	Elastic cladding	Span/240
					Brittle cladding	Span/300
			Imposed load	Cantilever span	Elastic cladding	Span/120
					Brittle cladding	Span/150
			Imposed load, wind load and crane load	Rafter supporting	Profiled metal sheeting	Span/180
					Plastered sheeting	Span/240
			Crane load (Manual operation)	Gantry	Wheels of crane	Span/500
			Crane load (Electric operation up to 50 t)	Gantry	Wheels of crane	Span/750
			Crane load (Electric operation over 50 t)	Gantry	Wheels of crane	Span/1000
		Lateral	No cranes (due to wind)	Column	Elastic cladding	Height/150
					Masonry/Brittle cladding	Height/240
			Crane load and wind load	Gantry (lateral)	(absolute)	Span/400
					Relative displacement between rails supporting wheels of crane	25 mm[1)]
			Crane load and wind load	Column/Frame	Gantry (Elastic cladding)	Height/200
					Gantry (Brittle cladding; cab operated)	Height/400
	Other Buildings	Vertical	Imposed load	Floor and roof	Elements not susceptible to cracking	Span/300
					Elements susceptible to cracking	Span/360
			Imposed load	Cantilever	Elements not susceptible to cracking	Span/150
					Elements susceptible to cracking	Span/180
		Lateral	Wind load	Building	Elastic cladding	Height/300
					Brittle cladding	Height/500
			Wind load	Inter-storey drift	—	Storey height/300

[1)] Crane supplier may be consulted for higher values

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12.6.4 *Fire Resistance*

Fire resistance of a steel member is a function of its mass, its geometry, and the actions to which it is subjected, its structural support condition, fire protection measures adopted and the fire to which it is exposed. Design provisions to resist fire are briefly discussed under **22**. Specialist literature may be referred to for more detailed information in design of fire resistance of steel structures.

13 DESIGN OF TENSION MEMBERS

13.1 Tension Members

Tension members are linear members in which axial forces act to cause elongation (stretch). Such members can sustain loads up to the ultimate load, at which stage they may fail by rupture at a critical section. However, if the gross area of the member yields over a major portion of its length before the rupture load is reached, the member may become non-functional due to excessive elongation. Plates and other rolled sections in tension may also fail by block shear of end bolted regions (*see* **13.4.1**).

The factored design tension T, in the members shall satisfy the following requirement:

$$T < T_d$$

where

T_d = Design strength of the member.

The design strength of a member under axial tension, T_d, is the lowest of the design strength due to yielding of gross section, T_{dg}; rupture strength of critical section, T_{dn}; and block shear T_{db}, given in **13.2**, **13.3** and **13.4**, respectively.

13.2 Design Strength Due to Yielding of Gross Section

The design strength of members under axial tension, T_{dg}, as governed by yielding of gross section, is given by:

$$T_{dg} = A_g f_y / \gamma_{m0}$$

where

A_g = gross area of cross section;

f_y = yield stress of the material; and

γ_{m0} = partial safety factor for failure in tension by yielding (*see* Table 5).

13.3 Design Strength Due to Rupture of Critical Section

13.3.1 *Plates*

The design strength in tension of a plate, T_{dn}, as governed by rupture of net cross sectional area, A_n, at the holes is given by:

$$T_{dn} = 0.9 A_n f_u / \gamma_{m1}$$

where

A_n = net effective area of the member given by;

f_u = ultimate stress of the material; and

γ_{m1} = partial safety factor for failure at ultimate stress (*see* Table 5).

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$$A_n = \left[b - nd_h + \sum \frac{p_{si}^2}{4g_i} \right] t$$

where

b, t = width and thickness of the plate, respectively;

d_h = diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case of directly punched holes);

g = gauge length between the bolt holes, as shown in Fig. 5;

i = subscript for summation of all the inclined legs;

n = number of bolt holes in the critical section; and

p_s = staggered pitch length between line of bolt holes, as shown in Fig. 5.

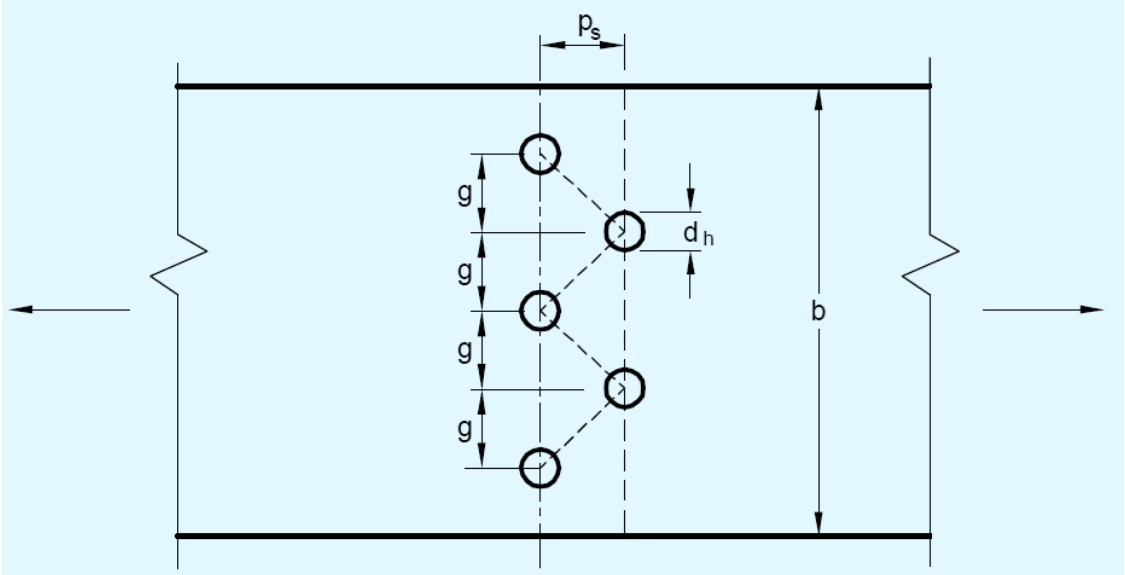

FIG. 5 PLATES WITH BOLTS HOLES IN TENSION

13.3.2 *Threaded Rods*

The design strength of threaded rods in tension, T_{dn}, as governed by rupture is given by:

$$T_{dn} = 0.9 A_c f_u / \gamma_{m1}$$

where

A_c = net root area at the threaded section.

13.3.3 *Single Angles*

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, T_{dn}, as governed by rupture at net section is given by:

$$T_{dn} = 0.9 A_{nc} f_u / \gamma_{m1} + \beta A_{go} f_y / \gamma_{m0}$$

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where

β = $1.4 - 0.076\ (w/t)\ (f_y/f_u)\ (b_s/L_c\ ) \leq (0.9 f_u \gamma_{m0}/f_y \gamma_{m1}) \geq 0.7$

where

A_{go} = gross area of the outstanding leg;

A_n = net area of the total cross section

A_{nc} = net area of the connected leg;

b_s = shear lag width, as shown in Fig. 6;

L_c = length of the end connection, that is the distance between the outermost bolts in the end joint measured along the load direction or length of the weld along the load direction;

t = thickness of the leg; and

w = outstand leg width.

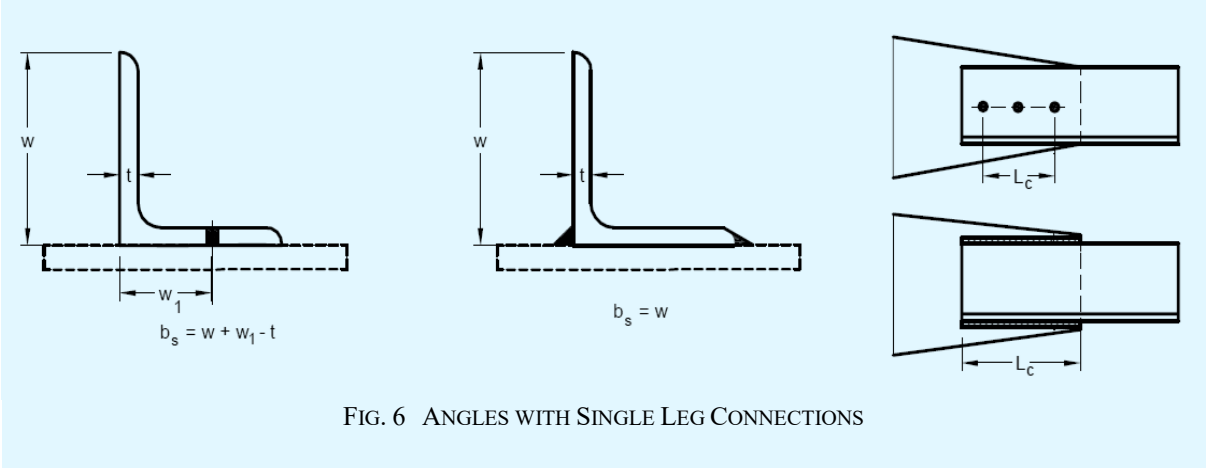

FIG. 6 ANGLES WITH SINGLE LEG CONNECTIONS

13.3.4 *Other Section*

The rupture strength, $T_{dn,}$ of the double angles, channels, I-sections and other rolled steel sections, connected by one or more elements to an end gusset is also governed by shear lag effects. The design tensile strength of such sections as governed by tearing of net section may also be calculated using equation in **13.3.3**, where β is calculated based on the shear lag distance, $b_{s,}$ taken from the farthest edge of the outstanding leg to the nearest bolt/weld line in the connected leg of the cross section.

13.4 Design Strength Due to Block Shear

The strength as governed by block shear at an end connection of plates and angles is calculated as given in **13.4.1**.

13.4.1 *Bolted Connections*

The block shear strength, T_{db} of connection shall be taken as the smaller of,

$$T_{db} = [A_{vg}\ f_y/(\sqrt{3}\ \gamma_{m0}) + 0.9 A_{tn}\ f_u\ /\gamma_{m1}\ ]$$

or

$$T_{db} = [0.9 A_{vn}\ f_u\ /(\sqrt{3}\ \gamma_{m1}) + A_{tg}\ f_y\ /\gamma_{m0}]$$

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where

A_{tg} , A_{tn} = minimum gross and net area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force, respectively (2-3 as shown in Fig. 7(B);

A_{vg} , A_{vn} = minimum gross and net area in shear along bolt line parallel to external force, respectively (1-2 and 3-4 as shown in Fig. 7(A) and 1-2 as shown in Fig. 7(B); and

f_u , f_y = ultimate and yield stress of the material, respectively.

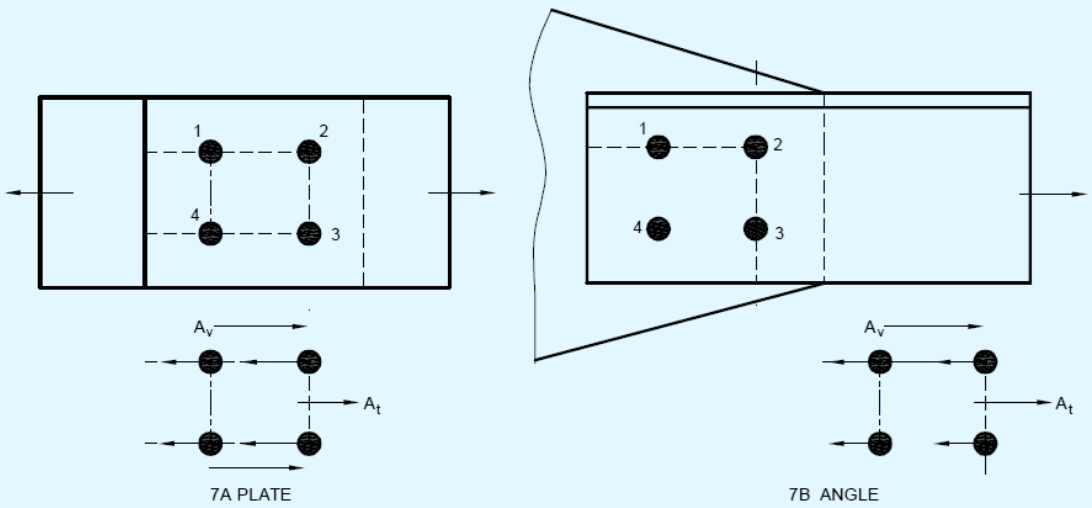

FIG. 7 BLOCK SHEAR FAILURE

13.4.2 *Welded Connection*

The block shear strength, T_{db} shall be checked for welded end connections by taking an appropriate section in the member around the end weld, which can shear off as a block.

14 DESIGN OF COMPRESSION MEMBERS

14.1 Design Strength

14.1.1 Common hot rolled and built-up steel members used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load. To account for all these factors, the strength of members subjected to axial compression is defined by buckling class a_0, a, b, c, or d as given Table 7.

14.1.2 The factored design compression, P in members shall satisfy the following requirement:

$$P < P_d$$

$$P_d = A_e f_{cd}$$

where

P_d = design strength of the member

A_e = effective sectional area as defined in **14.3.2**; and

f_{cd} = design compressive stress, given by.

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$$f_{cd} = \frac{\chi f_y}{\gamma_{m0}} \leq f_y/\gamma_{m0}$$

χ = stress reduction factor for different buckling class, slenderness ratio and yield stress and mode of buckling as given in **14.1.2.1** or **14.1.2.2**, as applicable (*see* Table 8); and

γ_{m0} = partial safety factor for material strength.

NOTE — Calculated values of design compressive stress, f_{cd} for different buckling classes are given in Table 9.

14.1.2.1 *Reduction factor for flexural buckling* (*See also* Table 8)

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 + \lambda^2}}$$

$\phi = 0.5[1+\alpha(\lambda - 0.2) + \lambda^2]$

λ = non-dimensional effective slenderness ratio:

$= \sqrt{f_y/f_{cr}} = \sqrt{f_y \left(KL/r\right)^2 / \pi^2 E}$

f_{cr} = Euler buckling stress about the relevant axis = $\frac{\pi^2 E}{(KL/r)^2}$

where

KL/r = effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration, r; and

α = imperfection factor given in Table 7.

14.1.2.2 *Reduction factor for torsional and torsional flexural buckling*

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 + \lambda_{TF}^2}}$$

$$\phi = 0.5\left[1 + \left(\frac{\lambda_{TF}}{\lambda_y}\right)^2 . \alpha_{TF}(\lambda_y - 0.2) + \lambda_{TF}^2\right]$$

$$\alpha_{TF} = \alpha\sqrt{\frac{A\, f_y(i_p^2 + d_y^2)}{6.25\, G\, I_T}}$$

$$\lambda_{TF} = \sqrt{f_y/f_{crTF}}$$

$$\lambda_Y = \sqrt{f_y/f_{cr,y}}$$

where

α = is the imperfection factor given in Table 7;

d_y = is the distance between intermediate lateral support and shear centre at the section;

i_p = is the polar radius of gyration given by;

i_p^2 = $(I_y + I_z)/A$;

I_T = is the torsional constant of the cross section;

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I_y, I_z = is the moment of inertia about y and z axis, respectively and shear centre;

λ_{TF} = non-dimensional effective slenderness ratio; and

λ_y = non-dimensional effective slenderness ratio.

f_{cry} = flexural buckling stress about the y axis ; and

f_{crTF} = torsional flexural buckling stress at the extreme compression fibre (Annex E).

14.1.2.3 The classification of different sections under different buckling class a_0, *a*, *b*, *c* or *d*, is given in Table 10. The stress reduction factor χ, and the design compressive stress f_{cd}, for different buckling class, yield stress, and effective slenderness ratio is given in Table 8 and Table 9 for convenience. The curves corresponding to different buckling class are presented in non-dimensional form, in Fig. 8.

Table 7 Imperfection Factor, α

(*Clauses* 14.1.1, 14.1.2.1 *and* 14.1.2.2)

Sl No.	Buckling Class	a_0	a	b	c	d
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	α	0.13	0.21	0.34	0.49	0.76

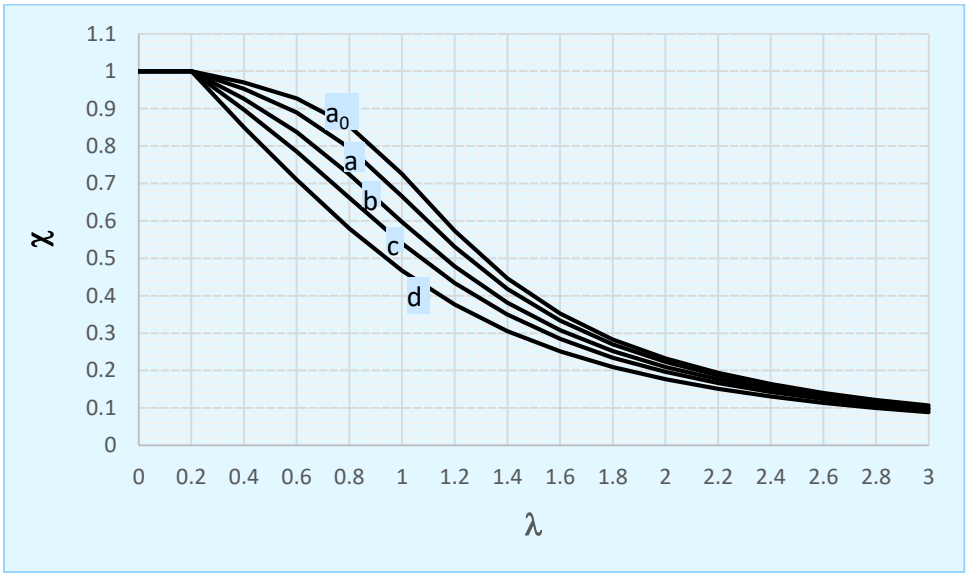

FIG. 8 COLUMN BUCKLING CURVES

14.2 Effective Length of Compression Members

14.2.1 The effective length *KL*, is calculated from the actual length *L*, of the member, considering the rotational and relative translational boundary conditions at the ends. The actual length shall be taken as the length from centre to centre of its intersections with the supporting members in the plane of the buckling deformation. In the case of a member with a free end, the free standing length from the centre of the intersecting member at the supported end, shall be taken as the actual length.

14.2.2 *Effective Length*

Where the boundary conditions in the plane of buckling can be assessed, the effective length, *KL* can be calculated on the basis of Table 11. Where frame analysis does not consider the equilibrium of a framed structure in the

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deformed shape (second-order analysis or advanced analysis), the effective length of compression members in such cases can be calculated using the procedure given in **D-1**. The effective length of stepped column in single storey buildings can be calculated using the procedure given in **D-2**.

14.2.3 *Eccentric Beam Connection*

In cases where the beam connections are eccentric in plan with respect to the axes of the column, the same conditions of restraint as in concentric connection shall be deemed to apply, provided the connections are carried across the flange or web of the columns as the case may be, and the web of the beam lies within, or in direct contact with the column section. Where practical difficulties prevent this, the effective length shall be taken as equal to the distance between points of restraint in non-sway frames.

14.2.4 *Compression Members in Trusses*

In the case of bolted, riveted or welded trusses and braced frames, the effective length, *KL*, of the compression members shall be taken as 0.7 to 1.0 times the distance between centres of connections, depending on the degree of end restraint provided. In the case of members of trusses buckling in the plane perpendicular to the plane of the truss, the effective length, *KL* shall be taken as the distance between the centres of intersection. The design of angle struts shall be as specified in **14.5**.

14.3 Design Details

14.3.1 *Thickness of Plate Elements*

Classification of members on the basis of the thickness of constituent plate elements shall satisfy the width-thickness ratio requirements specified in Table 2.

14.3.2 *Effective Sectional Area, A_e*

Except as modified in **10.8.2** (Class 4), the gross sectional area shall be taken as the effective sectional area for all compression members fabricated by welding, bolting and riveting so long as the section is semi-compact or better. Holes not fitted with rivets, bolts or pins shall be deducted from gross area to calculate effective sectional area.

14.3.3 *Eccentricity for Stanchions and Columns*

14.3.3.1 For the purpose of determining the stress in a stanchion or column section, the beam reactions or similar loads shall be assumed to be applied at an eccentricity of 100 mm from the face of the section or at the centre of bearing whichever dimension gives the greater eccentricity, and with the exception of the following two cases:

a) In the case of cap connection, the load shall be assumed to be applied at the face of the column or stanchion section or at the edge of packing, if used towards the span of the beam; and

b) In the case of a roof truss bearing on a cap, no eccentricity be taken for simple bearings without connections capable of developing any appreciable moment. In case of web member connection with face, actual eccentricity is to be considered.

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Table 8(A) Stress Reduction Factor, χ for Flexural Column Buckling Class, a_0

(*Clauses* 14.1.2.1 *and* 14.1.2.3)

KL/*r*	Yield Stress, f_y (MPa)																		
↓	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	1.000	0.999	0.998	0.998	0.997	0.997	0.996	0.995	0.994	0.992	0.991	0.990	0.989	0.988	0.987	0.986	0.984	0.983	0.981
30	0.986	0.985	0.983	0.982	0.981	0.980	0.979	0.977	0.975	0.973	0.971	0.969	0.967	0.965	0.964	0.961	0.958	0.955	0.952
40	0.970	0.968	0.966	0.965	0.963	0.961	0.960	0.956	0.953	0.949	0.946	0.943	0.939	0.936	0.932	0.927	0.921	0.915	0.909
50	0.951	0.948	0.945	0.943	0.940	0.937	0.934	0.929	0.923	0.917	0.911	0.905	0.899	0.892	0.885	0.874	0.863	0.852	0.840
60	0.927	0.922	0.918	0.914	0.909	0.905	0.900	0.891	0.881	0.870	0.860	0.848	0.837	0.824	0.812	0.792	0.772	0.752	0.731
70	0.895	0.888	0.881	0.874	0.867	0.859	0.852	0.836	0.819	0.802	0.784	0.766	0.747	0.728	0.710	0.682	0.655	0.629	0.605
80	0.851	0.841	0.830	0.819	0.808	0.796	0.784	0.760	0.736	0.712	0.687	0.664	0.641	0.619	0.598	0.568	0.540	0.515	0.491
90	0.792	0.777	0.762	0.747	0.731	0.716	0.701	0.670	0.641	0.614	0.587	0.563	0.539	0.518	0.498	0.470	0.445	0.422	0.401
100	0.721	0.702	0.683	0.665	0.647	0.629	0.612	0.580	0.550	0.523	0.498	0.475	0.453	0.434	0.416	0.391	0.369	0.349	0.332
110	0.643	0.622	0.602	0.583	0.564	0.547	0.530	0.499	0.471	0.446	0.423	0.403	0.384	0.366	0.350	0.329	0.310	0.293	0.278
120	0.568	0.547	0.527	0.509	0.491	0.475	0.459	0.431	0.405	0.383	0.363	0.344	0.328	0.312	0.299	0.280	0.264	0.249	0.236
130	0.500	0.480	0.462	0.445	0.429	0.414	0.399	0.374	0.351	0.331	0.313	0.297	0.282	0.269	0.257	0.241	0.227	0.214	0.203
140	0.441	0.423	0.406	0.390	0.376	0.362	0.350	0.327	0.307	0.289	0.273	0.259	0.246	0.234	0.223	0.209	0.197	0.186	0.176
150	0.391	0.374	0.359	0.345	0.332	0.319	0.308	0.288	0.270	0.254	0.240	0.227	0.216	0.205	0.196	0.183	0.172	0.163	0.154
160	0.348	0.333	0.319	0.306	0.294	0.283	0.273	0.255	0.239	0.225	0.212	0.201	0.191	0.181	0.173	0.162	0.152	0.144	0.136
170	0.311	0.298	0.285	0.273	0.263	0.253	0.244	0.227	0.213	0.200	0.189	0.179	0.170	0.161	0.154	0.144	0.135	0.128	0.121
180	0.280	0.268	0.256	0.246	0.236	0.227	0.219	0.204	0.191	0.179	0.169	0.160	0.152	0.145	0.138	0.129	0.121	0.114	0.108
190	0.253	0.242	0.231	0.222	0.213	0.205	0.197	0.184	0.172	0.162	0.152	0.144	0.137	0.130	0.124	0.116	0.109	0.103	0.097
200	0.230	0.219	0.210	0.201	0.193	0.186	0.179	0.166	0.156	0.146	0.138	0.131	0.124	0.118	0.112	0.105	0.099	0.093	0.088
210	0.209	0.200	0.191	0.183	0.176	0.169	0.163	0.151	0.142	0.133	0.125	0.119	0.113	0.107	0.102	0.095	0.090	0.084	0.080
220	0.191	0.183	0.175	0.167	0.161	0.154	0.149	0.138	0.129	0.122	0.115	0.108	0.103	0.098	0.093	0.087	0.082	0.077	0.073
230	0.176	0.168	0.160	0.154	0.147	0.142	0.136	0.127	0.119	0.112	0.105	0.099	0.094	0.090	0.086	0.080	0.075	0.071	0.067
240	0.162	0.155	0.148	0.141	0.136	0.131	0.126	0.117	0.109	0.103	0.097	0.091	0.087	0.083	0.079	0.074	0.069	0.065	0.061
250	0.150	0.143	0.136	0.131	0.125	0.121	0.116	0.108	0.101	0.095	0.089	0.084	0.080	0.076	0.073	0.068	0.064	0.060	0.057

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Table 8(B) Stress Reduction Factor, χ for Flexural Column Buckling Class a

(*Clauses* 14.1.2.1 *and* 14.1.2.3)

KL/r ↓	**Yield Stress, f_y (MPa)**																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	1.000	0.999	0.998	0.997	0.995	0.994	0.993	0.993	0.990	0.988	0.986	0.984	0.983	0.981	0.979	0.977	0.975	0.972	0.970
30	0.977	0.975	0.974	0.972	0.970	0.969	0.967	0.965	0.961	0.957	0.954	0.951	0.948	0.946	0.943	0.938	0.934	0.930	0.925
40	0.952	0.949	0.947	0.944	0.942	0.939	0.937	0.934	0.926	0.921	0.916	0.911	0.906	0.901	0.896	0.888	0.881	0.873	0.865
50	0.923	0.919	0.915	0.911	0.908	0.904	0.900	0.896	0.884	0.876	0.867	0.859	0.851	0.842	0.834	0.820	0.807	0.794	0.780
60	0.888	0.883	0.877	0.871	0.865	0.859	0.853	0.847	0.828	0.816	0.803	0.790	0.777	0.763	0.750	0.730	0.710	0.690	0.671
70	0.846	0.837	0.829	0.820	0.811	0.803	0.794	0.785	0.758	0.740	0.722	0.703	0.686	0.668	0.651	0.626	0.602	0.579	0.557
80	0.793	0.781	0.769	0.757	0.746	0.734	0.722	0.710	0.675	0.653	0.631	0.610	0.589	0.570	0.551	0.525	0.501	0.478	0.458
90	0.730	0.715	0.700	0.685	0.671	0.657	0.643	0.628	0.590	0.565	0.542	0.520	0.500	0.481	0.463	0.439	0.416	0.396	0.377
100	0.661	0.644	0.627	0.610	0.594	0.579	0.564	0.549	0.510	0.486	0.463	0.443	0.424	0.407	0.390	0.368	0.348	0.331	0.314
110	0.591	0.573	0.555	0.538	0.522	0.507	0.492	0.478	0.440	0.418	0.397	0.379	0.362	0.346	0.332	0.312	0.295	0.279	0.265
120	0.525	0.507	0.489	0.473	0.458	0.443	0.429	0.416	0.381	0.361	0.343	0.326	0.311	0.297	0.284	0.267	0.252	0.238	0.226
130	0.466	0.448	0.432	0.416	0.402	0.388	0.376	0.364	0.332	0.314	0.298	0.283	0.269	0.257	0.246	0.231	0.217	0.206	0.195
140	0.413	0.397	0.382	0.368	0.355	0.342	0.331	0.320	0.291	0.275	0.260	0.247	0.235	0.224	0.214	0.201	0.189	0.179	0.170
150	0.368	0.353	0.339	0.326	0.314	0.303	0.293	0.283	0.257	0.243	0.229	0.218	0.207	0.197	0.189	0.177	0.166	0.157	0.149
160	0.329	0.316	0.303	0.291	0.280	0.270	0.261	0.252	0.229	0.215	0.204	0.193	0.184	0.175	0.167	0.157	0.147	0.139	0.132
170	0.296	0.283	0.272	0.261	0.251	0.242	0.233	0.225	0.204	0.192	0.182	0.172	0.164	0.156	0.149	0.140	0.131	0.124	0.117
180	0.267	0.255	0.245	0.235	0.226	0.218	0.210	0.203	0.184	0.173	0.163	0.155	0.147	0.140	0.134	0.125	0.118	0.111	0.105
190	0.242	0.231	0.222	0.213	0.205	0.197	0.190	0.183	0.166	0.156	0.147	0.140	0.133	0.126	0.121	0.113	0.106	0.100	0.095
200	0.220	0.210	0.202	0.193	0.186	0.179	0.172	0.166	0.151	0.142	0.134	0.127	0.120	0.115	0.109	0.102	0.096	0.091	0.086
210	0.201	0.192	0.184	0.177	0.170	0.163	0.157	0.152	0.137	0.129	0.122	0.115	0.110	0.104	0.099	0.093	0.087	0.083	0.078
220	0.184	0.176	0.169	0.162	0.155	0.149	0.144	0.139	0.126	0.118	0.111	0.106	0.100	0.095	0.091	0.085	0.080	0.075	0.071
230	0.170	0.162	0.155	0.149	0.143	0.137	0.132	0.128	0.115	0.108	0.102	0.097	0.092	0.088	0.083	0.078	0.073	0.069	0.065
240	0.157	0.149	0.143	0.137	0.132	0.127	0.122	0.118	0.106	0.100	0.094	0.089	0.085	0.081	0.077	0.072	0.068	0.064	0.060
250	0.145	0.138	0.132	0.127	0.122	0.117	0.113	0.109	0.098	0.092	0.087	0.082	0.078	0.074	0.071	0.066	0.062	0.059	0.056

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Table 8(C) Stress Reduction Factor, χ for Flexural Column Buckling Class b

(*Clauses* 14.1.2.1 *and* 14.1.2.3)

***KL*/r** ↓	**Yield Stress, f_y (MPa)**																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	1.000	0.998	0.996	0.994	0.993	0.991	0.990	0.986	0.983	0.981	0.978	0.975	0.972	0.970	0.967	0.963	0.960	0.956	0.953
30	0.963	0.961	0.958	0.955	0.953	0.950	0.948	0.943	0.938	0.933	0.929	0.924	0.920	0.915	0.911	0.904	0.898	0.892	0.886
40	0.925	0.921	0.917	0.913	0.909	0.906	0.902	0.895	0.887	0.880	0.873	0.866	0.859	0.852	0.845	0.835	0.825	0.815	0.805
50	0.883	0.877	0.872	0.866	0.861	0.855	0.850	0.839	0.829	0.818	0.808	0.798	0.787	0.777	0.767	0.752	0.737	0.722	0.708
60	0.835	0.827	0.820	0.812	0.805	0.798	0.790	0.775	0.761	0.746	0.732	0.718	0.704	0.691	0.677	0.657	0.638	0.620	0.602
70	0.781	0.771	0.761	0.751	0.742	0.732	0.722	0.703	0.685	0.667	0.649	0.632	0.615	0.599	0.584	0.561	0.540	0.520	0.502
80	0.721	0.709	0.697	0.685	0.673	0.661	0.650	0.627	0.606	0.585	0.566	0.547	0.529	0.512	0.496	0.474	0.453	0.434	0.416
90	0.657	0.643	0.629	0.615	0.602	0.589	0.576	0.552	0.530	0.508	0.488	0.470	0.452	0.436	0.421	0.400	0.380	0.363	0.346
100	0.593	0.577	0.562	0.548	0.534	0.520	0.507	0.483	0.461	0.440	0.421	0.403	0.387	0.372	0.358	0.339	0.321	0.306	0.291
110	0.531	0.515	0.500	0.485	0.471	0.458	0.445	0.422	0.401	0.381	0.364	0.348	0.333	0.319	0.306	0.289	0.274	0.260	0.247
120	0.474	0.458	0.443	0.429	0.416	0.403	0.391	0.370	0.350	0.332	0.316	0.301	0.288	0.276	0.265	0.249	0.236	0.223	0.212
130	0.423	0.408	0.394	0.380	0.368	0.356	0.345	0.325	0.307	0.291	0.276	0.263	0.251	0.240	0.230	0.217	0.204	0.194	0.184
140	0.378	0.364	0.350	0.338	0.327	0.316	0.306	0.287	0.271	0.256	0.243	0.231	0.221	0.211	0.202	0.190	0.179	0.169	0.161
150	0.339	0.325	0.313	0.302	0.291	0.281	0.272	0.255	0.241	0.227	0.215	0.205	0.195	0.186	0.178	0.167	0.158	0.149	0.142
160	0.305	0.292	0.281	0.271	0.261	0.252	0.243	0.228	0.215	0.203	0.192	0.182	0.174	0.166	0.158	0.149	0.140	0.133	0.126
170	0.275	0.264	0.253	0.244	0.235	0.227	0.219	0.205	0.193	0.182	0.172	0.163	0.155	0.148	0.142	0.133	0.125	0.118	0.112
180	0.249	0.239	0.229	0.220	0.212	0.205	0.198	0.185	0.174	0.164	0.155	0.147	0.140	0.133	0.128	0.120	0.113	0.106	0.101
190	0.227	0.217	0.208	0.200	0.193	0.186	0.179	0.168	0.157	0.148	0.140	0.133	0.127	0.121	0.115	0.108	0.102	0.096	0.091
200	0.207	0.198	0.190	0.183	0.176	0.169	0.163	0.153	0.143	0.135	0.128	0.121	0.115	0.110	0.105	0.098	0.092	0.087	0.083
210	0.190	0.182	0.174	0.167	0.161	0.155	0.149	0.140	0.131	0.123	0.117	0.110	0.105	0.100	0.096	0.090	0.084	0.080	0.075
220	0.174	0.167	0.160	0.154	0.148	0.142	0.137	0.128	0.120	0.113	0.107	0.101	0.096	0.092	0.088	0.082	0.077	0.073	0.069
230	0.161	0.154	0.147	0.141	0.136	0.131	0.126	0.118	0.111	0.104	0.098	0.093	0.088	0.084	0.080	0.075	0.071	0.067	0.063
240	0.149	0.142	0.136	0.131	0.126	0.121	0.117	0.109	0.102	0.096	0.091	0.086	0.082	0.078	0.074	0.070	0.065	0.062	0.058
250	0.138	0.132	0.126	0.121	0.117	0.112	0.108	0.101	0.095	0.089	0.084	0.080	0.076	0.072	0.069	0.064	0.060	0.057	0.054

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Table 8(D) Stress Reduction Factor, χ for Column Flexural Buckling Class c

(*Clauses* 14.1.2.1 *and* 14.1.2.3)

***KL*/r** ↓	**Yield Stress, f_y (MPa)**																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	0.999	0.997	0.994	0.992	0.990	0.987	0.985	0.981	0.976	0.972	0.968	0.964	0.961	0.957	0.953	0.948	0.943	0.938	0.933
30	0.948	0.944	0.941	0.937	0.933	0.930	0.926	0.920	0.913	0.907	0.901	0.895	0.889	0.883	0.877	0.869	0.861	0.853	0.845
40	0.896	0.891	0.885	0.880	0.875	0.870	0.866	0.856	0.847	0.838	0.829	0.820	0.812	0.803	0.795	0.783	0.771	0.760	0.748
50	0.841	0.834	0.827	0.821	0.814	0.807	0.801	0.788	0.776	0.763	0.752	0.740	0.729	0.717	0.706	0.690	0.675	0.660	0.645
60	0.783	0.774	0.765	0.757	0.748	0.740	0.732	0.716	0.700	0.685	0.670	0.656	0.642	0.628	0.615	0.596	0.578	0.561	0.544
70	0.722	0.711	0.700	0.690	0.680	0.670	0.660	0.641	0.623	0.605	0.588	0.572	0.557	0.542	0.528	0.508	0.489	0.471	0.454
80	0.659	0.646	0.634	0.622	0.611	0.600	0.589	0.568	0.548	0.529	0.512	0.495	0.479	0.464	0.450	0.430	0.412	0.395	0.379
90	0.596	0.583	0.569	0.557	0.544	0.533	0.521	0.499	0.479	0.460	0.443	0.426	0.411	0.397	0.383	0.365	0.348	0.332	0.318
100	0.536	0.522	0.508	0.495	0.483	0.471	0.459	0.438	0.418	0.400	0.384	0.368	0.354	0.341	0.328	0.311	0.296	0.282	0.269
110	0.480	0.466	0.453	0.440	0.428	0.416	0.405	0.385	0.366	0.349	0.333	0.319	0.306	0.294	0.283	0.268	0.254	0.242	0.230
120	0.430	0.416	0.403	0.391	0.379	0.368	0.358	0.339	0.321	0.306	0.291	0.278	0.267	0.256	0.246	0.232	0.220	0.209	0.199
130	0.385	0.372	0.360	0.348	0.337	0.327	0.317	0.299	0.283	0.269	0.256	0.244	0.234	0.224	0.215	0.203	0.192	0.182	0.173
140	0.346	0.333	0.322	0.311	0.301	0.291	0.282	0.266	0.251	0.238	0.227	0.216	0.206	0.197	0.189	0.178	0.168	0.160	0.152
150	0.311	0.300	0.289	0.279	0.269	0.261	0.252	0.237	0.224	0.212	0.202	0.192	0.183	0.175	0.168	0.158	0.149	0.141	0.134
160	0.281	0.270	0.260	0.251	0.242	0.234	0.227	0.213	0.201	0.190	0.180	0.172	0.164	0.156	0.150	0.141	0.133	0.126	0.120
170	0.255	0.245	0.236	0.227	0.219	0.212	0.205	0.192	0.181	0.171	0.162	0.154	0.147	0.140	0.134	0.126	0.119	0.113	0.107
180	0.232	0.223	0.214	0.206	0.199	0.192	0.186	0.174	0.164	0.155	0.147	0.139	0.133	0.127	0.121	0.114	0.107	0.102	0.096
190	0.212	0.203	0.195	0.188	0.181	0.175	0.169	0.158	0.149	0.140	0.133	0.126	0.120	0.115	0.110	0.103	0.097	0.092	0.087
200	0.194	0.186	0.179	0.172	0.166	0.160	0.154	0.144	0.136	0.128	0.121	0.115	0.110	0.105	0.100	0.094	0.089	0.084	0.079
210	0.178	0.171	0.164	0.158	0.152	0.146	0.141	0.132	0.124	0.117	0.111	0.105	0.100	0.096	0.092	0.086	0.081	0.076	0.072
220	0.164	0.157	0.151	0.145	0.140	0.135	0.130	0.122	0.114	0.108	0.102	0.097	0.092	0.088	0.084	0.079	0.074	0.070	0.066
230	0.152	0.145	0.140	0.134	0.129	0.124	0.120	0.112	0.105	0.099	0.094	0.089	0.085	0.081	0.077	0.073	0.068	0.065	0.061
240	0.141	0.135	0.129	0.124	0.120	0.115	0.111	0.104	0.098	0.092	0.087	0.082	0.078	0.075	0.071	0.067	0.063	0.060	0.056
250	0.131	0.125	0.120	0.115	0.111	0.107	0.103	0.096	0.090	0.085	0.081	0.076	0.073	0.069	0.066	0.062	0.058	0.055	0.052

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Table 8(E) Stress Reduction Factor, χ for Flexural Column Buckling Class d

(*Clauses* 14.1.2.1 *and* 14.1.2.3)

***KL*/r** ↓	**Yield Stress, fy** (MPa)																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	0.999	0.995	0.991	0.988	0.984	0.980	0.977	0.970	0.964	0.958	0.952	0.946	0.940	0.935	0.930	0.922	0.915	0.908	0.901
30	0.922	0.916	0.911	0.906	0.901	0.896	0.891	0.881	0.872	0.863	0.855	0.847	0.839	0.831	0.823	0.813	0.802	0.792	0.782
40	0.848	0.841	0.834	0.828	0.821	0.815	0.808	0.796	0.784	0.773	0.762	0.751	0.741	0.731	0.721	0.707	0.694	0.681	0.668
50	0.777	0.768	0.760	0.752	0.744	0.736	0.728	0.713	0.699	0.685	0.672	0.659	0.647	0.635	0.624	0.608	0.592	0.577	0.563
60	0.707	0.697	0.687	0.678	0.668	0.659	0.651	0.634	0.617	0.602	0.587	0.573	0.560	0.547	0.535	0.517	0.501	0.486	0.471
70	0.640	0.629	0.618	0.607	0.597	0.587	0.578	0.559	0.542	0.526	0.510	0.496	0.482	0.469	0.456	0.439	0.423	0.408	0.394
80	0.576	0.564	0.553	0.542	0.531	0.521	0.511	0.492	0.474	0.458	0.442	0.428	0.414	0.402	0.390	0.373	0.358	0.344	0.330
90	0.517	0.505	0.493	0.482	0.471	0.461	0.451	0.432	0.415	0.399	0.384	0.370	0.357	0.345	0.334	0.319	0.304	0.292	0.280
100	0.464	0.451	0.440	0.428	0.418	0.408	0.398	0.380	0.363	0.348	0.334	0.321	0.309	0.298	0.288	0.274	0.261	0.249	0.239
110	0.416	0.404	0.392	0.381	0.371	0.361	0.352	0.335	0.319	0.305	0.292	0.281	0.270	0.259	0.250	0.237	0.226	0.215	0.206
120	0.373	0.361	0.350	0.340	0.330	0.321	0.313	0.297	0.282	0.269	0.257	0.246	0.236	0.227	0.219	0.207	0.197	0.187	0.179
130	0.336	0.325	0.314	0.305	0.295	0.287	0.279	0.264	0.251	0.239	0.228	0.218	0.209	0.200	0.193	0.182	0.173	0.164	0.157
140	0.303	0.292	0.283	0.274	0.265	0.257	0.250	0.236	0.224	0.213	0.203	0.194	0.185	0.178	0.171	0.161	0.153	0.145	0.138
150	0.274	0.264	0.255	0.247	0.239	0.231	0.224	0.212	0.201	0.190	0.181	0.173	0.165	0.159	0.152	0.144	0.136	0.129	0.123
160	0.249	0.240	0.231	0.223	0.216	0.209	0.203	0.191	0.181	0.171	0.163	0.155	0.149	0.142	0.137	0.129	0.122	0.116	0.110
170	0.227	0.218	0.210	0.203	0.196	0.190	0.184	0.173	0.164	0.155	0.147	0.140	0.134	0.128	0.123	0.116	0.110	0.104	0.099
180	0.207	0.199	0.192	0.185	0.179	0.173	0.167	0.157	0.149	0.141	0.134	0.127	0.122	0.116	0.111	0.105	0.099	0.094	0.089
190	0.190	0.183	0.176	0.169	0.164	0.158	0.153	0.144	0.136	0.128	0.122	0.116	0.111	0.106	0.101	0.095	0.090	0.085	0.081
200	0.175	0.168	0.162	0.156	0.150	0.145	0.140	0.132	0.124	0.118	0.112	0.106	0.101	0.097	0.093	0.087	0.082	0.078	0.074
210	0.161	0.155	0.149	0.143	0.138	0.134	0.129	0.121	0.114	0.108	0.102	0.097	0.093	0.089	0.085	0.080	0.075	0.071	0.068
220	0.149	0.143	0.138	0.133	0.128	0.123	0.119	0.112	0.105	0.100	0.094	0.090	0.086	0.082	0.078	0.074	0.069	0.066	0.062
230	0.138	0.133	0.128	0.123	0.118	0.114	0.110	0.104	0.097	0.092	0.087	0.083	0.079	0.075	0.072	0.068	0.064	0.061	0.058
240	0.129	0.123	0.119	0.114	0.110	0.106	0.103	0.096	0.090	0.085	0.081	0.077	0.073	0.070	0.067	0.063	0.059	0.056	0.053
250	0.120	0.115	0.110	0.106	0.102	0.099	0.095	0.089	0.084	0.079	0.075	0.071	0.068	0.065	0.062	0.058	0.055	0.052	0.049

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Table 9(A) Design Compressive Stress, f_{cd} (MPa) For Column Flexural Buckling Class a_0

(*Clauses* 14.1.2 *and* 14.1.2.3)

***KL*/r** ↓	**Yield Stress, f_y** (MPa)																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	191	200	209	218	226	235	253	271	289	306	324	342	359	377	403	429	456	482
30	179	188	197	205	214	223	231	249	266	283	300	317	334	351	368	393	418	443	467
40	176	185	193	202	210	218	227	243	260	276	292	308	324	340	356	379	402	424	446
50	173	181	189	197	205	213	221	236	252	267	282	296	310	324	338	358	377	395	412
60	168	176	184	191	198	206	213	227	240	253	266	278	289	300	310	324	337	349	359
70	163	170	176	183	189	195	201	213	223	233	242	251	258	265	271	279	286	292	297
80	155	160	166	171	176	181	185	194	201	207	212	217	221	225	228	232	236	239	241
90	144	148	152	156	160	163	166	171	175	178	182	184	186	188	190	192	194	196	197
100	131	134	137	139	141	143	145	148	150	152	154	155	157	158	159	160	161	162	163
110	117	119	120	122	123	124	125	127	129	130	131	132	133	133	134	135	135	136	136
120	103	104	105	106	107	108	109	110	111	111	112	113	113	114	114	115	115	115	116
130	91.0	91.7	92.4	93.0	93.5	94.0	94.4	95.2	95.8	96.3	96.8	97.2	97.6	97.9	98.2	98.6	98.9	99.2	99.4
140	80.3	80.8	81.2	81.6	82.0	82.3	82.6	83.2	83.6	84.0	84.3	84.6	84.9	85.1	85.3	85.6	85.9	86.1	86.3
150	71.1	71.5	71.8	72.1	72.3	72.6	72.8	73.2	73.5	73.8	74.0	74.3	74.5	74.6	74.8	75.0	75.2	75.4	75.5
160	63.3	63.6	63.8	64.0	64.2	64.4	64.6	64.8	65.1	65.3	65.5	65.7	65.8	66.0	66.1	66.3	66.4	66.5	66.7
170	56.6	56.8	57.0	57.2	57.3	57.5	57.6	57.8	58.0	58.2	58.3	58.5	58.6	58.7	58.8	58.9	59.1	59.2	59.3
180	50.9	51.1	51.2	51.3	51.5	51.6	51.7	51.9	52.0	52.1	52.3	52.4	52.5	52.6	52.6	52.7	52.8	52.9	53.0
190	46.0	46.1	46.2	46.3	46.4	46.5	46.6	46.8	46.9	47.0	47.1	47.2	47.3	47.3	47.4	47.5	47.6	47.6	47.7
200	41.7	41.9	41.9	42.0	42.1	42.2	42.2	42.4	42.5	42.6	42.6	42.7	42.8	42.8	42.9	43.0	43.0	43.1	43.2
210	38.0	38.1	38.2	38.3	38.3	38.4	38.5	38.6	38.6	38.7	38.8	38.8	38.9	39.0	39.0	39.1	39.1	39.2	39.2
220	34.8	34.9	34.9	35.0	35.1	35.1	35.2	35.2	35.3	35.4	35.4	35.5	35.5	35.6	35.6	35.7	35.7	35.8	35.8
230	32.0	32.0	32.1	32.1	32.2	32.2	32.3	32.3	32.4	32.4	32.5	32.5	32.6	32.6	32.7	32.7	32.7	32.8	32.8
240	29.4	29.5	29.5	29.6	29.6	29.7	29.7	29.8	29.8	29.9	29.9	29.9	30.0	30.0	30.0	30.1	30.1	30.2	30.2
250	27.2	27.3	27.3	27.3	27.4	27.4	27.4	27.5	27.5	27.6	27.6	27.6	27.7	27.7	27.7	27.8	27.8	27.8	27.9

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Table 9(B) Design Compressive Stress, f_{cd} (MPa) For Column Flexural Buckling Class a

(*Clauses* 14.1.2 *and* 14.1.2.3)

***KL*/r** ↓	**Yield Stress, f_y** (MPa)																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	213	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	191	200	208	217	226	235	252	270	287	305	322	339	357	374	400	425	451	476
30	178	186	195	203	212	220	229	245	262	279	295	311	328	344	360	384	408	431	454
40	173	181	189	197	205	213	221	237	253	268	283	298	313	328	342	363	384	405	425
50	168	176	183	191	198	205	213	227	241	255	268	281	294	306	318	336	352	368	383
60	162	169	175	182	189	195	202	214	226	237	248	258	268	278	286	299	310	320	329
70	154	160	166	171	177	182	188	197	207	215	223	230	237	243	249	256	263	268	274
80	144	149	154	158	163	167	171	178	184	190	195	199	204	207	210	215	219	222	225
90	133	137	140	143	146	149	152	157	161	164	168	170	173	175	177	179	182	184	185
100	120	123	125	128	130	132	133	136	139	141	143	145	146	148	149	151	152	153	154
110	107	109	111	112	114	115	116	118	120	121	123	124	125	126	127	128	129	129	130
120	95.5	96.7	97.9	98.9	100	101	101	103	104	105	106	107	107	108	109	109	110	110	111
130	84.6	85.5	86.3	87	87.7	88.3	88.8	89.8	90.6	91.3	92.0	92.5	93.0	93.5	93.9	94.4	94.9	95.3	95.7
140	75.2	75.8	76.4	76.9	77.4	77.8	78.2	78.9	79.5	80.0	80.5	80.9	81.3	81.6	81.9	82.3	82.6	83.0	83.2
150	67.0	67.4	67.9	68.2	68.6	68.9	69.2	69.7	70.2	70.6	70.9	71.2	71.5	71.8	72.0	72.3	72.6	72.9	73.1
160	59.9	60.3	60.6	60.9	61.1	61.4	61.6	62.0	62.4	62.7	62.9	63.2	63.4	63.6	63.8	64.0	64.3	64.5	64.6
170	53.8	54.1	54.3	54.6	54.8	55.0	55.1	55.5	55.7	56.0	56.2	56.4	56.6	56.7	56.9	57.1	57.3	57.4	57.6
180	48.6	48.8	49.0	49.2	49.3	49.5	49.6	49.9	50.1	50.3	50.5	50.6	50.8	50.9	51.0	51.2	51.3	51.5	51.6
190	44.0	44.2	44.3	44.5	44.6	44.7	44.9	45.1	45.3	45.4	45.6	45.7	45.8	45.9	46.0	46.2	46.3	46.4	46.5
200	40.0	40.2	40.3	40.4	40.5	40.7	40.7	40.9	41.1	41.2	41.3	41.4	41.5	41.6	41.7	41.8	41.9	42.0	42.1
210	36.6	36.7	36.8	36.9	37.0	37.1	37.2	37.3	37.4	37.6	37.7	37.8	37.8	37.9	38.0	38.1	38.2	38.3	38.3
220	33.5	33.6	33.7	33.8	33.9	34.0	34.0	34.2	34.3	34.4	34.5	34.5	34.6	34.7	34.7	34.8	34.9	35.0	35.0
230	30.8	30.9	31.0	31.1	31.2	31.2	31.3	31.4	31.5	31.6	31.6	31.7	31.8	31.8	31.9	31.9	32.0	32.1	32.1
240	28.5	28.5	28.6	28.7	28.7	28.8	28.8	28.9	29.0	29.1	29.1	29.2	29.3	29.3	29.4	29.4	29.5	29.5	29.6
250	26.3	26.4	26.5	26.5	26.6	26.6	26.7	26.7	26.8	26.9	26.9	27.0	27.0	27.1	27.1	27.2	27.2	27.3	27.3

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Table 9(C) Design Compressive Stress, f_{cd} (MPa) for Column Flexural Buckling Class b

(*Clauses* 14.1.2 *and* 14.1.2.3)

***KL*/r** ↓	**Yield Stress, f_y** (MPa)																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	199	208	217	225	234	251	268	285	302	319	336	353	369	394	419	443	468
30	175	183	192	200	208	216	224	240	256	271	287	302	318	333	348	370	392	414	435
40	168	176	183	191	198	206	213	228	242	256	270	283	297	310	323	342	360	378	395
50	161	167	174	181	188	194	201	214	226	238	250	261	272	283	293	308	322	335	347
60	152	158	164	170	176	181	187	197	207	217	226	235	243	251	259	269	279	287	295
70	142	147	152	157	162	166	171	179	187	194	201	207	213	218	223	230	236	241	246
80	131	135	139	143	147	150	154	160	165	170	175	179	183	186	190	194	198	201	204
90	120	123	126	129	131	134	136	141	144	148	151	154	156	159	161	163	166	168	170
100	108	110	112	114	116	118	120	123	126	128	130	132	134	135	137	139	140	142	143
110	96.5	98.3	100	101	103	104	105	107	109	111	112	114	115	116	117	118	119	121	121
120	86.2	87.5	88.6	89.7	90.7	91.7	92.5	94.1	95.4	96.6	97.7	98.6	100	100	101	102	103	104	104
130	76.9	77.8	78.7	79.5	80.3	81.0	81.6	82.7	83.7	84.6	85.4	86.1	86.8	87.3	87.9	88.6	89.2	89.8	90.3
140	68.7	69.4	70.1	70.7	71.3	71.8	72.3	73.1	73.9	74.6	75.2	75.7	76.2	76.6	77.1	77.6	78.1	78.5	78.9
150	61.6	62.1	62.6	63.1	63.6	64.0	64.3	65.0	65.6	66.1	66.6	67.0	67.4	67.7	68.1	68.5	68.9	69.2	69.5
160	55.4	55.8	56.2	56.6	56.9	57.3	57.5	58.1	58.5	59.0	59.3	59.7	60.0	60.3	60.5	60.9	61.2	61.5	61.7
170	50.0	50.3	50.7	51.0	51.2	51.5	51.7	52.2	52.5	52.9	53.2	53.5	53.7	53.9	54.1	54.4	54.7	54.9	55.1
180	45.3	45.6	45.9	46.1	46.3	46.5	46.7	47.1	47.4	47.7	47.9	48.1	48.3	48.5	48.7	48.9	49.2	49.3	49.5
190	41.2	41.5	41.7	41.9	42.1	42.2	42.4	42.7	42.9	43.2	43.4	43.6	43.7	43.9	44.0	44.2	44.4	44.6	44.7
200	37.6	37.8	38.0	38.2	38.3	38.5	38.6	38.9	39.1	39.3	39.5	39.6	39.8	39.9	40.0	40.2	40.3	40.5	40.6
210	34.5	34.7	34.8	35.0	35.1	35.2	35.3	35.5	35.7	35.9	36.0	36.2	36.3	36.4	36.5	36.6	36.8	36.9	37.0
220	31.7	31.9	32.0	32.1	32.2	32.3	32.4	32.6	32.8	32.9	33.0	33.1	33.2	33.3	33.4	33.6	33.7	33.8	33.9
230	29.2	29.4	29.5	29.6	29.7	29.8	29.9	30.0	30.1	30.3	30.4	30.5	30.6	30.7	30.7	30.8	30.9	31.0	31.1
240	27.1	27.2	27.3	27.3	27.4	27.5	27.6	27.7	27.8	27.9	28.0	28.1	28.2	28.3	28.3	28.4	28.5	28.6	28.7
250	25.1	25.2	25.3	25.3	25.4	25.5	25.6	25.7	25.8	25.9	26.0	26.0	26.1	26.2	26.2	26.3	26.4	26.5	26.5

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Table 9(D) Design Compressive Stress, f_{cd} (MPa) for Column Flexural Buckling Class c

(*Clauses* 14.1.2 *and* 14.1.2.3)

***KL*/r**	**Yield Stress, f_y, (MPa)**																		
↓	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	199	207	216	224	233	250	266	283	299	316	332	348	364	388	412	435	458
30	172	180	188	196	204	211	219	234	249	264	278	293	307	321	335	355	376	395	415
40	163	170	177	184	191	198	205	218	231	244	256	268	280	292	304	320	337	352	367
50	153	159	165	172	178	183	189	201	212	222	232	242	252	261	270	282	295	306	317
60	142	148	153	158	163	168	173	182	191	199	207	215	222	228	235	244	252	260	267
70	131	136	140	144	148	152	156	163	170	176	182	187	192	197	202	208	213	218	223
80	120	123	127	130	133	136	139	145	149	154	158	162	165	169	172	176	180	183	186
90	108	111	114	116	119	121	123	127	131	134	137	140	142	144	146	149	152	154	156
100	97.5	100	102	104	105	107	109	112	114	116	119	120	122	124	125	127	129	131	132
110	87.3	89.0	90.5	92.0	93.3	94.6	95.7	97.9	100	102	103	104	106	107	108	110	111	112	113
120	78.2	79.4	80.6	81.7	82.7	83.7	84.6	86.2	87.6	88.9	90.1	91.1	92.1	93.0	93.8	94.9	95.9	96.8	97.6
130	70.0	71.0	71.9	72.8	73.5	74.3	75.0	76.2	77.3	78.3	79.2	80.0	80.7	81.4	82.0	82.9	83.6	84.3	84.9
140	62.9	63.6	64.4	65.0	65.6	66.2	66.7	67.7	68.6	69.3	70.0	70.7	71.2	71.8	72.3	72.9	73.5	74.1	74.6
150	56.6	57.2	57.8	58.3	58.8	59.2	59.7	60.4	61.1	61.7	62.3	62.8	63.3	63.7	64.1	64.6	65.1	65.5	65.9
160	51.1	51.6	52.1	52.5	52.9	53.3	53.6	54.2	54.8	55.3	55.7	56.1	56.5	56.9	57.2	57.6	58.0	58.4	58.7
170	46.4	46.8	47.1	47.5	47.8	48.1	48.4	48.9	49.3	49.8	50.1	50.5	50.8	51.1	51.3	51.7	52.0	52.3	52.6
180	42.2	42.5	42.8	43.1	43.4	43.6	43.9	44.3	44.7	45.0	45.3	45.6	45.8	46.1	46.3	46.6	46.9	47.1	47.3
190	38.5	38.8	39.0	39.3	39.5	39.7	39.9	40.3	40.6	40.9	41.1	41.4	41.6	41.8	42.0	42.2	42.5	42.7	42.9
200	35.3	35.5	35.7	35.9	36.1	36.3	36.5	36.8	37.0	37.3	37.5	37.7	37.9	38.1	38.2	38.4	38.6	38.8	39.0
210	32.4	32.6	32.8	33.0	33.1	33.3	33.4	33.7	33.9	34.1	34.3	34.5	34.7	34.8	34.9	35.1	35.3	35.4	35.6
220	29.9	30.1	30.2	30.4	30.5	30.6	30.8	31.0	31.2	31.4	31.5	31.7	31.8	31.9	32.1	32.2	32.4	32.5	32.6
230	27.6	27.8	27.9	28.0	28.2	28.3	28.4	28.6	28.8	28.9	29.1	29.2	29.3	29.4	29.5	29.7	29.8	29.9	30.0
240	25.6	25.7	25.9	26.0	26.1	26.2	26.3	26.4	26.6	26.7	26.9	27.0	27.1	27.2	27.3	27.4	27.5	27.6	27.7
250	23.8	23.9	24.0	24.1	24.2	24.3	24.4	24.5	24.7	24.8	24.9	25.0	25.1	25.2	25.3	25.4	25.5	25.6	25.7

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Table 9(E) Design Compressive Stress, f_{cd} (MPa) for Column Buckling Class d

(*Clauses* 14.1.2 *and* 14.1.2.3)

***KL*/r** ↓	**Yield Stress, f_y** (MPa)																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	198	206	215	223	231	247	263	279	294	310	325	340	355	377	399	421	442
30	168	175	182	189	197	204	211	224	238	251	264	277	290	302	314	332	350	367	384
40	154	161	167	173	179	185	191	203	214	225	235	246	256	266	275	289	303	316	328
50	141	147	152	157	162	167	172	182	191	199	208	216	224	231	238	249	258	268	277
60	129	133	137	142	146	150	154	161	168	175	182	188	193	199	204	212	219	225	231
70	116	120	124	127	130	133	137	142	148	153	158	162	167	171	174	180	184	189	193
80	105	108	111	113	116	118	121	125	129	133	137	140	143	146	149	153	156	159	162
90	94.1	96.4	98.6	101	103	105	107	110	113	116	119	121	123	126	128	130	133	135	137
100	84.3	86.2	87.9	89.6	91.1	92.6	94.0	96.7	99.1	101	103	105	107	108	110	112	114	116	117
110	75.6	77.0	78.4	79.7	81.0	82.1	83.2	85.3	87.1	88.8	90.4	91.8	93.1	94.4	95.5	97.1	98.5	100	101
120	67.8	69.0	70.1	71.1	72.1	73.0	73.9	75.5	77.0	78.3	79.5	80.6	81.7	82.6	83.5	84.7	85.8	86.9	87.8
130	61.0	62.0	62.8	63.7	64.5	65.2	65.9	67.2	68.3	69.4	70.4	71.2	72.1	72.8	73.5	74.5	75.4	76.2	76.9
140	55.0	55.8	56.5	57.2	57.8	58.4	59.0	60.0	61.0	61.8	62.6	63.3	64.0	64.6	65.2	66.0	66.7	67.3	67.9
150	49.8	50.4	51.0	51.6	52.1	52.6	53.1	53.9	54.7	55.4	56.0	56.6	57.2	57.7	58.1	58.8	59.3	59.9	60.4
160	45.2	45.7	46.2	46.7	47.1	47.5	47.9	48.6	49.3	49.9	50.4	50.9	51.3	51.7	52.1	52.7	53.1	53.6	54.0
170	41.2	41.6	42.1	42.4	42.8	43.1	43.5	44.1	44.6	45.1	45.5	45.9	46.3	46.7	47.0	47.4	47.8	48.2	48.6
180	37.7	38.0	38.4	38.7	39.0	39.3	39.6	40.1	40.5	41.0	41.3	41.7	42.0	42.3	42.6	43.0	43.3	43.6	43.9
190	34.5	34.9	35.2	35.4	35.7	35.9	36.2	36.6	37.0	37.4	37.7	38.0	38.2	38.5	38.7	39.1	39.4	39.6	39.9
200	31.8	32.0	32.3	32.5	32.8	33.0	33.2	33.6	33.9	34.2	34.5	34.7	35.0	35.2	35.4	35.7	35.9	36.2	36.4
210	29.3	29.6	29.8	30.0	30.2	30.4	30.5	30.9	31.2	31.4	31.7	31.9	32.1	32.3	32.5	32.7	32.9	33.1	33.3
220	27.1	27.3	27.5	27.7	27.9	28.0	28.2	28.5	28.7	29.0	29.2	29.4	29.6	29.7	29.9	30.1	30.3	30.5	30.6
230	25.2	25.3	25.5	25.7	25.8	26.0	26.1	26.4	26.6	26.8	27.0	27.1	27.3	27.5	27.6	27.8	27.9	28.1	28.2
240	23.4	23.6	23.7	23.9	24.0	24.1	24.2	24.5	24.7	24.8	25.0	25.2	25.3	25.4	25.5	25.7	25.9	26.0	26.1
250	21.8	22.0	22.1	22.2	22.3	22.5	22.6	22.8	22.9	23.1	23.2	23.4	23.5	23.6	23.7	23.9	24.0	24.1	24.2

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Table 10 Buckling Class of Cross Sections

(*Clause* 14.1.2.3)

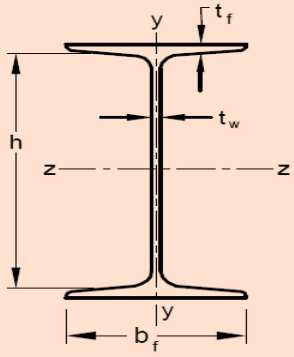
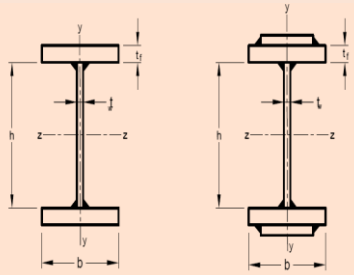
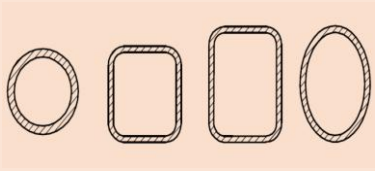
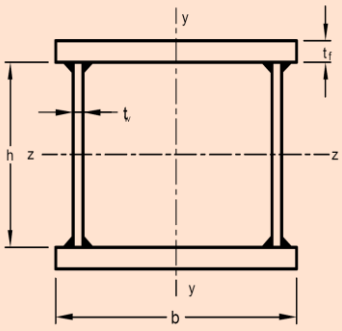
Cross Section	Limits		Buckling About Axis	Buckling Class	
				$f_y \leq 420$ MPa	$f_y > 420$ MPa
(1)	(2)		(3)	(4)	(5)
Rolled I-Sections 	$h/b_f > 1.2$	$t_f \leq 40$ mm	*z-z* *y-y*	a b	a_0 a
		$t_f > 40$mm	*z-z* *y-y*	b c	a b
	$h/b_f \leq 1.2$	$t_f \leq 100$ mm	*z-z* *y-y*	b c	a b
		$t_f > 100$ mm	*z-z* *y-y*	d d	c c
Welded I-Section 	$t_f \leq 40$ mm		*z-z* *y-y*	b c	b c
	$t_f > 40$ mm		*z-z* *y-y*	c d	c d
Hollow Section 	Hot rolled		any	a	a_0
	Cold formed		any	c	c
Welded Box Section, Cruciform column 	Generally (except as below)		any	b	b
	Thick welds $s_w > 0.5t_t$ and $b/t_f < 30$ and $h/t_w < 30$		any	c	c

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Table 10 (*Concluded*)

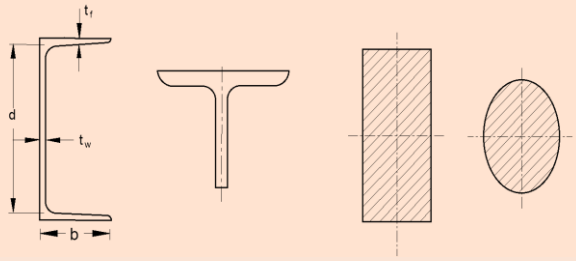
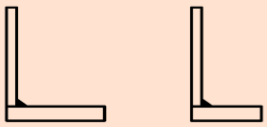
Cross Section	Limits	Buckling About Axis	Buckling Class	
			$f_y \leq 420$ MPa	$f_y > 420$ MPa
(1)	(2)	(3)	(4)	(5)
Channel, T, and Solid Sections 		any	c	c
Rolled angle or L sections		any	b	a
Welded angle or L sections $t \leq 40$ mm 		any	c	c

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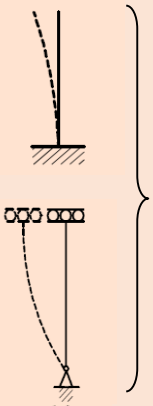
Table 11 Effective Length of Prismatic Compression Members

(*Clause* 14.2.2)

Boundary Conditions				Schematic Representation	Effective Length
At One End		At the Other End			
Translation	Rotation	Translation	Rotation		
(1)	(2)	(3)	(4)	(5)	(6)
Restrained	Restrained	Free	Free		2.0*L*
Restrained	Free	Free	Restrained		
Restrained	Free	Restrained	Free		1.0*L*
Restrained	Restrained	Free	Restrained		1.2*L*
Restrained	Restrained	Restrained	Free		0.8*L*
Restrained	Restrained	Restrained	Restrained		0.65*L*

NOTE — *L* is the unsupported length of the compression member (*see* 13.2.1).

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14.3.3.2 In continuous columns, the bending moments due to eccentricities of loading on the columns at any floor may be divided equally between the columns above and below that floor level, provided that the moment of inertia of one column section, divided by its effective length does not exceed 1.5 times the corresponding value of the other column. Where this ratio is exceeded, the bending moment shall be divided in proportion to the moment of inertia of the column sections divided by their respective effective lengths.

14.3.4 *Splices*

14.3.4.1 Where the ends of compression members are prepared for bearing over the whole area, they shall be spliced to hold the connected members accurately in position, and to resist bending or tension, if present. Such splices should maintain the intended member stiffness about each axis. Splices should be located as close to the point of inflection as possible. Otherwise their capacity should be adequate to carry magnified moment (*see* **16.3.2.2**). The ends of compression members faced for bearing shall invariably be machined to ensure perfect contact of surfaces in bearing.

14.3.4.2 Where such members are not faced for complete bearing, the splices shall be designed to transmit all the forces to which the members are subjected.

14.3.4.3 Wherever possible, splices shall be proportioned and arranged so that the centroidal axis of the splice coincides as nearly as possible with the centroidal axes of the members being jointed, in order to avoid eccentricity; but where eccentricity is present in the joint, the resulting stress shall be accounted for.

14.4 Column Bases

14.4.1 *General*

Column bases should have sufficient stiffness and strength to transmit axial force, bending moments and shear forces at the base of the columns to their foundation without exceeding the load carrying capacity of the supports. Anchor bolts and shear keys should be provided wherever necessary. Shear resistance at the proper contact surface between steel base and concrete/grout may be calculated using a friction coefficient of 0.45. Design recommendations of anchor bolts in concrete foundations may be referred as per good practice [C-6A(38)].

The nominal bearing pressure between the base plate and the support below may be determined on the basis of linearly varying distribution of pressure. The maximum bearing pressure on concrete under the limit states design load should not exceed the bearing strength equal to $0.3f_{ck}\,[\sqrt{(A_1/A_2)}]$ and $0.9f_{ck}$, whichever is smaller, where f_{ck} is the smaller of characteristic cube strength of concrete or bedding material, A_1 is the area of concrete pedestal on which the base plate is sitting, and A_2 is the surface area of the steel base plate transferring the load to the concrete pedestal.

If the size of the base plate is larger than that required to limit the bearing pressure on the base support, an equal projection, c of the base plate beyond the face of the column and gusset may be taken as effective in transferring the column load as given in Fig. 9, such that bearing pressure on the effective area does not exceed bearing capacity of concrete base.

14.4.2 *Gusseted Bases*

For stanchion with gusseted bases, the gusset plates, angle cleats, stiffeners, fastenings, etc, in combination with the bearing area of the shaft, shall be sufficient to take the loads, bending moments and reactions to the base plate without exceeding specified strength. All the bearing surfaces shall be machined to ensure perfect contact.

14.4.2.1 Where the ends of the column shaft and the gusset plates are not faced for complete bearing, the welds, fastenings connecting them to the base plate shall be sufficient to transmit all the forces to which the base is subjected.

14.4.2.2 *Column and base plate connections*

Where the end of the column is connected directly to the base plate by means of full penetration butt welds, the connection shall be deemed to transmit to the base all the forces and moments to which the column is subjected.

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14.4.3 *Slab Bases*

Columns with slab bases need not be provided with gussets, but sufficient fastenings shall be provided to retain the parts securely in place and to resist all moments and forces, other than direct compression, including those arising during transit, unloading and erection. Minimum of four number of anchor bolts shall be used to anchor column bases to foundation and minimum of six numbers in the case of slab bases transferring earthquake forces.

14.4.3.1 The minimum thickness, t_s, of rectangular slab bases, supporting columns under axial compression shall be

$$t_s = \sqrt{2.5\, w(a^2 - 0.3b^2)\gamma_{m0} / f_y} \quad > \; t_f$$

where

a, b = larger and smaller projection, respectively of the slab base beyond the rectangle circumscribing the column;

t_f = flange thickness of compression member; and

w = uniform pressure from below on the slab base under the factored load axial compression.

When only the effective area of the base plate is used as in **14.4.1**, c^2 may be used in the above equation (*see* Fig. 9) instead of $(a^2 - 0.3b^2)$.

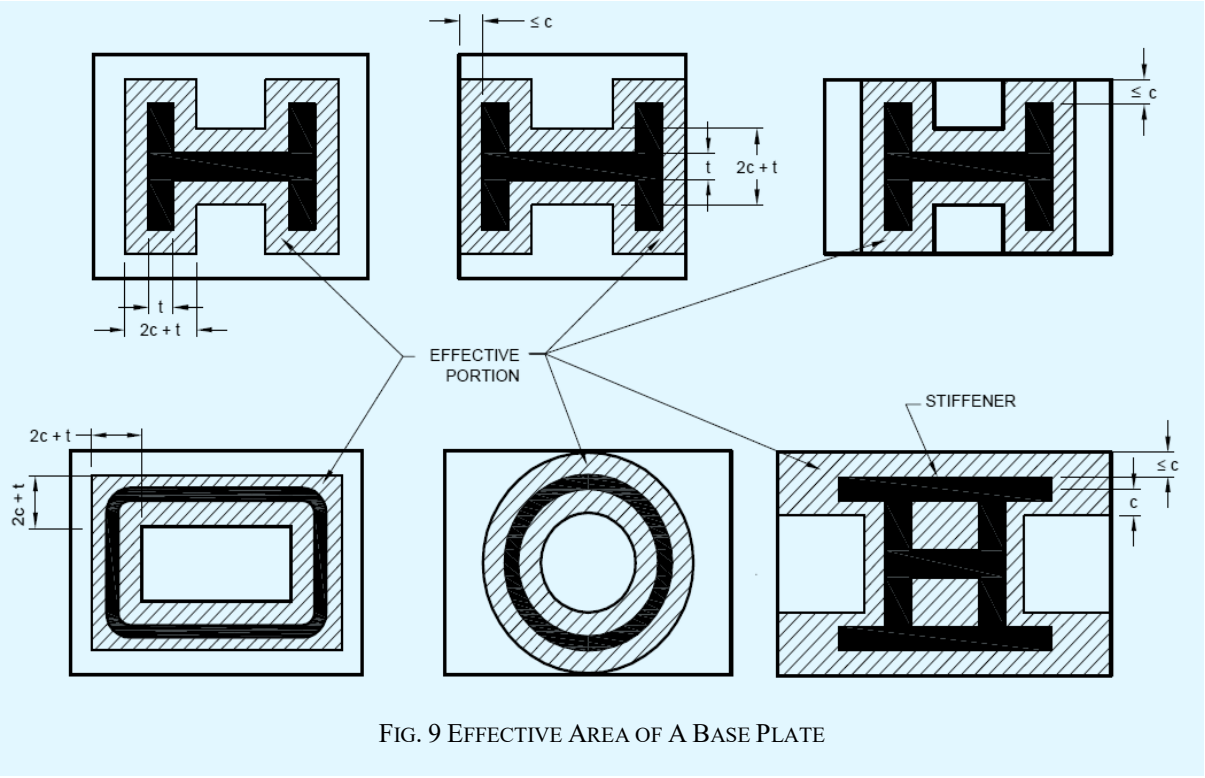

FIG. 9 EFFECTIVE AREA OF A BASE PLATE

14.4.3.2 When the slab does not distribute the column load uniformly, due to eccentricity of the load etc, special calculation shall be made to show that the base is adequate to resist the moment due to the non-uniform pressure from below.

14.4.3.3 Bases for bearing upon concrete or masonry need not be machined on the underside.

14.4.3.4 In cases where the cap or base is fillet welded directly to the end of the column without boring and shouldering, the contact surfaces shall be machined to give a perfect bearing and the welding shall be sufficient

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to transmit the forces as required in **14.4.3**. Where full strength butt welds are provided, machining of contact surfaces is not required.

14.5 Angle Struts

14.5.1 *Single Angle Struts*

The compression in single angles may be transferred either concentrically to its centroid through the end connections or eccentrically by connecting one of its legs to a gusset or adjacent member.

14.5.1.1 *Concentric loading*

When a single angle is concentrically loaded in compression, the design strength may be evaluated using **14.1.2**, by calculating the non-dimensional slenderness ratio, λ_{vv}, for buckling about the minor axis, *v*-*v*, as given by:

$$\lambda_{vv} = \frac{\left(\frac{l_{vv}}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$$

r_{vv} = radius of gyration of the angle section about the minor axis; and

l_{vv} = laterally unsupported length preventing translation normal to the minor axis

14.5.1.2 *Loaded through one leg*

The design strength of single angle members loaded in compression through one of its legs, f_{cde}, is affected by flexural torsional buckling and bending. The design compressive strength of such eccentrically loaded angle members, *in lieu of* a more exact second order design under combined bending axial compression, may be evaluated approximately as given below.

The design compressive strength, of single angles loaded through gusset connection to one leg parallel to the *a*-*a* axis (*a*-*a* axis being either the *z*-*z* or the *y*-*y* axis, depending on which leg is connected to the gusset) of the member may be calculated as:

$$f_{cde} = K_f \chi_{aa} f_y / \gamma_{m0} \quad \leq f_{cd} \text{ (given in 14.1.2)}$$

where

f_{cde} = Design compressive stress of single angle.

K_f = $k_1 + k_2\lambda_{aa} + k_3\lambda_\phi$;

χ_{aa} = Stress reduction factor (*see* Table 8) for buckling class 'b', calculated as per **14.1.2.1**, using the non-dimensional slenderness ratio, λ_{aa}.

where

k_1, k_2, k_3 = constants depending upon the end condition, as given in Table 12; and

K_f = modification factor to account for eccentric end connection.

$$\lambda_{aa} = \frac{\left(\frac{l_{aa}}{r_{aa}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$$

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$$\lambda_{\varphi} = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$$

b_1, b_2 = width of the connected and outstanding legs of the angle, respectively;

l_{aa} = centre to centre length of the lateral support preventing translation of the member perpendicular to a-a axis (a-a axis being either the z-z or the y-y axis, depending on which leg is connected to the gusset);

r_{aa} = radius of gyration of the angle member about the *a-a* axis (*a-a* axis being either the *z-z* or the *y-y* axis, depending on which leg is connected to the gusset), parallel to the plane of the end gusset plates;

t = thickness of the leg; and

ε = is the yield stress ratio, $\sqrt{\frac{250}{f_y}}$, f_y being in MPa

Table 12 Constants k_1, k_2, and k_3

(*Clause* 14.5.1.2)

Sl No.	End Connection	Gusset/Connecting Member Fixity [1)]	k_1	k_2	k_3
(1)	(2)	(3)	(4)	(5)	(6)
i)	Fully welded or connected with two or more bolts	Fixed	0.798	0.563	-2.072
		Hinged	0.401	0.420	-1.040
ii)	Single bolt	Fixed	0.418	0.547	-1.400
		Hinged	0.374	0.415	-2.072

[1)] In-plane rotational restraint provided to the gusset by the supporting member. For partial restraint, k_f can be interpolated between the results for fixed and hinged cases.

14.5.2 *Double Angle Struts*

14.5.2.1 For double angle discontinuous struts, connected back to back, on opposite sides of the gusset or a section, by not less than two bolts or rivets in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially. The effective length, *KL*, in the plane of end gusset shall be taken as between 0.7 times and 0.85 times the distance between intersections, depending on the degree of the restraint provided. The effective length, *KL*, in the plane perpendicular to that of the end gusset, shall be taken as equal to the distance between centres of intersections. The calculated average compressive stress shall not exceed the values based on **14.1.2**. The angles shall be connected together over their lengths so as to satisfy the requirements of **14.8** and **17.2.5**.

14.5.2.2 Double angle discontinuous struts connected back-to-back, to one side of a gusset or section by one or more bolts or rivets in each angle, or by the equivalent in welding, shall be designed in accordance with **14.5.1** and the angles shall be connected together over their lengths so as to satisfy the requirements of **14.8** and **17.2.5**.

14.5.3 *Continuous Members*

Double angle continuous struts such as those forming the flanges, chords or ties of trusses or trussed girders, or the legs of towers shall be designed as axially loaded compression members, and the effective length shall be taken in accordance with **14.2.4**.

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14.5.4 *Combined Stresses*

In addition to axial loads, if the struts carry loads which cause transverse bending, the combined bending and axial stresses shall be checked in accordance with **16.3**. For determining the permissible axial and bending stresses, the effective length shall be taken in accordance with the **14.2** and **15.3**.

14.6 Laced Columns

14.6.1 *General*

14.6.1.1 Members comprising two main components laced and tied, should where practicable, have a radius of gyration about the axis perpendicular to the plane of lacing not less than the radius of gyration about the axis parallel to the plane of lacing [*see* Fig.10(A) and Fig. 10(B)].

14.6.1.2 As far as practicable, the lacing system shall be uniform throughout the length of the column.

14.6.1.3 Except for tie plates as specified in **14.7**, double laced systems [*see* Fig. 10(B)] and single laced systems [*see* Fig. 10(A)] on opposite sides of the main components shall not be combined with cross members (ties) perpendicular to the longitudinal axis of the strut [*see* Fig. 10(C)], unless all forces resulting from deformation of the strut members are calculated and provided for in the design of lacing and its fastenings.

14.6.1.4 Single laced systems, on opposite faces of the components being laced together shall preferably be in the same direction so that one is the shadow of the other, instead of being mutually opposed in direction.

14.6.1.5 The effective slenderness ratio, $(KL/r)_e$, of laced columns shall be taken as 1.05 times the $(KL/r)_0$, the actual maximum slenderness ratio, in order to account for shear deformation effects.

14.6.2 *Width of Lacing Bars*

In bolted/riveted construction, the minimum width of lacing bars shall be three times the nominal diameter of the end bolt/rivet.

14.6.3 *Thickness of Lacing Bars*

The thickness of flat lacing bars shall not be less than one-fortieth of its effective length for single lacings and one-sixtieth of the effective length for double lacings.

Rolled sections or tubes of equivalent strength may be permitted instead of flats, for lacings.

14.6.4 *Angle of Inclination*

Lacing bars, whether in double or single systems, shall be inclined at an angle not less than 40º nor more than 70º to the axis of the built-up member.

14.6.5 *Spacing*

14.6.5.1 The maximum spacing of lacing bars, whether connected by bolting, riveting or welding, shall also be such that the maximum slenderness ratio of the components of the main member (a_1/r_1), between consecutive lacing connections is not greater than 50 or 0.7 times the most unfavourable slenderness ratio of the member as a whole, whichever is less, where a_1 is the unsupported length of the individual member between lacing points, and r_1 is the minimum radius of gyration of the individual member being laced together.

14.6.5.2 Where lacing bars are not lapped to form the connection to the components of the members, they shall be so connected that there is no appreciable interruption in the triangulation of the system.

14.6.6 *Design of Lacings*

14.6.6.1 The lacing shall be proportioned to resist a total transverse shear, V_t, at any point in the member, equal to at least 2.5 percent of the axial force in the member and shall be divided equally among all transverse-lacing systems in parallel planes.

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14.6.6.2 For members carrying calculated bending stress due to eccentricity of loading, applied end moments and/or lateral loading, the lacing shall be proportioned to resist the actual shear due to bending, in addition to that specified in **14.6.6.1**.

14.6.6.3 The slenderness ratio, *KL/r*, of the lacing bars shall not exceed 145. In bolted/riveted construction, the effective length of lacing bars for the determination of the design strength shall be taken as the length between the inner end fastener of the bars for single lacing, and as 0.7 of this length for double lacings effectively connected at intersections. In welded construction, the effective lengths shall be taken as 0.7 times the distance between the inner ends of welds connecting the single lacing bars to the members.

NOTE — The required section for lacing bars as compression/tension members shall be determined by using the appropriate design stresses, f_{cd} subject to the requirements in **14.6.3** to **14.6.6** and T_d in **13.1**.

14.6.7 *Attachment to Main Members*

The bolting, riveting or welding of lacing bars to the main members shall be sufficient to transmit the force calculated in the bars. Where welded lacing bars overlap the main members, the amount of lap measured along either edge of the lacing bar shall be not less than four times the thickness of the bar or the thickness of the element of the members to which it is connected, whichever is less. The welding should be sufficient to transmit the load in the bar and shall, in any case, be provided along each side of the bar for the full length of lap.

14.6.8 *End Tie Plates*

Laced compression members shall be provided with tie plates as per **14.7** at the ends of lacing systems and at intersection with other members/stays and at points where the lacing systems are interrupted.

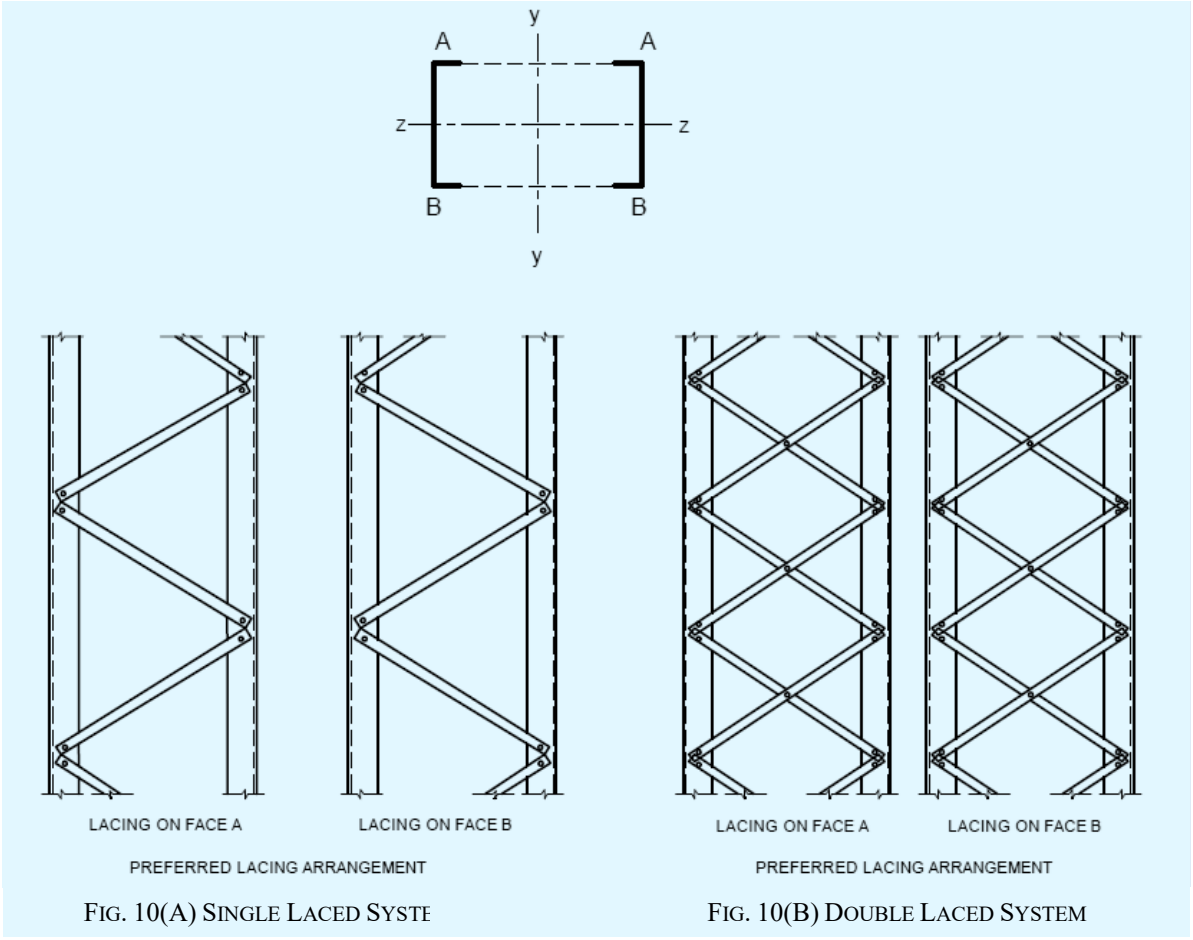

FIG. 10(A) SINGLE LACED SYSTE

FIG. 10(B) DOUBLE LACED SYSTEM

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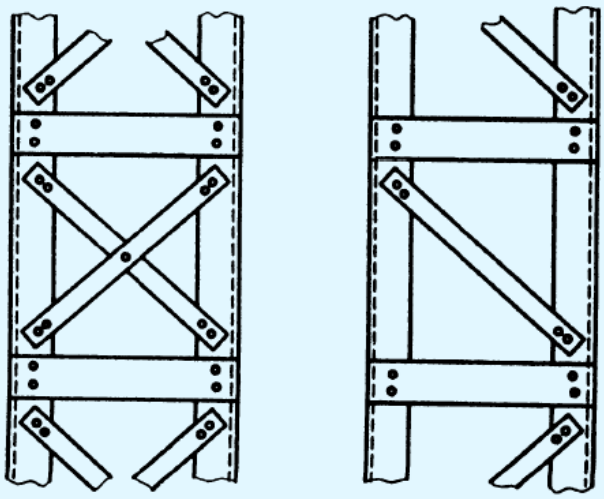

FIG. 10(C) DOUBLE LACED AND SINGLE LACED SYSTEM COMBINED WITH CROSS MEMBERS

FIG. 10 LACED COLUMNS

14.7 Battened Columns

14.7.1 *General*

14.7.1.1 Compression members composed of two main components battened should preferably have the individual members of the same cross section and symmetrically disposed about their major axis. Where practicable, the compression members should have a radius of gyration about the axis perpendicular to the plane of the batten not less than the radius of gyration about the axis parallel to the plane of the batten (*see* Fig. 11).

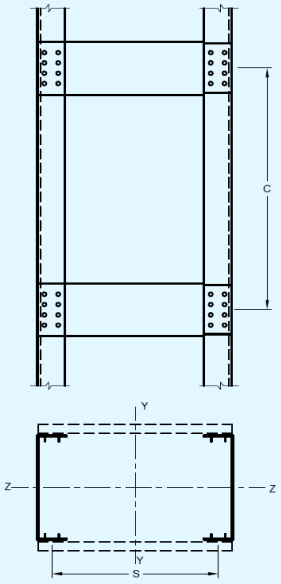

FIG. 11 BATTEN COLUMN SECTION

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14.7.1.2 Battened compression members, not complying with the requirements specified in this Section or those subjected to eccentricity of loading, applied moments or lateral forces in the plane of the battens (*see* Fig. 11), shall be designed according to the exact theory of elastic stability or empirically, based on verification by tests.

NOTE — If the column section is subjected to eccentricity or other moments about an axis perpendicular to battens, the battens and the column section should be specially designed for such moments and shears.

14.7.1.3 The battens shall be placed opposite to each other at each end of the member and at points where the member is stayed in its length and as far as practicable, be spaced and proportioned uniformly throughout. The number of battens shall be such that the member is divided into not less than three bays within its actual length from centre to centre of end connections.

14.7.1.4 The effective slenderness ratio $(KL/r)_e$ of battened columns, shall be taken as 1.1 times the $(KL/r)_0$, the maximum actual slenderness ratio of the column, to account for shear deformation effects.

14.7.2 *Design of Battens*

14.7.2.1 *Battens*

Battens shall be designed to carry the bending moments and shear forces arising from transverse shear force V_t equal to 2.5 percent of the total axial force on the whole compression member, at any point in the length of the member, divided equally between parallel planes of battens. Battened member carrying calculated bending moment due to eccentricity of axial loading, calculated end moments or lateral loads parallel to the plane of battens, shall be designed to carry actual shear in addition to the above shear. The main members shall also be checked for the same shear force and bending moments as for the battens.

Battens shall be of plates, angles, channels, or I-sections and at their ends shall be riveted, bolted or welded to the main components so as to resist simultaneously a shear $V_b = V_t\ C/NS$ along the column axis and a moment $M = V_t C/2N$ at each connection,

where

C = distance between centre-to-centre of battens, longitudinally;

N = number of parallel planes of battens;

S = minimum transverse distance between the centroid of the rivet/bolt group/welding connecting the batten to the main member; and

V_t = transverse shear force as defined above.

14.7.2.2 *Tie plates*

Tie plates are members provided at the ends of battened and laced members, and shall be designed by the same method as battens. In no case shall a tie plate and its fastenings be incapable of carrying the forces for which the lacing or batten has been designed.

14.7.2.3 *Size*

When plates are used for battens, the end battens and those at points where the member is stayed in its length shall have an effective depth, longitudinally, not less than the perpendicular distance between the centroids of the main members. The intermediate battens shall have an effective depth of not less than three quarters of this distance, but in no case shall the effective depth of any batten be less than twice the width of one member, in the plane of the battens. The effective depth of a batten shall be taken as the longitudinal distance between outermost bolts, rivets or welds at the ends. The thickness of batten or the tie plates shall be not less than one fiftieth of the distance between the innermost connecting lines of rivets, bolts or welds, perpendicular to the main member.

14.7.2.4 The requirement of bolt size and thickness of batten specified above does not apply when angles, channels or I-sections are used for battens with their legs or flanges perpendicular to the main member. However, it should be ensured that the ends of the compression members are tied to achieve adequate rigidity.

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14.7.3 *Spacing of Battens*

In battened compression members where the individual members are not specifically checked for shear stress and bending moments, the spacing of battens, centre-to-centre of its end fastenings, shall be such that the slenderness ratio (KL/r) of any component over that distance shall be not greater than 50 nor greater than 0.7 times the slenderness ratio of the member as a whole about its z-z axis (parallel to the battens).

14.7.4 *Attachment to Main Members*

14.7.4.1 *Welded connections*

Where tie or batten plates overlap the main members, the amount of lap shall be not less than four times the thickness of the plate. The length of weld connecting each edge of the batten plate to the member shall, in aggregate, be not less than half the depth of the batten plate. At least one-third of the weld shall be placed at each end of this edge. The length of weld and depth of batten plate shall be measured along the longitudinal axis of the main member.

In addition, the welding shall be returned along the other two edges of the plates transversely to the axis of the main member for a length not less than the minimum lap specified above.

14.8 Compression Members Composed of Two Components Back-to-Back

14.8.1 Compression members composed of two angles, channels, or tees back-to-back in contact or separated by a small distance, shall be connected together by riveting, bolting or welding so that the ratio of most unfavorable slenderness of each member between the intermediate connections is not greater than 40 or 0.6 times the most unfavorable ratio of slenderness of the strut as a whole, whichever is less [*see* **17**].

14.8.2 In no case shall the ends of the strut be connected together with less than two rivets or bolts or their equivalent in welding, and there shall be not less than two additional connections in between, spaced equidistant along the length of the strut. Where the members are separated back-to-back, the rivets or bolts through these connections shall pass through solid washers or packing in between. Where the legs of the connected angles or the connected tees are 125 mm wide or more, or where webs of channels are 150 mm wide or over, not less than two rivets or bolts shall be used in each connection, one on line of each gauge mark.

14.8.3 Where these connections are made by welding, solid packing shall be used to effect the jointing unless the members are sufficiently close together to permit direct welding, and the members shall be connected by welding along both pairs of edges of the main components.

14.8.4 The rivets, bolts or welds in these connections shall be sufficient to carry the shear force and moments, if any, specified for battened struts (*see* **14.7.3**), and in no case shall the rivets or bolts be less than 16 mm diameter for members up to and including 10 mm thick; 20 mm diameter for members up to and including 16 mm thick; and 22 mm diameter for members over 16 mm thick.

Compression members connected by such riveting, bolting or welding shall not be subjected to transverse loading in a plane perpendicular to the riveted, bolted or welded surfaces.

14.8.5 Where the components are in contact back-to-back, the spacing of the rivets, bolts or intermittent welds shall not exceed the maximum spacing for compression members as given under **17**.

15 DESIGN OF MEMBERS SUBJECTED TO BENDING

15.1 General

Members subjected to predominant bending shall have adequate design strength to resist bending moment, shear force, and concentrated forces imposed upon and their combinations. Further, the members shall satisfy the deflection limitation presented under **12**, as serviceability criteria. Member subjected to other forces in addition to bending or biaxial bending shall be designed in accordance with the provisions under **16**.

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15.1.1 *Effective Span of Beams*

The effective span of a beam shall be taken as the distance between the centres of the supports, except where the point of application of the reaction is taken as eccentric at the support, when it shall be permissible to take the effective span as the length between the assumed lines of the reactions.

15.2 Design Strength in Bending (Flexure)

The design bending strength of beam, adequately supported against lateral torsional buckling (laterally supported beam) is governed by the yield stress (*see* 15.2.1). When a beam is not adequately supported against lateral buckling (laterally un-supported beams) the design bending strength may be governed by lateral torsional buckling strength (*see* 15.2.2).

The factored design moment, M at any section, in a beam due to external actions, shall satisfy:

$$M \leq M_d$$

where

M_d = design bending strength of the section, calculated as given in 15.2.1.2.

15.2.1 *Laterally Supported Beam*

A beam may be assumed to be adequately supported at the supports, provided the compression flange has full lateral restraint and nominal torsional restraint at supports supplied by web cleats, partial depth end plates, fin plates or continuity with the adjacent span. Full lateral restraint to compression flange may be assumed to exist if the frictional or other positive restraint of a floor connection to the compression flange of the member is capable of resisting a lateral force not less than 2.5 percent of the maximum force in the compression flange of the member. This may be considered to be uniformly distributed along the flange, provided gravity loads constitute the dominant loading on the member and the floor construction is capable of resisting this lateral force.

The design bending strength of a section which are not susceptible to web buckling under shear before yielding (where $d/t_w \leq 67\varepsilon$) shall be determined according to 15.2.1.2.

15.2.1.1 *Section with webs susceptible to shear buckling before yielding*

When the flanges are plastic, compact or semi-compact but the web is susceptible to shear buckling before yielding (*see* 15.4.2.1), the design bending strength shall be calculated using one of the following methods:

a) the bending moment and axial force acting on the section may be assumed to be resisted by flanges only and the web is designed only to resist shear (*see* 15.4); and

b) the bending moment and axial force acting on the section may be assumed to be resisted by the whole section. In such a case, the web shall be designed for combined shear and normal stresses using simple elastic theory in case of semi-compact webs and simple plastic theory in the case of compact and plastic webs.

15.2.1.2 When the factored design shear force does not exceed 0.6 V_d, where V_d is the design shear strength of the cross section (*see* 15.4), the design bending strength, M_d shall be taken as:

$$M_d = \beta_b Z_p f_y / \gamma_{m0}$$

To avoid irreversible deformation under serviceability loads, M_d shall be less than $1.2 Z_e f_y / \gamma_{m0}$ in case of simply supported and cantilever beams and $1.5 Z_e f_y / \gamma_{m0}$ in all other cases. of;

where

β_b = 1.0 for plastic and compact sections;

β_b = Z_e / Z_p for semi-compact sections;

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f_y = yield stress of the material;

γ_{m0} = partial safety factor (*see* 12.4.1); and

Z_p, Z_e = plastic and elastic section moduli of the cross section, respectively

15.2.1.3 When the design shear force (factored), V exceeds $0.6V_d$, where V_d is the design shear strength of the cross section (*see* 15.4) the design bending strength, M_d shall be taken as:

$$M_d = M_{dv}$$

where

M_{dv} = design bending strength under high shear as defined in 16.2.

15.2.1.4 *Holes in the tension zone*

a) The effect of holes in the tension flange, on the design bending strength need not be considered if:

$$(A_{nf}/A_{gf}) \geq (f_y/f_u)\,(\gamma_{m1}/\gamma_{m0})\,/\,0.9$$

where

A_{nf}/A_{gf} = ratio of net to gross area of the flange in tension;

f_y/f_u = ratio of yield and ultimate stress of the material; and

γ_{m1}/γ_{m0} = ratio of partial safety factors against ultimate to yield stress (*see* 12.4.1).

When the A_{nf}/A_{gf} does not satisfy the above requirement, the reduced effective flange area, A_{ef} satisfying the above equation may be taken as the effective flange area in tension, instead of A_{gf}.

b) The effect of holes in the tension region of the web on the design flexural strength need not be considered, if the limit given in 15.2.1.4(a) above is satisfied for the complete tension zone of the cross section, comprising the tension flange and tension region of the web; and

c) Fastener holes in the compression zone of the cross section need not be considered in design bending strength calculation, except for oversize and slotted holes or holes without any fastener.

15.2.1.5 *Shear lag effects*

The shear lag effects in flanges may be disregarded provided:

a) For outstand elements (supported along one edge), $b_o \leq L_o / 20$; and

b) For internal elements (supported along two edges), $b_i \leq L_o / 10$.

where

b_o = width of the flange with outstand;

b_i = width of the flange as an internal element; and

L_o = length between points of zero moment (inflection) in the span.

Where these limits are exceeded, the effective width of flange for design strength may be calculated using specialist literature, or conservatively taken as the value satisfying the limit given above.

15.2.2 *Laterally Unsupported Beams*

Resistance to lateral torsional buckling need not be checked separately (member may be treated as laterally supported, *see* 15.2.1) in the following cases:

a) bending is about the minor axis of the section;

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b) section is hollow (rectangular/ tubular) or solid bars; and

c) in case of major axis bending, λ_{LT} (as defined herein) is less than 0.4.

The design bending strength of laterally unsupported beam as governed by lateral torsional buckling is given by:

$$M_d = \beta_b Z_p f_{bd}$$

where

β_b = 1.0 for plastic and compact sections;

= Z_e/Z_p for semi-compact sections (Class 3);

= Z_{eff}/Z_p for slender sections (Class 4);

f_{bd} = $\chi_{LT} f_y / \gamma_{m0}$;

= design bending compressive stress;

f_m = the moment gradient effect parameter given in the Table 14;

χ_{LT} = bending stress reduction factor to account for lateral torsional buckling, in doubly symmetrical sections with lateral support at the ends given by;

$$= \frac{f_m}{\left\{\varphi_{LT}+\left[\varphi_{LT}^2 - f_m\lambda_{LT}^2\right]^{0.5}\right\}} \leq 1.0 \ ;$$

Z_p, Z_e, Z_{eff} = plastic section modulus and elastic section modulus (Class 3) and effective section modulus (class 4) with respect to extreme compression fibre;

α_{LT} = the imperfection parameter is given in Table 13; and

$$\varphi_{LT} = 0.5\left[1 + f_m\left(\left(\frac{\lambda_{LT}}{\lambda_y}\right)^2 \alpha_{LT}(\lambda_y - 0.2) + {\lambda_{LT}}^2\right)\right];$$

The non-dimensional slenderness ratio, λ_{LT}, is given by:

$$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{crLT}} \leq \sqrt{1.2 Z_e f_y / M_{crLT}} = \sqrt{f_y / f_{crLT}} \qquad \lambda_y = \sqrt{f_y / f_{cry}}$$

where

f_{cry} = the non-dimensional weak axis slenderness ratio for laterally supported length equal to distance between lateral supports to the compression flange;

f_{crLT} = extreme fibre bending compressive stress corresponding to elastic lateral torsional buckling moment (*see* **15.2.2.1**); and

M_{crLT} = elastic lateral torsional buckling moment calculated in accordance with **15.2.2.1**.

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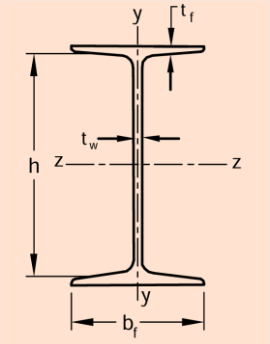
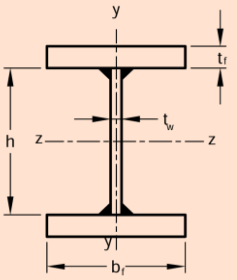
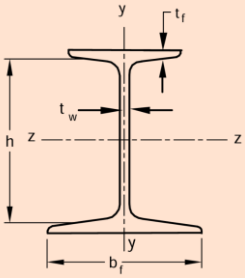
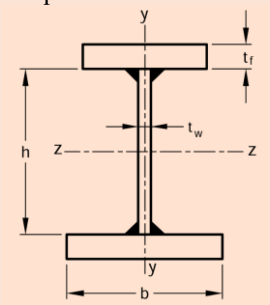
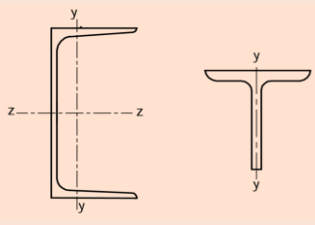
Table 13 Imperfection Parameter, α_{LT} for Lateral Torsional Buckling of Beams

(*Clause* 15.2.2)

Cross Section	Limits		α_{LT}
Rolled I or H Sections 	$h/b > 1.2$	$t_f \leq 40$ mm	$0.12\sqrt{\frac{Z_{ez}}{Z_{ey}}} \leq 0.34$
		$t_f > 40$ mm	$0.16\sqrt{\frac{Z_{ez}}{Z_{ey}}} \leq 0.49$
	$h/b \leq 1.2$	—	$0.16\sqrt{\frac{Z_{ez}}{Z_{ey}}} \leq 0.49$
Welded I Sections 	$t_f \leq 40$ mm		$0.21\sqrt{\frac{Z_{ez}}{Z_{ey}}} \leq 0.64$
	$t_f > 40$ mm		$0.25\sqrt{\frac{Z_{ez}}{Z_{ey}}} \leq 0.76$
Rolled unequal I or H Sections 	$h/b_{min} \leq 2.0$		0.21
	$h/b_{min} > 2.0$		0.34
Welded unequal I section 	$h/b_{min} \leq 2.0$		0.64
	$h/b_{min} > 2.0$		0.76
Other rolled Sections 	-------		0.76

NOTE — The selection of lateral torsional buckling imperfection factor is based on b_{min} irrespective of it being tension or compression flange.

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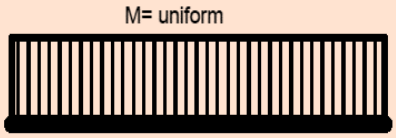
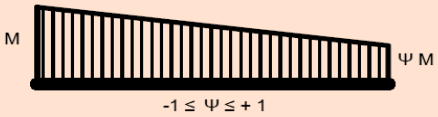
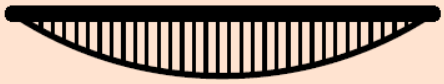
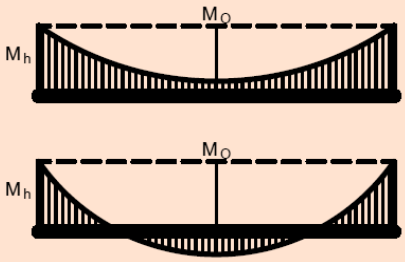
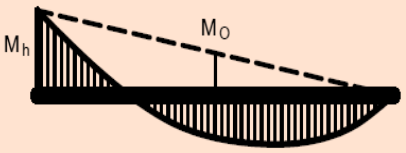
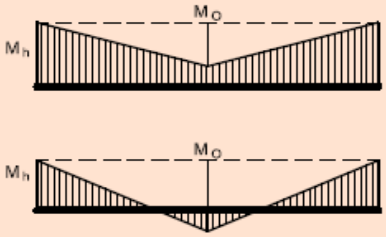
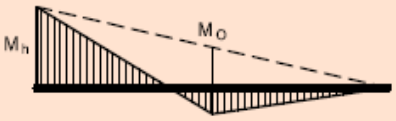
Table 14 Moment Gradient Effect Parameter

(*Clauses* 15.2.2 *and* 15.2.2.2)

Load Case	Factor f_M (*Clause* 15.2.2)	Factor k_c (Clause 15.2)
(1)	(2)	(3)
	1.0	1.0
	$1.25 - 0.1\psi - 0.15\ \psi^2$	$\frac{1}{1.33 - 0.33\ \psi}$
	1.05	0.94
	For $0 \leq \frac{M_0}{M_h} < 2.0$: $1.0 + 1.35\ \frac{M_0}{M_h} - 0.33\ \left[\frac{M_0}{M_h}\right]^3$ For $\frac{M_0}{M_h} \geq 2.0$: 1.05	$\frac{M_0}{M_h} < 1.0$: 1.0 $\frac{M_0}{M_h} \geq 1.0$: 0.9
	For $0 \leq \frac{M_0}{M_h} < 1.47$: $1.25 + 0.5\ \left[\frac{M_0}{M_h}\right]^2 - 0..275\ \left[\frac{M_0}{M_h}\right]^4$ For $\frac{M_0}{M_h} \geq 1.47$: 1.05	$\frac{M_0}{M_h} < 0.5$: 0.75 $\frac{M_0}{M_h} \geq 0.5$: 0.91
	1.10	0.86
	For $0 \leq \frac{M_0}{M_h} < 2.0$: $1.0 + 1.25\ \frac{M_0}{M_h} - 0.30\ \left[\frac{M_0}{M_h}\right]^3$ For $\frac{M_0}{M_h} \geq 2.0$: 1.10	$\frac{M_0}{M_h} < 1.0$: 1.0 $\frac{M_0}{M_h} \geq 1.0$: 0.77
	For $0 \leq \frac{M_0}{M_h} < 1.5$: $1.25 + 0.325\ \left[\frac{M_0}{M_h}\right]^2 - 0.175\ \left[\frac{M_0}{M_h}\right]^4$ For $\frac{M_0}{M_h} \geq 1.50$: 1.10	$\frac{M_0}{M_h} < 0.5$: 0.75 $\frac{M_0}{M_h} \geq 0.5$: 0.82

Where

M_0 is the bending moment at midspan, and may be determined by subtracting the average of the tow hogging end moments, form the midspan sagging moment for simply supported boundary condition.

For $\frac{M_0}{M_h} < 0.0$, f_m and k_c may be conservatively taken as 1.0.

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15.2.2.1 *Elastic lateral torsional buckling moment*

In case of simply supported, prismatic members with doubly symmetric cross section subjected to uniform moment, the elastic lateral torsional buckling moment, $M_{crLT,}$ can be determined from:

$$M_{crLT} = \sqrt{\left\{\left(\frac{\pi^2 E I_y}{(L_{LT})^2}\right)\left(G\, I_t + \frac{\pi^2 E\, I_w}{(L_{LT})^2}\right)\right\}}$$

where

L_{LT} = effective lateral unbraced length for lateral torsional buckling (*see* **15.3**);

I_t = torsional constant $= \sum b_i t_i^3 / 3$ for open section;

I_w = warping constant; and

I_y = moment of inertia about the weaker, (y), axis.

M_{cr} for different beam sections, considering loading, support condition, and asymmetric section, shall be more accurately calculated using the method given in Annex E. The above equations for calculating M_{cr} may be conservatively used for channel sections.

15.2.2.2 *Simplified method for lateral torsional buckling*

A simplified method based on effective compression flange lateral buckling as given below may be used for monosymmetric and doubly symmetric I and H sections with and without lateral restraints, to determine the equivalent compression flange lateral buckling stress reduction factor, $\chi_{LTc,,}$ that is substituted in equation given in **15.2.2** for calculating the design bending strength, M_d,

The stress reduction factor for lateral buckling of equivalent compression flange, χ_{LTc}, is evaluated using equations given in **14.1.2.1** for compression members, by treating the equivalent compression flange as a strut and substituting equivalent non-dimensional slenderness , $\lambda_{LTc,}$, in the place of λ.

where

$$\lambda_{LTc} = k_c \beta_c \lambda_{c,z}$$

k_c is a slenderness correction factor for moment gradient between restraints, as given in Table 14.

$$\beta_c = \sqrt{\frac{0.06\,\frac{h}{t_{f,max}}}{\lambda_{c,z} + \frac{t_{f,max}}{t_{f,min}}}}$$

$t_{f,max}$ is the maximum thickness of the top and bottom flange of the section

$t_{f,min}$ is the minimum thickness of the top and bottom flange of the section

$$\lambda_{c,z} = \sqrt{\frac{f_y}{f_{cr,cz}}}$$

where

A_c = is the area of equivalent compression flange, given by;

A_c = $A_{cf}+A_w/2$ for load applied at compression flange;

A_c = $A_{cf}+A_w/6$ for load applied at shear centre of the beam section;

A_c = A_{cf} for load applied at tension flange;

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A_{cf} = is the area of compression flange (= b_c t_{fc});

A_w = is the area of the web (=h t_w); and

$f_{cr,cz}$ = is the elastic critical axial stress of the equivalent compression flange of area of the baeam, A_c, as given below, buckling about the centre line axis of the web as a strut.

15.3 Effective Length for Lateral Torsional Buckling

15.3.1 For simply supported beams and girders of span length, L, where no lateral restraint to the compression flanges is provided, but where each end of the beam is restrained against torsion, the effective length L_{LT} of the lateral buckling to be used in **15.2.2.1** shall be taken as in Table 15.

In simply supported beams with intermediate lateral restraints against lateral torsional buckling, the effective length for lateral torsional buckling, L_{LT} to be used in **15.2.2.1**, may be conservatively taken as the length of the relevant segment in between the lateral restraints, or can be calculated using other known methods or rigorous analysis**.** In the case of intermediate partial lateral restraints, the effective length, L_{LT} shall be taken as equal to 1.2 times the length of the relevant segment in between the partial lateral restraints.

Restraint against torsional rotation at supports in these beams may be provided by:

a) web or flange cleats; or

b) bearing stiffeners acting in conjunction with the bearing of the beam; or

c) lateral end frames or external supports providing lateral restraint to the compression flanges at the ends; or

d) their being built into walls.

Table 15 Effective Length for Simply Supported Beams, L_{LT}

(*Clause* 15.3.1)

Sl No.	Conditions of Restraint at Supports		Loading Condition	
	Torsional Restraint	Warping Restraint	Normal	Destabilizing
(1)	(2)	(3)	(4)	(5)
i)	Fully restrained	Both flanges fully restrained	0.70 L	0.85 L
ii)	Fully restrained	Only compression flange fully restrained	0.75 L	0.90 L
iii)	Fully restrained	Both flanges partially restrained	0.80 L	0.95 L
iv)	Fully restrained	Only compression flange partially restrained	0.85 L	1.00 L
v)	Fully restrained	No restraint in both flanges	1.00 L	1.20 L
vi)	Partially restrained by bottom flange support connection	No restraint in both flanges	1.0 L+ 2D	1.2 L+2D
vii)	Partially restrained by bottom flange bearing support	No restraint in both flanges	1.2 L+2D	1.4 L+2D

NOTES

1 Torsional restraint prevents rotation about the longitudinal axis.

2 Warping restraint prevents rotation of the flange in its plane.

3 D is the overall depth of the beam.

4 In case of continuous beams, L shall be taken as the distance between points of inflection, and the restraint conditions at the point of inflections should be considered.

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15.3.2 For beams, which are provided with members giving effective lateral restraint to the compression flange at intervals along the span, in addition to the end torsional restraint required in **15.3.1**, the effective length for lateral torsional buckling shall be taken as the distance, centre-to-centre of the restraint members in the critical segment under normal loading condition and 1.2 times this distance, where the load is acting on the beam above the shear centre and towards the shear centre, so as to have greater destabilizing effect during lateral torsional buckling deformation.

15.3.3 For cantilever beams of projecting length L, the effective length L_{LT} to be used in **15.2.2.1** shall be taken as in Table 16 for different support conditions.

15.3.4 Where a member is provided intermediate lateral supports to improve the lateral buckling strength, these restraints should have sufficient strength and stiffness to prevent lateral movement of the compression flange at that point, relative to the end supports. The intermediate lateral restraints should be either connected to an appropriate bracing system capable of transferring the restraint force to the effective lateral support at the ends of the member, or should be connected to an independent robust part of the structure capable of transferring the restraint force. Two or more parallel member requiring such lateral restraint shall not be simply connected together assuming mutual dependence for the lateral restraint.

The intermediate lateral restraints should be connected to the member as close to the compression flange as practicable. Such restraints should be closer to the shear centre of the compression flange than to the shear centre of the section. However, if torsional restraint preventing relative rotation between the two flanges is provided, the intermediate lateral restraint may be connected at any appropriate level. The point of contraflexure in beams with double curvature should be additionally effectively restrained against torsion in order to be considered it as a brace location.

For beams which are provided with members giving effective lateral restraint at intervals along the span, the effective lateral restraint shall be capable of resisting a force of 2.5 percent of the maximum force in the compression flange taken as divided equally between the points at which the restraint members are provided. Further, each restraint point should be capable of resisting 1 percent of the maximum force in the compression flange.

15.3.4.1 In a series of such beams, with solid webs, which are connected together by the same system of restraint members, the sum of the restraining forces required shall be taken as 2.5 percent of the maximum flange force in one beam only.

15.3.4.2 In the case of a series of latticed beams, girders or roof trusses which are connected together by the same system of restraint members, the sum of the restraining forces required shall be taken as 2.5 percent of the maximum force in the compression flange plus 1.25 percent of this force for every member of the series other than the first, up to a maximum total of 7.5 percent.

15.3.5 Purlins adequately restrained by sheeting need not be normally checked for the restraining forces required by rafters, roof trusses or portal frames that carry predominately roof loads provided there is bracing of adequate stiffness in the plane of rafters or roof sheeting which is capable of acting as a stressed skin diaphragm.

15.3.6 In case of beams with double curvature bending, adequate direct lateral support to the compression flange in the hogging moment region may be provided as given above for simply supported beam. The effect of support to the tension (top) flange in the hogging moment region on lateral restraint to the compression flange may be considered as per specialist literature.

15.3.7 In case of cantilever beams with tip loading, at the free end the tension flange also shall be effectively braced against lateral movement.

15.4 Shear

The design of plated webs with and without stiffeners are treated in this section. The design recommendations of Corrugated plate girders are presented in Annex G.

The factored design shear force, V, in a beam due to external actions shall satisfy:

$$V \leq V_d$$

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where

V_d = design strength

= V_n / γ_{m0}

where

γ_{m0} = partial safety factor against shear failure (*see* **12.4.1**)

The nominal shear strength of a cross section, V_n, may be governed by plastic shear resistance (*see* **15.4.1**) or strength of the web as governed by shear buckling (*see* **15.4.2**).

15.4.1 The nominal plastic shear resistance under pure shear is given by:

$$V_n = V_p$$

where

$$V_p = \frac{A_v \, f_{yw}}{\sqrt{3}}$$

A_v = shear area; and

f_{yw} = yield strength of the web.

15.4.1.1 The shear area may be calculated as given below:

a) I-and channel sections:

1) *Major Axis Bending*:

Hot-rolled : $h \, t_w$

Welded : $d \, t_w$

2) *Minor Axis Bending*:

Hot-rolled or welded : $2b \, t_f$

b) Rectangular hollow sections of uniform thickness:

Loaded parallel to depth (h) : $A \, h / (b+h)$

Loaded parallel to width (b) : $A \, b / (b+h)$

c) Circular hollow tubes of uniform thickness : $2A / \pi$

d) Plates and solid bars : A

where

A = cross section area;

b = overall breadth of tubular section, breadth of I-section flanges;

d = clear depth of the web between flanges;

h = overall depth of the section;

t_f = thickness of the flange; and

t_w = thickness of the web.

NOTE — Fastener holes need not be accounted for in plastic design shear strength calculation provided that:

$$A_{vn} \geq (f_y / f_u)(\gamma_{m1} / \gamma_{m0}) \, A_v / 0.9$$

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If A_{vn} does not satisfy the above condition, the effective shear area may be taken as that satisfying the above limit. Block shear failure criteria may be verified at the end connections. Provisions given under **16** may be referred to for design strength under combined high shear and bending.

15.4.2 *Resistance to Shear Buckling*

15.4.2.1 Resistance to shear buckling shall be verified as specified, when

$d/t_w > 67\varepsilon_w$ for a web without stiffeners, and

$> 67\varepsilon_w\sqrt{\frac{K_v}{5.35}}$ for a web with stiffeners

where

K_v = shear buckling coefficient (*see* **15.4.2.2**); and

$\varepsilon_w = \sqrt{250 / f_{yw}}$

15.4.2.2 *Shear buckling design methods*

The nominal shear strength, V_n, of webs with or without intermediate stiffeners as governed by buckling may be evaluated using one of the following methods:

a) *Simple post-critical method* — The simple post critical method, based on the shear buckling strength can be used for webs of I-section girders, with or without intermediate transverse stiffener, provided that the web has transverse stiffeners at the supports. The nominal shear strength is given by:

$$V_n = V_{cr}$$

where

V_{cr} = shear force corresponding to web buckling; and

= $A_v\tau_b$

where

τ_b = shear stress corresponding to web buckling, determined as follows:

1) when $\lambda_w \leq 0.8$, $\tau_b = f_{yw}/\sqrt{3}$
2) when $0.8 < \lambda_w < 1.2$, $\tau_b = [1-0.8(\lambda_w - 0.8)]\left(f_{yw}/\sqrt{3}\right)$
3) when $\lambda_w \geq 1.2$, $\tau_b = f_{yw}/\left(\sqrt{3}\,\lambda_w^{\,2}\right)$

where

λ_w = Non -dimensional web slenderness ratio for shear buckling stress, given by:

$$\lambda_w = \sqrt{f_{yw}/(\sqrt{3}\,\tau_{cr,e})}$$

$\tau_{cr,e}$ = the elastic critical shear stress of the web = $\dfrac{K_v\pi^2 E}{12\left(1-\mu^2\right)[d/t_w]^2}$

where

μ = Poisson's ratio; and

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K_v = 5.35 when transverse stiffeners are provided only at supports

= $4.0 + 5.35 /(c/d)^2$ for $c/d < 1.0$

= $5.35 + 4.0 /(c/d)^2$ for $c/d \geq 1.0$

where c, d are the spacing of transverse stiffeners and depth of the web, respectively.

b) *Tension field method* — The tension field method, based on the post-shear buckling strength, may be used for webs with intermediate transverse stiffeners, in addition to the transverse stiffeners at supports, provided the panels adjacent to the panel under tension field action, or the end posts provide anchorage for the tension fields and if $c/d \geq 1.0$, where c, d are the spacing of transverse stiffeners and depth of the web, respectively.

In the tension field method, the nominal shear resistance, V_n, is given by:

$$V_n = V_{tf}$$

where

$$V_{tf} = [A_v \tau_b + 0.9 w_{tf} t_w f_v \sin\phi] \leq V_p$$

where

τ_b = shear stress corresponding to buckling of web **15.4.2.2**(a);

f_v = yield strength of the tension field obtained from $\left[f_{yw}^2 - 3\tau_b^2 + \psi^2 \right]^{0.5} - \psi$;

ψ = $1.5\ \tau_b \sin 2\phi$;

ϕ = inclination of the tension field nearly $= \tan^{-1} \dfrac{\left(\dfrac{d}{c}\right)}{1.5}$;

w_{tf} = width of the tension field, given by:

= $d \cos\phi - (c - s_c - s_t) \sin\phi$;

f_{yw} = yield stress of the web;

d = depth of the web;

c = spacing of transverse stiffeners in the web; and

s_c, s_t = anchorage lengths of tension field along the compression and tension flange respectively, obtained from:

$$s = \frac{2}{\sin\phi}\left[\frac{M_{fr}}{f_{yw} t_w}\right]^{0.5} \leq c$$

M_{fr} = reduced plastic moment capacity of the respective flange plate (disregarding any edge stiffener) after accounting for the axial force, N_f in the flange, due to overall bending and any external axial force in the cross section, and is calculated as :

$$M_{fr} = 0.25 b_f t_f^2 f_{yf}\left[1 - \{N_f / (b_f t_f f_{yf} / \gamma_{m0})\}^2\right]$$

b_f, t_f = width and thickness of the relevant flange respectively; and

f_{yf} = yield stress of the flange.

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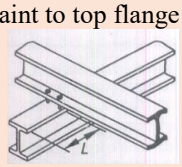
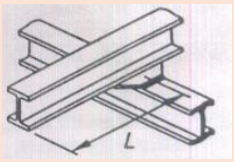
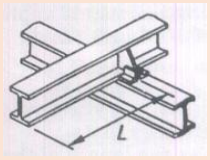
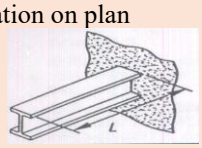
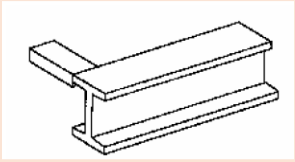
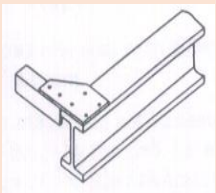
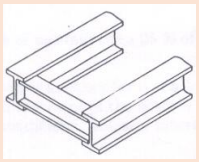
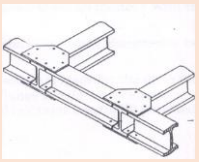
Table 16 Effective Length, L_{LT} for Cantilever of Length, L

(*Clause* 15.3.3)

Restraint Condition		Loading Conditions	
At Support	At tip	Normal	Destabilizing
(2)	(3)	(4)	(5)
a) Continuous, with lateral restraint to top flange 	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	3.0*L* 2.7*L* 2.4*L* 2.1*L*	7.5*L* 7.5*L* 4.5*L* 3.6*L*
b) Continuous, with partial torsional restraint 	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	2.0*L* 1.8*L* 1.6*L* 1.4*L*	5.0*L* 5.0*L* 3.0*L* 2.4*L*
c) Continuous, with lateral and torsional restraint 	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	1.0*L* 0.9*L* 0.8*L* 0.7*L*	2.5*L* 2.5*L* 1.5*L* 1.2*L*
d) Restrained laterally, torsionally and against rotation on plan 	1) Free 2) Lateral restraint to top flange 3) Torsional restraint 4) Lateral and torsional restraint	0.8*L* 0.7*L* 0.6*L* 0.5*L*	1.4*L* 1.4*L* 0.6*L* 0.5*L*

Tip Restraint Conditions

a) Free	b) Lateral restraint to top flange	c) Torsional restraint	d) Lateral and torsional restraint
			

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15.5 Stiffened Web Panels

15.5.1 *End Panels Design* (*see* Fig.12)

The girders in which the interior panel (panel A) is designed using tension field action, the end panels shall be designed using only simple post critical method, according to **15.4.2.2**(a).

Additionally, the end panel along with the stiffeners should be checked as a beam spanning between the flanges to resist a shear force, $R_{tf,}$ and a moment, $M_{tf,}$ due to tension field forces as given in **15.5.3**. Further, end stiffener should be capable of resisting the reaction plus a compressive force due to the moment, equal to M_{tf} [*see* Fig. (12)].

15.5.2 *End Panels Designed Using Tension Field Action* (*see* Fig. 13 and Fig. 14)

The end panel (Panel A) shall be designed according to **15.4.2.2**(b).

Additionally, it should be provided with an end post consisting of a single or double stiffener, (*see* Fig. 13 and Fig. 14), satisfying the following.

a) *Single stiffener* (*see* Fig. 13) — The top of the end post should be rigidly connected to the flange using full strength welds.

The end post should be capable of resisting the reaction plus a moment from the anchor forces equal to 2/3 M_{tf} due to tension field forces, where M_{tf} is obtained from **15.5.3**. The width and thickness of the end post are not to exceed the width and thickness of the flange.

b) *Double stiffener* (*see* Fig. 14) —The end post should be checked as a beam spanning between the flanges of the girder and capable of resisting a shear force R_{tf} and a moment, M_{tf} due to the tension field forces as given in **15.5.3**.

15.5.3 *Anchor Forces*

The resultant longitudinal shear, R_{tf}, and a moment M_{tf} from the anchor of tension field forces are evaluated as given below:

$$R_{tf} = \frac{H_q}{2} \text{ and } M_{tf} = \frac{H_q d}{10}$$

where

$$H_q = 1.25 V_p \left(1 - \frac{V_{cr}}{V_p}\right)^{1/2}$$

$$V_p = \frac{dtf_y}{\sqrt{3}}$$

d = web depth,

If the actual factored shear force, V in the panel designed using tension field approach is less than the shear strength, V_{tf} as given in **15.4.2.2**(b), then the values of H_q may be reduced by the ratio $\frac{V - V_{cr}}{V_{tf} - V_{cr}}$

where

V_{cr} = critical shear strength for the panel designed utilizing tension field action as given in **15.4.2.2**(a); and

V_{tf} = basic shear strength for the panel utilizing tension field action as given in **15.4.2.2**(b).

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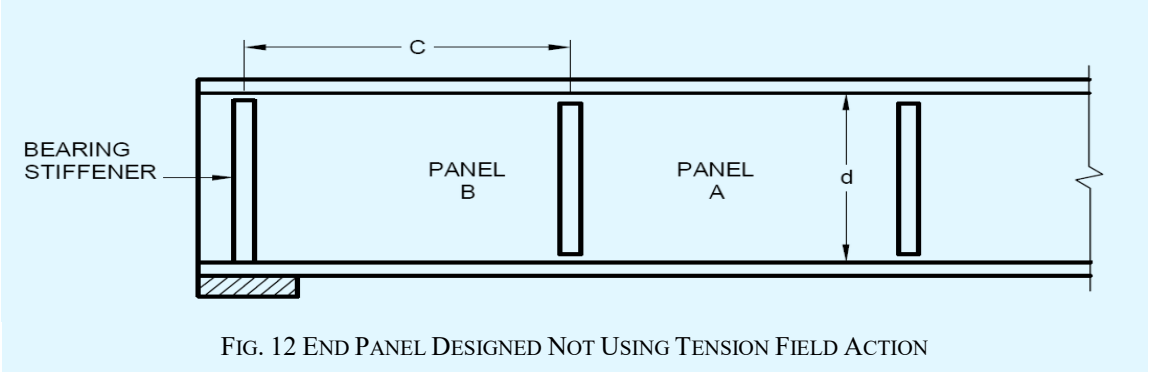

FIG. 12 END PANEL DESIGNED NOT USING TENSION FIELD ACTION

NOTES

1 Panel A is designed utilizing tension field action as given in 15.4.2.2(b).

2 Panel B is designed using simple post critical method as given in 15.4.2.2(a).

3 Bearing stiffener is designed for the compressive force due to bearing plus compressive force due to the moment M_{tf} as given in 15.5.3.

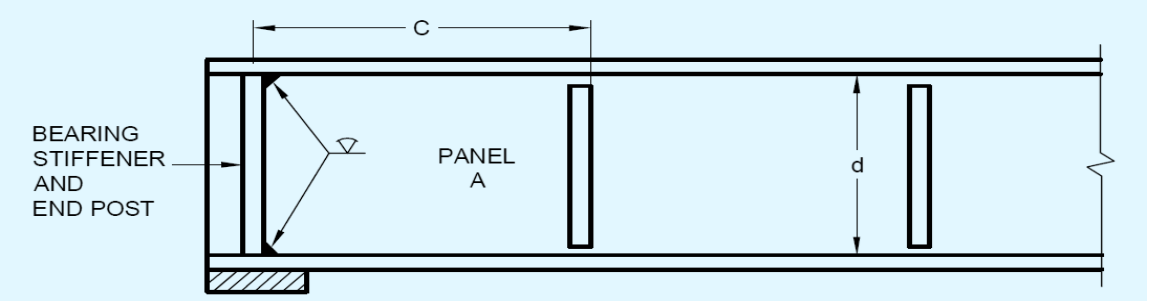

FIG. 13 END PANEL DESIGNED USING TENSION FIELD ACTION (SINGLE STIFFENER)

NOTES

1 Panel A is designed utilizing tension field action as given in 15.4.2.2(b).

2 Bearing stiffener and end post is designed for combination of compressive loads due to bearing and a moment equal to 2/3 M_{tf} as given in 15.5.3.

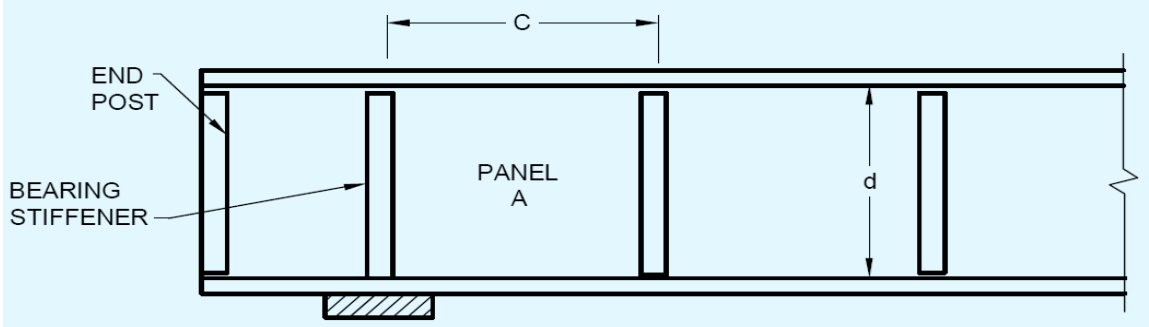

FIG. 14 END PANEL DESIGNED USING TENSION FIELD ACTION (DOUBLE STIFFENER)

NOTES

1 Panel A is designed utilizing tension field action as given in 15.4.2.2(b).

2 Bearing stiffener is designed for compressive force due to bearing as given in 15.4.2.2(a).

3 End post is designed for horizontal shear R_{tf} and moment M_{tf} as given in 15.5.3.

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15.5.4 *Panels with Openings* — Panels with opening of dimension greater than 10 percent of the minimum panel dimension should be designed without using tension field action as given in 15.4.2.2(b). The adjacent panels should be designed as an end panel as given in 15.5.1 or 15.5.2, as appropriate.

15.6 Design of Beams and Plate Girders with Solid Webs

15.6.1 *Minimum Web Thickness*

The thickness of the web in a section shall satisfy the requirements given in 15.6.1.1 and 15.6.1.2.

15.6.1.1 *Serviceability requirement*

a) when transverse stiffeners are not provided:

1) $\frac{d}{t_w} \leq 200\varepsilon_w$ (web connected to flanges along both longitudinal edges); and

2) $\frac{d}{t_w} \leq 90\varepsilon_w$ (web connected to flanges along one longitudinal edge only).

b) when only transverse stiffeners are provided (in webs connected to flanges along both longitudinal edges):

1) when $3d \geq c \geq d$

$$\frac{d}{t_w} \leq 200\varepsilon_w;$$

2) when $0.74\ d \leq c < d$

$$\frac{c}{t_w} \leq 200\varepsilon_w;$$

3) when $c < 0.74d$

$$\frac{d}{t_w} \leq 270\varepsilon_w;$$ and

4) when $c > 3d$, the web shall be considered as unstiffened.

c) when transverse stiffeners and longitudinal stiffeners at one level only are provided (distance from the compression flange equal to 2/5 of the distance from the compression flange angle, plate or tongue plate to the neutral axis) according to 15.7.13(a):

1) when $2.4d \geq c \geq d$

$$\frac{d}{t_w} \leq 250\varepsilon_w$$

2) when $0.74\ d \leq c \leq d$

$$\frac{c}{t_w} \leq 250\varepsilon_w$$

3) when $c < 0.74\ d$

$$\frac{d}{t_w} \leq 340\varepsilon_w$$

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d) when a second longitudinal stiffener (located at neutral axis is provided) $\frac{d}{t_w} \leq 400\varepsilon_w$

where

c = spacing of transverse stiffener (*see* Fig. 12 and Fig. 13);

d = depth of the web;

f_{yw} = yield stress of the web;

t_w = thickness of the web; and

ε_w = yield stress ratio of web = $\sqrt{250/f_{yw}}$.

15.6.1.2 *Compression flange buckling requirement*

In order to avoid buckling of the compression flange into the web, the web thickness shall satisfy the following:

a) When transverse stiffeners are not provided;

$$\frac{d}{t_w} \leq 345\varepsilon_f^2$$

b) When transverse stiffeners are provided, and

1) when $c \geq 1.5\ d$

$$\frac{d}{t_w} \leq 345\varepsilon_f^2$$

2) when $c < 1.5d$

$$\frac{d}{t_w} \leq 345\varepsilon_f$$

where

c = spacing of transverse stiffener (*see* Fig. 12 and Fig. 13);

d = depth of the web;

f_{yf} = yield stress of compression flange;

t_w = thickness of the web; and

ε_f = yield stress ratio of flange = $\sqrt{250/f_{yf}}$.

15.6.2 *Sectional Properties*

The effective sectional area of compression flanges shall be the gross area with deductions for excessive width of plates as specified for compression members [*see* **14**] and for open holes occurring in a plane perpendicular to the direction of stress at the section being considered (*see* **15.2.1.4**).

The effective sectional area of tension flanges shall be the gross sectional area with deductions for holes as specified in **15.2.1.4**.

The effective sectional area for parts in shear shall be taken as specified in **15.4.1.1**.

15.6.3 *Flanges*

15.6.3.1 In riveted or bolted construction, flange angles shall form as large a part of the area of the flange as practicable (preferably not less than one-third) and the number of flange plates shall be kept to a minimum.

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In exposed situations, where flange angles are used, at least one plate of the top flange shall extend over the full length of the girder, unless the top edge of the web is machined flush with the flange angles. Where two or more flange plates are used, tacking rivets shall be provided, if necessary to comply with the requirements of **17**.

Each flange plate shall extend beyond its theoretical cut-off point, and the extension shall contain sufficient rivets, bolts or welds to develop in the plate, the load calculated for the bending moment on the girder section (taken to include the curtailed plate) at the theoretical cut-off point.

The outstand of flange plates, that is the projection beyond the outer line of connections to flange angles, channel or joist flanges or in the case of welded constructions their projection beyond the face of the web or tongue plate, shall not exceed the values given in **10.8.2** (*see* Table 2).

In the case of box girders, the thickness of any plate, or the aggregate thickness of two or more plates, when these plates are tacked together to form the flange, shall satisfy the requirements given in **10.8.2** (*see* Table 2).

15.6.3.2 *Flange splices*

Flange splices should preferably, not be located at points of maximum stress. Where splice plates are used, their area shall be not less than 5 percent in excess of the area of the flange element spliced; their centre of gravity shall coincide, as nearly as possible, with that of the element spliced. There shall be enough bolts, rivets or welds on each side of the splice to develop the load in the element spliced plus 5 percent but in no case should the strength developed be less than 50 percent of the effective strength of the material spliced. In welded construction, flange plates shall be joined by complete penetration butt welds, wherever possible. These butt welds shall develop the full strength of the plates.

15.6.3.3 *Connection of flanges to web*

The flanges of plate girders shall be connected to the web by sufficient rivets, bolts or welds to transmit the maximum horizontal shear force resulting from the bending moment gradient in the girder, combined with any vertical loads which are directly applied to the flange. If the web is designed using tension field method as given in **15.4.2.2** (b), the weld should be able to transfer the tension field stress, f_{yw} acting on the web.

15.6.3.4 *Bolted/Riveted construction*

For girders in exposed situations and which do not have flange plates for their entire length, the top edge of the web plate shall be flush with or above the angles, and the bottom edge of the web plate shall be flush with or set back from the angles.

15.6.3.5 *Welded construction*

The gap between the web plates and flange plates shall be kept to a minimum and for fillet welds shall not exceed 1 mm at any point before welding.

15.6.4 *Webs*

15.6.4.1 *Effective sectional area of web of plate girder*

The effective cross sectional area shall be taken as the full depth of the web plate multiplied by the thickness.

NOTE — Where webs are varied in thickness in the depth of the section by the use of tongue plates or the like, or where the proportion of the web included in the flange area is 25 percent or more of the overall depth, the above approximation is not permissible and the maximum shear stress shall be computed on theory.

15.6.4.2 *Splices in webs*

Splices and cutouts for service ducts in the webs should preferably not be located at points of maximum shear force and heavy concentrated loads.

Splices in the webs of the plate girders and rolled sections shall be designed to resist the shears and moments at the spliced section (*see* Annex F).

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In riveted or bolted construction, splice plates shall be provided on each side of the web. In welded construction, web splices shall preferably be made with complete penetration butt welds.

15.6.4.3 Where additional plates are required to augment the strength of the web, they shall be placed on each side of the web and shall be equal in thickness. The proportion of shear force assumed to be resisted by these plates shall be limited by the amount of horizontal shear which they can transmit to the flanges through their fastenings, and such reinforcing plates and their fastenings shall be carried up to the points at which the flange without the additional plates is adequate.

15.7 Stiffener Design

15.7.1 *General*

15.7.1.1 When the web of a member acting alone (that is, without stiffeners) proves inadequate, stiffeners for meeting the following requirements should be provided:

a) *Intermediate transverse web stiffener* — To improve the buckling strength of a slender web due to shear (*see* **15.7.2**).

b) *Load carrying stiffener* — To prevent local buckling of the web due to concentrated loading (*see* **15.7.3** and **15.7.5**).

c) *Bearing stiffener* — To prevent local crushing of the web due to concentrated loading (*see* **15.7.4** and **15.7.6**).

d) *Torsion stiffener* — To provide torsional restraint to beams and girders at supports (*see* **15.7.9**).

e) *Diagonal stiffener* — To provide local reinforcement to a web under shear and bearing (*see* **15.7.7**).

f) *Tension stiffener* — To transmit tensile forces applied to a web through a flange (*see* **15.7.8**).

The same stiffeners may perform more than one function and their design should comply with the requirements of all the functions for which designed.

15.7.1.2 *Outstand of web stiffeners*

Unless the outer edge is continuously stiffened, the outstand from the face of the web should not exceed $20t_q\varepsilon$.

When the outstand of web stiffener is between $14t_q\varepsilon$ and $20t_q\varepsilon$, then the stiffener design should be on the basis of a core section with an outstand of only $14t_q\varepsilon$, where t_q is the thickness of the stiffener.

15.7.1.3 *Stiff bearing length*

The stiff bearing length of any element, b_1, is that length which cannot deform appreciably in bending. To determine b_1, the dispersion of load through a steel bearing element should be taken as 45° through solid material, such as bearing plates, flange plates, etc (*see* Fig.15).

15.7.1.4 *Eccentricity*

Where a load or reaction is applied eccentric to the centreline of the web or where the centroid of the stiffener does not lie on the centreline of the web, the resulting eccentricity of loading should be accounted for in the design of the stiffener.

15.7.1.5 *Buckling resistance of stiffeners*

The buckling resistance F_{qd} should be based on the design compressive stress f_{cd} (*see* **14.1.2**) of a strut (curve c), the radius of gyration being taken about the axis parallel to the web. The effective section is the full area or core area of the stiffener (*see* **15.7.1.2**) together with an effective length of web on each side of the centreline of the stiffeners, limited to 20 times the web thickness. The design strength used should be the minimum value obtained for buckling about the web or the stiffener.

The effective length for intermediate transverse stiffeners used in calculating the buckling resistance, F_{qd}, should be taken as 0.7 times the length, L of the stiffener.

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The effective length for load carrying web stiffeners used in calculating the buckling resistance, F_{xd}, assumes that the flange through which the load or reaction is applied is effectively restrained against lateral movement relative to the other flange, the effective length should be taken as:

a) $KL = 0.7L$, when flange is restrained against rotation in the plane of the stiffener (by other structural elements); and

b) $KL = L$, when flange is not so restrained.

where

L = length of the stiffener.

If the load or reaction is applied to the flange by a compression member, then unless effective lateral restraint is provided at that point, the stiffener should be designed as part of the compression member applying the load, and the connection between the column and beam flange shall be checked for the effects of the strut action.

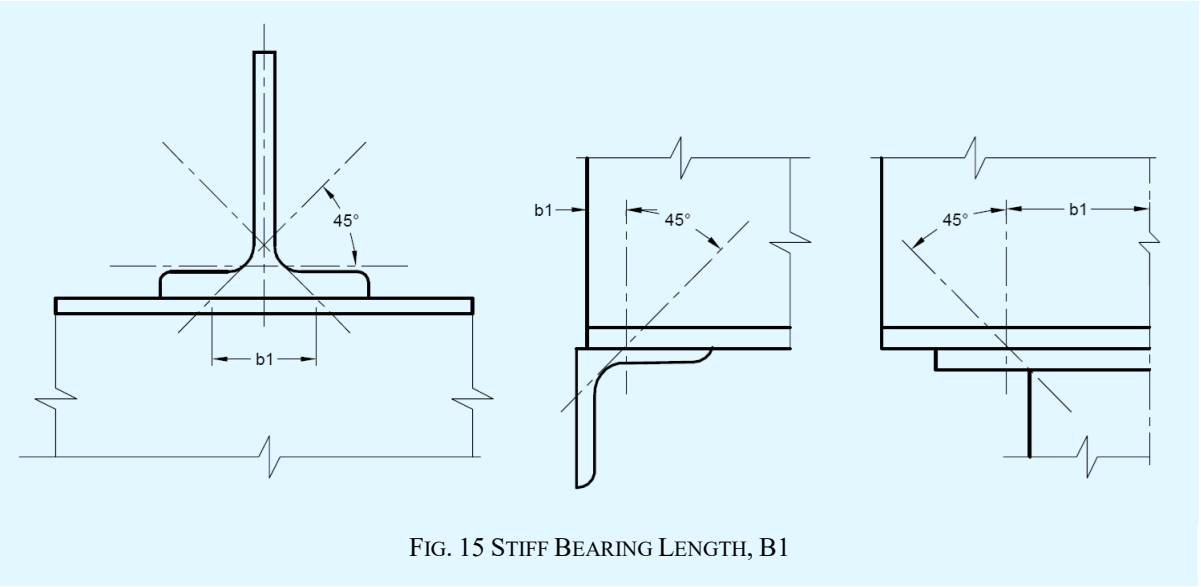

FIG. 15 STIFF BEARING LENGTH, B1

15.7.2 *Design of Intermediate Transverse Web Stiffeners*

15.7.2.1 *General*

Intermediate transverse stiffeners may be provided on one or both sides of the web.

15.7.2.2 *Spacing*

Spacing of intermediate stiffeners, where provided, shall comply with **15.6.1** depending on the thickness of the web.

15.7.2.3 *Outstand of stiffeners*

The outstand of the stiffeners should comply with **15.7.1.2**.

15.7.2.4 *Minimum stiffeners*

Transverse web stiffeners not subject to external loads or moments should have a second moment of area, I_s about the centreline of the web, if stiffeners are on both sides of the web and about the face of the web, if single stiffener

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on only one side of the web is used such that:

if $c/d \geq \sqrt{2}$, $I_s \geq 0.75\, d\, t_w^3$, and

if $c/d < \sqrt{2}$, $I_s \geq \frac{1.5 d^3 t_w^3}{c^2}$

where

c = actual stiffener spacing;

d = depth of the web; and

t_w = minimum required web thickness for spacing using tension field action, as given in 15.4.2.1.

15.7.2.5 *Buckling check on intermediate transverse web stiffeners*

Stiffeners not subjected to external loads or moments should be checked for a stiffener force:

$$F_q = V - V_{cr} / \gamma_{m0} \leq F_{qd}$$

where

F_{qd} = design resistance of the intermediate stiffeners;

V_{cr} = shear buckling resistance of the web panel designed without using tension field action as given in 15.4.2.2 (a); and

V = factored shear force adjacent to the stiffener.

Stiffeners subject to external loads and moments should meet the conditions for load carrying web stiffeners in 15.7.3. In addition they should satisfy the following interaction expression:

$$\frac{F_q - F_x}{F_{qd}} + \frac{F_x}{F_{xd}} + \frac{M_q}{M_{yq}} \leq 1$$

where

F_q = stiffener force given above;

F_{qd} = design resistance of an intermediate web stiffener corresponding to buckling about an axis parallel to the web (*see* 15.7.1.5);

F_x = external load or reaction at the stiffener;

F_{xd} = design resistance of a load carrying stiffener corresponding to buckling about axis parallel to the web (*see* 15.7.1.5);

M_q = moment on the stiffener due to eccentrically applied load and transverse load, if any; and

M_{yq} = yield moment capacity of the stiffener based on its elastic modulus about its centroidal axis parallel to the web.

If $F_q < F_x$, then $(F_q - F_x)$ should be taken as zero;

15.7.2.6 *Connection of intermediate stiffeners to web*

Intermediate transverse stiffeners not subject to external loading should be connected to the web so as to withstand a shear between each component of the stiffener and the web (in kN/mm) of not less than:

$$t_w^2 / (5 b_s)$$

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where

t_w = web thickness, in mm; and

b_s = outstand width of the stiffener, in mm.

For stiffeners subject to external loading, the shear between the web and the stiffener due to such loading has to be added to the above value.

Stiffeners not subject to external loads or moments may terminate clear of the tension flange and in such a situation the distance cut short from the line of the weld should not be more than $4t_w$.

15.7.3 *Load Carrying Stiffeners*

15.7.3.1 *Web check*

Load carrying web stiffeners should be provided where compressive forces applied through a flange by loads or reactions exceed the buckling strength, F_{cdw}, of the unstiffened web, calculated using the following:

The effective length of the web for evaluating the slenderness ratio is calculated as in **15.7.1.5**. The area of cross section is taken as $(b_1+n_1)\ t_w$.

where

b_1 = width of stiff bearing on the flange (*see* **15.7.1.3**); and

n_1 = dispersion of the load through the web at 45°, to the level of half the depth of the cross-section.

The buckling strength of this web about axis parallel to the web is calculated as given in **14.1.2.1**, using curve 'c'.

15.7.4 *Bearing Stiffeners*

Bearing stiffeners should be provided for webs where forces applied through a flange by loads or reactions exceeding the local capacity of the web at its connection to the flange, F_w, given by:

$$F_w = (b_1 + n_2) t_w f_{yw} / \gamma_{m0}$$

where

b_1 = Stiff bearing length (*see* **15.7.1.3**);

n_2 = Length obtained by dispersion through the flange to the web junction at a slope of 1 : 2.5 to the plane of the flange;

t_w = Thickness of the web; and

f_{yw} = Yield stress of the web.

15.7.5 *Design of Load Carrying Stiffeners*

15.7.5.1 *Buckling check*

The external load or reaction, F_x on a stiffener should not exceed the buckling resistance, F_{xd} of the stiffener as given in **15.7.1.5**.

Where the stiffener also acts as an intermediate stiffener it should be checked for the effect of combined loads in accordance with **15.7.2.5**.

15.7.5.2 *Bearing check*

Load carrying web stiffeners should also be of sufficient size that the bearing strength of the stiffener, $F_{psd,}$ given below is not less than the load transferred, F_x

$$F_{psd} = A_q f_{yq} /(0.8\gamma_{m0}) \geq F_x$$

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where

F_x = external load or reaction,

A_q = area of the stiffener in contact with the flange, and

f_{yq} = yield stress of the stiffener.

15.7.6 *Design of Bearing Stiffeners*

Bearing stiffeners should be designed for the applied load or reaction less the local capacity of the web as given in **15.7.4**. Where the web and the stiffener material are of different strengths the lesser value should be assumed to calculate the capacity of the web and the stiffener. Bearing stiffeners should project nearly as much as the overhang of the flange through which load is transferred.

15.7.7 *Design of Diagonal Stiffeners*

Diagonal stiffeners should be designed to carry the portion of the applied shear and bearing that exceeds the capacity of the web.

Where the web and the stiffener are of different strengths, the value for design should be taken as given in **15.7.6**.

15.7.8 *Design of Tension Stiffeners*

Tension stiffeners should be designed to carry the portion of the applied load or reaction less the capacity of the web as given in **15.7.4** for bearing stiffeners.

Where the web and the stiffener are of different strengths, the value for design should be taken as given in **15.7.6**.

15.7.9 *Torsional Stiffeners*

Where bearing stiffeners are required to provide torsional restraint at the supports of the beam, they should meet the following criteria:

a) Conditions of **15.7.4**; and

b) Second moment of area of the stiffener section about the centreline of the web, I_s should be such that:

$$I_s \geq 0.34\,\alpha_s\,D^3\,T_{cf}$$

where

α_s = 0.006 for $L_{LT} / r_y \leq 50$;

= $0.3/(L_{LT}/r_y)$ for $50 < L_{LT}/r_y \leq 100$;

= $30/(L_{LT}/r_y)^2$ for $L_{LT}/r_y > 100$;

D = Overall depth of beam at support;

T_{cf} = Maximum thickness of compression flange in the span under consideration;

L_{LT} = Laterally unsupported effective length of the compression flange of the beam; and

r_y = Radius of gyration of the beam about the minor axis.

15.7.10 *Connection to Web of Load Carrying and Bearing Stiffeners* — Stiffeners, which resist loads or reactions applied through a flange, should be connected to the web by sufficient welds or fasteners to transmit a design force equal to the lesser of:

a) tension capacity of the stiffener; and

b) sum of the forces applied at the two ends of the stiffener when they act in the same direction or the larger of the forces when they act in opposite directions.

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Stiffeners, which do not extend right across the web, should be of such length that the shear stress in the web due to the design force transmitted by the stiffener does not exceed the shear strength of the web. In addition, the capacity of the web beyond the end of the stiffener should be sufficient to resist the applied force.

15.7.11 *Connection to Flanges*

15.7.11.1 *In tension*

Stiffeners required to resist tension should be connected to the flange transmitting the load by continuous welds or non-slip fasteners.

15.7.11.2 *In compression*

Stiffeners required to resist compression should either be fitted against the loaded flange or connected by continuous welds or non-slip fasteners.

The stiffener should be fitted against or connected to both flanges when:

a) a load is applied directly over a support; or

b) it forms the end stiffener of a stiffened web; or

c) it acts as a torsion stiffener.

15.7.12 *Hollow Sections*

Where concentrated loads are applied to hollow sections consideration should be given to local stresses and deformations and the section reinforced as necessary.

15.7.13 *Horizontal Stiffeners*

Where horizontal stiffeners are used in addition to vertical stiffeners, they shall be as follows:

a) One horizontal stiffener shall be placed on the web at a distance from the compression flange equal to 2/5 of web depth, d when the thickness of the web is less than the limits specified in 15.6.1. The stiffener shall be designed so that I_s is not less than $4ct_w^3$ where I_s and t_w are as defined in 15.7.2.4 and c is the actual distance between the vertical stiffeners;

b) A second horizontal stiffener (single or double) shall be placed at the neutral axis of the girder when the thickness of the web is less than the limit specified in 15.6.1. This stiffener shall be designed so that I_s is not less than $d_2 t_w^3$ where I_s and t_w are as defined in 15.7.2.4 and d_2 is twice the clear distance from the compression flange angles, plates or tongue plates to the neutral axis;

c) Horizontal web stiffeners shall extend between vertical stiffeners, but need not be continuous over them; and

d) Horizontal stiffeners may be in pairs arranged on each side of the web, or single located on one side of the web.

15.8 Box Girders

The design and detailing of box girders shall be such as to give full advantage of its higher load carrying capacity. Box girder shall be designed in accordance with specialist literature. The diaphragms and horizontal stiffeners should conform to 15.7.12 and 15.7.13.

15.8.1 All diaphragms shall be connected so as to transfer the resultant shears to the web and flanges.

15.8.2 Where the concentrated or moving load does not act directly on top of the web, the local effect shall be considered in the design of flanges and the diaphragms.

15.9 Purlins and Sheeting Rails (Girts)

All purlins shall be designed in accordance with the requirements for beams as specified in 15.2.1 and 15.2.2. The

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limitations of bending stress based on lateral instability of the compression flange shall be considered if the purlin does not receive adequate support from bracings and sheathings in the plane of compression flange. In any case the maximum bending moment shall not exceed the values specified in 15.2.1. The calculated deflections should not exceed those permitted for the type of roof cladding used, as specified in 12.6.1. In calculating the bending moment, advantage may be taken of the continuity of the purlin over supports. The bending about the two axes shall be determined separately and checked according to the biaxial bending requirements specified under 16.

15.10 Bending in a Non-Principal Plane

15.10.1 When the flexural deflection of a member is constrained to a non-principal plane by lateral restraints preventing lateral deflection, then the force exerted by the restraints shall be determined, and the principal axes bending moments acting on the member shall be calculated from these forces and applied forces, by a rational analysis. The combined effect of bending about the principal axes shall satisfy the requirements given under 16.

15.10.2 When the deflections of a member loaded in a non-principal plane are unconstrained; the principal axes bending moments shall be calculated by a rational analysis. The combined effect of bending about the principal axes shall satisfy the requirements given under 16.

16 MEMBER SUBJECTED TO COMBINED FORCES

16.1 General

This Section governs the design of members subjected to combined forces, such as shear force and bending, axial force and bending, or shear force, axial force and bending.

16.2 Combined Shear and Bending

16.2.1 No reduction in moment capacity of the section is necessary as long as the cross section is not subjected to high shear force (factored value of applied shear force is less than or equal to 60 percent of the shear strength of the section as given in 15.4). The moment capacity may be taken as, M_d (*see* 15.2) without any reduction.

16.2.2 When the factored value of the applied shear force is high (exceeds the limit specified in 16.2.1), the factored moment of the section should be less than the moment capacity of the section under higher shear force, M_{dv}, calculated as given below:

a) Plastic or compact section:

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2\, Z_e f_y / \gamma_{m0}$$

where

$\beta = (2V/V_d - 1)^2$;

M_d = plastic design moment of the whole section disregarding high shear force effect (*see* 15.2.1.2); considering web buckling effects (*see* 15.2.1.1);

V = factored applied shear force as governed by web yielding or web buckling;

V_d = design shear strength as governed by web yielding or web buckling (*see* 15.4.1 or 15.4.2);

M_{fd} = plastic design strength of the area of the cross section excluding the shear area, considering partial safety factor γ_{m0}; and

Z_e = elastic section modulus of the whole section.

b) Semi-compact section:

$$M_{dv} = Z_e f_y / \gamma_{m0}$$

16.3 Combined Axial Force and Bending Moment

Under combined axial force and bending moment, section strength as governed by material failure and member strength as governed by buckling failure shall be checked in accordance with 16.3.1 and 16.3.2 respectively.

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16.3.1 *Section Strength*

16.3.1.1 *Plastic and compact sections*

In the design of members subjected to combined axial force (tension or compression) and bending moment, the following should be satisfied:

$$\left(\frac{M_y}{M_{ndy}}\right)^{\alpha_1} + \left(\frac{M_z}{M_{ndz}}\right)^{\alpha_2} \leq 1.0$$

Conservatively, the following equation may also be used under combined axial force and bending moment:

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$$

where

α_1, α_2 = constants as given in Table 17;

A_g = gross area of the cross section;

M_{dy}, M_{dz} = design strength under corresponding moment acting alone (*see* **15.2**);

M_{ndy}, M_{ndz} = design reduced flexural strength under combined axial force and the respective uniaxial moment acting alone (*see* **16.3.1.2**);

M_y, M_z = factored applied moments about the minor and major axis of the cross section, respectively;

N = factored applied axial force (Tension, T or Compression, P);

N_d = design strength in tension, T_d as obtained from **12** or in compression due to yielding given by $N_d = A_g f_y/\gamma_{m0}$; and

γ_{m0} = partial factor of safety in yielding.

16.3.1.2 For plastic and compact sections without bolts holes, the following approximations may be used for evaluating M_{ndy} and M_{ndz}:

a) Plates:

$$M_{nd} = M_d (1 - n^2)$$

b) Welded I or H-sections:

$$M_{ndy} = M_{dy}\left[1 - \left(\frac{n-a}{1-a}\right)^2\right] \leq M_{dy} \qquad (n \geq a)$$

$$M_{ndz} = M_{dz} (1 - n) / (1 - 0.5a) \leq M_{dz}$$

where

$n = N/N_d$ and $a = (A - 2\,b\,t_f)/A \leq 0.5$

c) For standard I or H sections:

$M_{ndz} = 1.11\, M_{dz} (1 - n) \leq M_{dz}$

for $n \leq 0.2$, $M_{ndy} = M_{dy}$

for $n > 0.2$, $M_{ndy} = 1.56\, M_{dy} (1 - n) (n + 0.6)$

d) For rectangular hollow sections and welded box sections:

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When the section is symmetric about both axes and without bolt holes:

$$M_{ndy} = M_{dy}\,(1 - n)\,/(1 - 0.5a_f) \leq M_{dy}$$

$$M_{ndz} = M_{dz}\,(1 - n)\,/\,(1 - 0.5a_w) \leq M_{dz}$$

where

$a_f = (A - 2\,h\,t_w)\,/A \leq 0.5$

$a_w = (A - 2\,b\,t_f)\,/A \leq 0.5$

e) Circular hollow tubes without bolt holes:

$$M_{nd} = 1.04\,M_d\,(1 - n^{1.7}) \leq M_d$$

Table 17 Constants α_1 and α_2

(*Clause* 16.3.1.1)

Sl No.	Section	α_1	α_2
(1)	(2)	(3)	(4)
i)	I and Channel	$5n \geq 1$	2
ii)	Circular tubes	2	2
iii)	Rectangular tubes	$1.66/(1 - 1.13n^2) \leq 6$	$1.66/(1 - 1.13n^2) \leq 6$
iv)	Solid rectangles	$1.73 + 1.8n^3$	$1.73 + 1.8\,n^3$

NOTE — $n = N/N_d$

16.3.1.3 *Semi-compact section*

In the absence of high shear force (*see* **16.2.1**), semi-compact section design is satisfactory under combined axial force and bending, if the maximum longitudinal stress under combined axial force and bending, f_x satisfies the following criteria:

$$f_x \leq f_y/\gamma_{m0}$$

For cross section without holes, the above criterion reduces to,

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$$

where

N_d, M_{dy}, M_{dz} are as defined in **16.3.1.1**.

16.3.2 *Overall Member Strength*

Members subjected to combined axial force and bending moment shall be checked for overall buckling failure as given in this section.

16.3.2.1 *Bending and axial tension*

The reduced effective moment, M_{eff}, under tension and bending calculated as given below, should not exceed the bending strength due to lateral torsional buckling, M_d (*see* **15.2.2**),

$$M_{eff} = \left[M - \psi\,T\,Z_{ec}\,/\,A\right] \leq M_d$$

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where

A = area of cross section;

M, T = factored applied moment and tension, respectively;

Z_{ec} = elastic section modulus of the section with respect to extreme compression fibre;

ψ = 0.8, if T and M can vary independently, or otherwise; and

= 1.0.

16.3.2.2 *Bending and axial compression*

Members subjected to combined axial compression and biaxial bending shall satisfy the following interaction relationships:

$$\frac{P}{P_{dy}} + K_{yy}\frac{M_y}{M_{dy}} + K_{yz}\frac{M_z}{M_{dz}} \leq 1.0$$

$$\frac{P}{P_{dz}} + K_{zy}\frac{M_y}{M_{dy}} + K_{zz}\frac{M_z}{M_{dz}} \leq 1.0$$

where

K_{yy}, k_{yz}, K_{zy}, K_{zz} = interaction factors as given in Table 18;

M_{dy}, M_{dz} = design bending strength about y (minor) or z (major) axis, considering laterally unsupported length of the cross section [*see* 15];

M_y, M_z = maximum factored applied bending moments about y and z-axis of the member, respectively;

P = applied axial compression under factored load; and

P_{dy}, P_{dz} = design strength under axial compression as governed by buckling about minor (y) and major (z) axis respectively.

Table 18 Interaction Factors (Section 10.3.2.2)

(*Clause* 16.3.2.2)

Sl No.	Type of Section	Slenderness Ratio	Plastic Sections (Class 1, Class 2)	Elastic Sections (Class 3, Class 4)
(1)	(2)	(3)	(4)	(5)
i)	All	$\lambda_z < 1.0$	$K_{zz} = C_{mz}[1+(\lambda_z-0.2)n_z]$	$K_{zz} = C_{mz}[1+0.6\lambda_z n_z]$
		$\lambda_z \geq 1.0$	$K_{zz} = C_{mz}[1+0.8 n_z]$	$K_{zz} = C_{mz}[1+0.6 n_z]$
ii)	I sections	$\lambda_y < 1.0$	$K_{yy} = C_{my}[1+(2\lambda_y-0.6)n_y]$	$K_{yy} = C_{my}[1+0.6\lambda_y n_y]$
		$\lambda_y \geq 1.0$	$K_{yy} = C_{my}[1+1.4 n_y]$	$K_{yy} = C_{my}[1+0.6 n_y]$
iii)	Hollow sections	$\lambda_y < 1.0$	$K_{yy} = C_{my}[1+(\lambda_y-0.2)n_y]$	$K_{yy} = C_{my}[1+0.6\lambda_y n_y]$
		$\lambda_y \geq 1.0$	$K_{yy} = C_{my}[1+0.8 n_y]$	$K_{yy} = C_{my}[1+0.6 n_y]$
iv)	All	All	$K_{zy} = 0.66 K_{yy}$	$K_{zy} = K_{yy}$

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Table 18 (*Concluded*)

Sl No.	Type of Section	Slenderness Ratio	Plastic Sections (Class 1, Class 2)	Elastic Sections (Class 3, Class 4)
(1)	(2)	(3)	(4)	(5)
v)	Not susceptible to lateral torsional buckling	All	K_{yz} =0.6 K_{zz}	K_{yz} =0.8 K_{zz}
vi)	Susceptible to lateral torsional buckling	$\lambda_y < 0.4$	$K_{yz} = 1 - \frac{0.1\ \lambda_y n_y}{C_{mLT} - 0.25} \leq 0.6\ \lambda_y$	$K_{yz} = 1 - \frac{0.05\ \lambda_y n_y}{C_{mLT} - 0.25}$
		$0.4 \leq \lambda_y < 1.0$	$K_{yz} = 1 - \frac{0.1\ \lambda_y n_y}{C_{mLT} - 0.25}$	$K_{yz} = 1 - \frac{0.05\ \lambda_y n_y}{C_{mLT} - 0.25}$
		$\lambda_y \geq 1.0$	$K_{yz} = 1 - \frac{0.1 n_y}{C_{mLT} - 0.25}$	$K_{yz} = 1 - \frac{0.05\ n_y}{C_{mLT} - 0.25}$

Where
n_y, n_z = ratio of actual applied axial force to the design axial strength for buckling about the y and z-axis, respectively C_{my}, C_{mz}, C_{mLT} = Equivalent uniform moment factor for lateral torsional buckling as per Table 19.

Table 19 Equivalent Uniform Moment Factor

(*Table* 18)

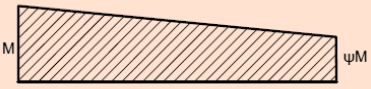
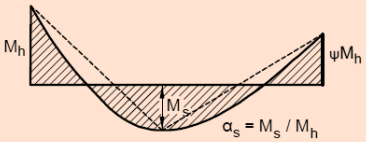
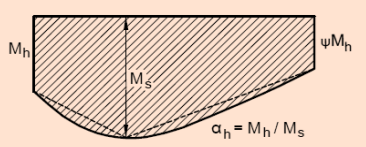
Bending Moment Diagram	Range		C_{my}, C_{mz}, C_{mLT}	
			Uniform Loading	Concentrated Load
(1)	(2)		(3)	(4)
	$-1 \leq \psi \leq 1$		$0.6 + 0.4\ \psi \geq 0.4$	
	$0 \leq \alpha_s \leq 1$	$-1 \leq \psi \leq 1$	$0.2 + 0.8\ \alpha_s \geq 0.4$	$0.2 + 0.8\ \alpha_s \geq 0.4$
	$-1 \leq \alpha_s \leq 0$	$0 \leq \psi \leq 1$	$0.1 - 0.8\ \alpha_s \geq 0.4$	$-0.8\ \alpha_s \geq 0.4$
		$-1 \leq \psi \leq 0$	$0.1(1-\psi) - 0.8\ \alpha_s \geq 0.4$	$-0.2\psi - 0.8\ \alpha_s \geq 0.4$
	$0 \leq \alpha_h \leq 1$	$-1 \leq \psi \leq 1$	$0.95 - 0.05\ \alpha_h$	$0.90 + 0.10\ \alpha_h$
	$-1 \leq \alpha_h \leq 0$	$0 \leq \psi \leq 1$	$0.95 + 0.05\ \alpha_h$	$0.90 + 0.10\ \alpha_h$
		$-1 \leq \psi \leq 0$	$0.95 + 0.05\ \alpha_h\ (1+2\ \psi)$	$0.90 + 0.1\alpha_h\ (1+2\ \psi)$
For members with sway buckling mode, the equivalent uniform moment factor C_{my} = C_{mz} = 0.9.				

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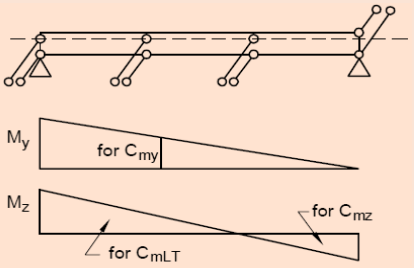
Table 19 (*Concluded*)

Bending Moment Diagram	**Range**	**C_{my}, C_{mz}, C_{mLT}**	
		Uniform Loading	Concentrated Load
(1)	(2)	(3)	(4)
C_{my}, C_{mz}, C_{mLT} shall be obtained according to the bending moment diagram between the relevant braced points			
Moment factor direction C_{my} C_{mz} C_{mLT}	Bending axis z-z y-y z-z	Points braced in y-y z-z z-z	

17 CONNECTIONS

17.1 General

17.1.1 This Section deals with the design and detailing requirements for joints between members. Connection elements consist of components such as cleats, gusset plates, brackets, connecting plates and connectors such as rivets, bolts, pins, and welds. The connections in a structure shall be designed so as to be consistent with the assumptions made in the analysis of the structure and comply with the requirements specified in this section. Connections shall be capable of transmitting the calculated design actions.

17.1.2 Where members are connected to the surface of a web or the flange of a section, the ability of the web or the flange to transfer the applied forces locally should be checked and where necessary, local stiffening provided.

17.1.3 Ease of fabrication and erection should be considered in the design of connections. Attention should be paid to clearances necessary for field erection, tolerances, tightening of fasteners, welding procedures, subsequent inspection, surface treatment and maintenance.

17.1.4 The ductility of steel assists the distribution of forces generated within a joint. Effects of residual stresses and stresses due to tightening of fasteners and normal tolerances of fit-up need not therefore be considered in connection design, provided ductile behaviour is ensured.

17.1.5 In general, use of different forms of fasteners to transfer the same force shall be avoided. However, when different forms of fasteners are used to carry a shear load or when welding and fasteners are combined, then one form of fastener shall be normally designed to carry the total load. Nevertheless, fully tensioned friction grip bolts may be designed to share the load with welding, provided the bolts are fully tightened to develop necessary pretension after welding.

17.1.6 The partial safety factor in the evaluation of design strength of connections shall be taken as given in Table 5.

17.2 Location Details of Fasteners

17.2.1 *Clearances for Holes for Fasteners*

Bolts may be located in standard size, over size, short slotted or long slotted hole, as follows:

a) *Standard clearance hole* — Except where fitted bolts, bolts in low-clearance or oversize holes are specified, the diameter of standard clearance holes for fasteners shall be as given in Table 20;

b) *Oversize hole* — Holes of size larger than the standard clearance holes, as given in Table 20 may be used in slip resistant connections and hold down bolted connections, only where specified, provided the oversize;

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c) Holes in the outer ply is covered by a cover plate of sufficiently large size and thickness and having a hole not larger than the standard clearance hole (and hardened washer in slip resistant connections); and

d) *Short and long slots* — Slotted holes of size larger than the standard clearance hole, as given in Table 20 may be used in slip resistant connections and hold down bolted connections, only where specified, provided the oversize holes in the outer ply is covered by a cover plate of sufficiently large size and thickness and having a hole of size not larger than the standard clearance hole (and hardened washer in slip resistant connection).

Table 20 Clearances for Fastener Holes

(*Clauses* 17.2.1 *and* 24.2.2.1)

Sl No.	Nominal Size of Fastener, ***d*** mm	Size of the Hole = Nominal Diameter of the Fastener + Clearances mm			
		Standard clearance in diameter and width of slot	Over size clearance in diameter	Clearance in the length of the slot	
				Short Slot	Long Slot
(1)	(2)	(3)	(4)	(5)	(6)
i)	12 to 14	1.0	3.0	4.0	2.5 *d*
ii)	16 to 22	2.0	4.0	6.0	2.5 *d*
iii)	24	2.0	6.0	8.0	2.5 *d*
iv)	Larger than 24	3.0	8.0	10.0	2.5 *d*

17.2.2 *Minimum Spacing*

The distance between centres of fasteners shall not be less than 2.5 times the nominal diameter of the fastener.

17.2.3 *Maximum Spacing*

17.2.3.1 The distance between the centres of any two adjacent fasteners shall not exceed 32*t* or 300 mm, whichever is less, where *t* is the thickness of the thinner plate.

17.2.3.2 The distance between the centres of two adjacent fasteners (pitch) in a line lying in the direction of stress, shall not exceed 16*t* or 200 mm, whichever is less, in tension members and 12*t* or 200 mm, whichever is less, in compression members; where *t* is the thickness of the thinner plate. In the case of compression members wherein forces are transferred through butting faces, this distance shall not exceed 4.5 times the diameter of the fasteners for a distance equal to 1.5 times the width of the member from the butting faces.

17.2.3.3 The distance between the centres of any two consecutive fasteners in a line adjacent and parallel to an edge of an outside plate shall not exceed 100 mm plus 4*t* or 200 mm, whichever is less, in compression and tension members; where *t* is the thickness of the thinner outside plate.

17.2.3.4 When fasteners are staggered at equal intervals and the gauge does not exceed 75 mm, the spacing specified in **17.2.3.2** and **17.2.3.3** between centres of fasteners may be increased by 50 percent, subject to the maximum spacing specified in **17.2.3.1**.

17.2.4 *Edge and End Distances*

17.2.4.1 The edge distance is the distance at right angles to the direction of stress from the centre of a hole to the adjacent edge. The end distance is the distance in the direction of stress from the centre of a hole to the end of the element.

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In slotted holes, the edge and end distances should be measured from the edge or end of the material to the centre of its end radius or the centre line of the slot, whichever is smaller. In oversize holes, the edge and end distances should be taken as the distance from the relevant edge/end plus half the diameter of the standard clearance hole corresponding to the fastener, less the nominal diameter of the oversize hole.

17.2.4.2 The minimum edge and end distances from the centre of any hole to the nearest edge of a plate shall not be less than 1.7 times the hole diameter in case of sheared or hand-flame cut edges; and 1.5 times the hole diameter in case of rolled, machine-flame cut, sawn and planed edges.

17.2.4.3 The maximum edge distance to the nearest line of fasteners from an edge of any un-stiffened part should not exceed 12 $t\varepsilon$, where $\varepsilon = (250/f_y)^{1/2}$ and t is the thickness of the thinner outer plate. This would not apply to fasteners interconnecting the components of back to back tension members. Where the members are exposed to corrosive influences, the maximum edge distance shall not exceed 40 mm plus 4t, where t is the thickness of thinner connected plate.

17.2.5 *Tacking Fasteners*

17.2.5.1 In case of members covered under **17.2.4.3**, when the maximum distance between centres of two adjacent fasteners as specified in **17.2.4.3** is exceeded, tacking fasteners not subjected to calculated stress shall be used.

17.2.5.2 Tacking fasteners shall have spacing in a line not exceeding 32 times the thickness of the thinner outside plate or 300 mm, whichever is less. Where the plates are exposed to the weather, the spacing in line shall not exceed 16 times the thickness of the thinner outside plate or 200 mm, whichever is less. In both cases, the distance between the lines of fasteners shall not be greater than the respective pitches.

17.2.5.3 All the requirements specified in **17.2.5.2** shall generally apply to compression members, subject to the stipulations given under **14** affecting the design and construction of compression members.

17.2.5.4 In tension members [*see* **13**] composed of two flats, angles, channels or tees in contact back to back or separated back to back by a distance not exceeding the aggregate thickness of the connected parts, tacking fasteners with solid distance pieces shall be provided at a spacing in line not exceeding 1 000 mm.

17.2.5.5 For compression members covered under **14**, tacking fasteners in a line shall be spaced at a distance not exceeding 600 mm.

17.2.6 *Countersunk Heads*

For countersunk heads, one-half of the depth of the countersinking shall be neglected in calculating the length of the fastener in bearing in accordance with **17.3.3**. For fasteners in tension having countersunk heads, the tensile strength shall be reduced by 33.3 percent. No reduction is required to be made in shear strength calculations.

17.3 Bearing Type Bolts

17.3.1 *Effective Areas of Bolts*

17.3.1.1 Since threads can occur in the shear plane, the area A_e for resisting shear should normally be taken as the net tensile stress area, A_n of the bolts. For bolts where the net tensile stress area is not defined, A_n shall be taken as the area at the root of the threads.

17.3.1.2 Where it can be shown that the threads do not occur in the shear plane, A_e may be taken as the cross section area, A_s at the shank.

17.3.1.3 In the calculation of thread length, allowance should be made for tolerance and thread run off.

17.3.2 A bolt subjected to a factored shear force (V_{sb}) shall satisfy the condition

$$V_{sb} \leq V_{db}$$

where V_{db} is the design strength of the bolt, taken as the smaller of the value as governed by shear, V_{dsb} (*see* **17.3.3**) and bearing, V_{dpb} (*see* **17.3.4**).

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17.3.3 *Shear Capacity of Bolt*

The design strength of the bolt, V_{dsb} as governed by shear strength is given by:

$$V_{dsb} = V_{nsb} / \gamma_{mb}$$

where

V_{nsb} = nominal shear capacity of a bolt, calculated as follows.

$$V_{nsb} = \frac{f_u}{\sqrt{3}}\left(n_n A_{nb} + n_s A_{sb}\right)$$

where

A_{nb} = net shear area of the bolt at threads, may be taken as the area corresponding to root diameter at the thread;

A_{sb} = nominal plain shank area of the bolt;

f_u = ultimate tensile strength of a bolt;

n_n = number of shear planes with threads intercepting the shear plane; and

n_s = number of shear planes without threads intercepting the shear plane.

17.3.3.1 *Long joints*

When the length of the joint, l_j of a splice or end connection in a compression or tension element containing more than two bolts (that is, the distance between the first and last rows of bolts in the joint, measured in the direction of the load transfer) exceeds $15d$ in the direction of load, the nominal shear capacity (*see* **17.3.2**), V_{db} shall be reduced by the factor β_{lj}, given by:

$$\beta_{lj} = 1.075 - l_j / (200\ d) \text{ but } 0.75 \leq \beta_{lj} \leq 1.0 = 1.075 - 0.005(l_j / d)$$

where

d = Nominal diameter of the fastener.

NOTE — This provision does not apply when the distribution of shear over the length of joint is uniform, as in the connection of web of a section to the flanges.

17.3.3.2 *Large grip lengths*

When the grip length, l_g (equal to the total thickness of the connected plates) exceeds 5 times the diameter, d of the bolts, the design shear capacity shall be reduced by a factor β_{lg}, given by:

$$\beta_{lg} = 8d /(3d + l_g) = 8 /(3 + l_g/d)$$

β_{lg} shall not be more than β_{lj} given in **17.3.3.1**. The grip length, l_g shall in no case be greater than $8d$.

17.3.3.3 *Packing plates*

The design shear capacity of bolts carrying shear through a packing plate in excess of 6 mm shall be decreased by a factor, β_{pk} given by:

$$\beta_{pk} = (1 - 0.0125 t_{pk})$$

where

t_{pk} = thickness of the thicker packing, in mm.

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17.3.4 *Bearing Capacity of the Bolt*

The design bearing strength of a bolt on any plate, V_{dpb} as governed by bearing is given by:

$$V_{dpb} = V_{npb} / \gamma_{mb}$$

where

V_{npb} = Nominal bearing strength of a bolt; and

= $2.5\ k_b\ d\ t\ f_u$.

where

d_0 = diameter of the hole;

d = nominal diameter of the bolt;

f_{ub}, f_u = Ultimate tensile stress of the bolt and the ultimate tensile stress of the plate, respectively;

e, p = end and pitch distances of the fastener along bearing direction;

k_b = is smaller of $\frac{e}{3d_0}$, $\frac{p}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$, 1.0; and

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction, or if the bolts are countersunk, the thickness of the plate minus one half of the depth of countersinking.

The bearing resistance (in the direction normal to the slots in slotted holes) of bolts in holes other than standard clearance holes may be reduced by multiplying the bearing resistance obtained as above, V_{npb}, by the factors given below:

a) Over size and short slotted holes – 0.7; and

b) Long slotted holes – 0.5.

NOTE — The block shear of the edge distance due to bearing force may be checked as given in 12.4.

17.3.5 *Tension Capacity*

A bolt subjected to a factored tensile force, T_b shall satisfy,

$$T_b \leq T_{db}$$

where

T_{db} = T_{nb} / γ_{mb}; and

T_{nb} = nominal tensile capacity of the bolt, calculated as.

$$T_{nb} = 0.90\ f_{ub} A_n < f_{yb} A_{sb} (\gamma_{mb} / \gamma_{m0})$$

where

A_{sb} = shank area of the bolt;

A_n = net tensile stress area as specified in the appropriate Indian Standard (for bolts where the tensile stress area is not defined, An shall be taken as the area at the bottom of the threads);

f_{ub} = ultimate tensile stress of the bolt; and

f_{yb} = yield stress of the bolt.

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17.3.6 *Bolt Subjected to Combined Shear and Tension*

A bolt required to resist both design shear force (V_{sd}) and design tensile force (T_b) at the same time shall satisfy,

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0$$

where

V_{db} = design shear capacity (*see* 17.3.2);

V_{sb} = factored shear force acting on the bolt;

T_b = factored tensile force acting on the bolt, and

T_{db} = design tension capacity (*see* 17.3.5).

17.4 Friction Grip Type Bolting

17.4.1 In friction grip type bolting, initial pretension in bolt (usually high strength) develops clamping force at the interfaces of elements being joined. The frictional resistance to slip between the plate surfaces subjected to clamping force opposes slip due to externally applied shear. Friction grip type bolts and nuts shall conform to the accepted standard [C-6A(39)]. Their installation procedures shall conform to good practice [C-6A(15)].

17.4.2 Where slip between bolted plates cannot be tolerated at working loads/ strength limit states loads (slip critical connections), the requirements of 17.4.3 shall be satisfied at service and strength limit states loads, respectively. However, the requirements of 17.4.4 shall be satisfied by all connections at the strength limit states.

17.4.3 *Slip Resistance*

Design for friction type bolting, where slip resistance is required at factored design force V_{sf}, shall satisfy the following:

$$V_{sf} \leq V_{dsf}$$

where

V_{dsf} = V_{nsf} / γ_{mf}; and

V_{nsf} = nominal shear capacity of a bolt as governed by slip for friction type connection, calculated as follows.

$$V_{nsf} = \mu_f n_e K_h F_o$$

where

A_{nb} = net area of the bolt at threads;

f_o = proof stress (= 0.70 f_{ub});

F_o = minimum bolt tension (proof load) at installation and may be taken as A_{nb} f_o;

K_h = 1.0 for fasteners in clearance holes;

= 0.85 for fasteners in oversized and short slotted holes and for fasteners in long slotted holes loaded perpendicular to the slot;

n_e = number of effective interfaces offering frictional resistance to slip;

μ_f = coefficient of friction (slip factor) as specified in Table 21 ($\mu_f \leq 0.55$);

= 0.7 for fasteners in long slotted holes loaded parallel to the slot;

γ_{mf} = 1.10 (if slip resistance is designed at service load); and

= 1.25 (if slip resistance is designed at ultimate load).

NOTE — V_{nsf} may be evaluated at a service load or ultimate load using appropriate partial safety factors, depending upon whether slip resistance is required at service load or ultimate load.

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Table 21 Typical Average Values for Coefficient of Friction (μ_f)

(*Clause* 17.4.3)

Sl No.	Treatment of Surface	Coefficient of Friction, μ_f
(1)	(2)	(3)
i)	Surfaces not treated	0.20
ii)	Surfaces blasted with short or grit with any loose rust removed, no pitting	0.50
iii)	Surfaces blasted with shot or grit and hot-dip galvanized	0.10
iv)	Surfaces blasted with shot or grit and spray-metallized with zinc (thickness 50 µm to 70 µm)	0.25
v)	Surfaces blasted with shot or grit and painted with ethyl zinc silicate coat (thickness 30 µm to 60 µm)	0.30
vi)	Sand blasted surface, after light rusting	0.52
vii)	Surfaces blasted with shot or grit and painted with ethyl zinc silicate coat (thickness 60 µm to 80 µm)	0.30
viii)	Surfaces blasted with shot or grit and painted with alkali zinc silicate coat (thickness 60 µm to 80 µm)	0.30
ix)	Surface blasted with shot or grit and spray metallized with aluminium (thickness > 50 µm)	0.50
x)	Clean mill scale	0.33
xi)	Sand blasted surface	0.48
xii)	Red lead painted surface	0.1

17.4.3.1 *Long Joints*

The provision for the long joints in **17.3.3.1** shall apply to friction grip connections also.

17.4.4 *Capacity after Slipping*

When friction type bolts are designed not to slip only under service loads, the design capacity at ultimate load may be calculated as per bearing type connection (*see* **17.3.2** and **17.3.3**).

NOTE — The block shear resistance of the edge distance due to bearing force may be checked as given in **13.4**.

17.4.5 *Tension Resistance*

A friction bolt subjected to a factored tension force (T_f) shall satisfy:

$$T_f \leq T_{df}$$

where

$T_{df} = T_{nf} / \gamma_{mf}$

T_{nf} = nominal tensile strength of the friction bolt, calculated as:

$$T_{nf} = 0.9\, f_{ub}\, A_n \leq f_{yb}\, A_{sb} (\gamma_{m1} / \gamma_{m0})$$

where

A_n = net tensile stress area as specified in various parts of the accepted standard [C-6A(40)] (for bolts where the tensile stress area is not defined, An shall be taken as the area at the root of the threads);

A_{sb} = shank area of the bolt;

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f_{ub} = ultimate tensile stress of the bolt; and

γ_{mf} = partial factor of safety.

17.4.6 *Combined Shear and Tension*

Bolts in a connection for which slip in the serviceability limit state shall be limited, which are subjected to a tension force, T, and shear force, V, shall satisfy:

$$\left(\frac{V_{sf}}{V_{df}}\right)^2 + \left(\frac{T_f}{T_{df}}\right)^2 \leq 1.0$$

where

V_{df} = design shear strength;

V_{sf} = applied factored shear at design load;

T_{df} = design tension strength; and

17.4.7 Where prying force, Q as illustrated in Fig. 16 shall be calculated as given below and added to the tension in the bolt:

$$Q = \frac{l_v}{2\,l_e}\left[T_e - \frac{\beta\eta\, f_o b_e t^4}{27 l_e l_v^2}\right]$$

where

l_e = distance between prying force and bolt centreline and is the minimum of either the end distance or the value given by; and

l_v = distance from the bolt centreline to the toe of the fillet weld or to half the root radius for a rolled section.

$$l_e = 1.1\, t \sqrt{\frac{\beta\, f_o}{f_y}}$$

where

β = 2 for non pre-tensioned bolt and 1 for pre-tensioned bolt;

b_e = effective width of flange per pair of bolts;

f_o = proof stress in consistent units;

η = 1.5; and

t = thickness of the end plate.

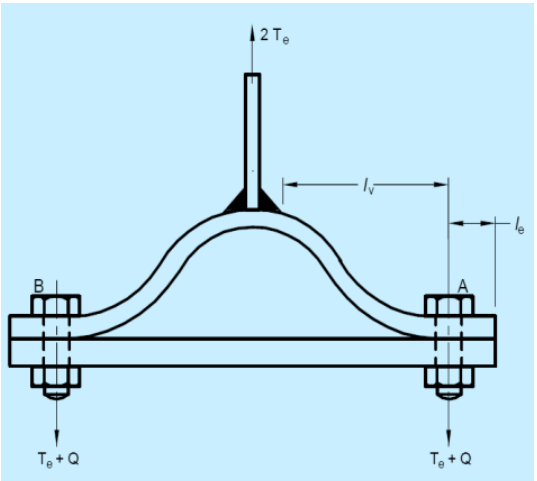

FIG. 16 COMBINED PRYING FORCE AND TENSION

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17.5 Welds and Welding

17.5.1 *General*

Requirements of welds and welding shall conform to the accepted standards [C-6A(41)], as appropriate.

17.5.1.1 *End returns*

Fillet welds terminating at the ends or sides of parts should be returned continuously around the corners for a distance of not less than twice the size of the weld, unless it is impractical to do so. This is particularly important on the tension end of parts carrying bending loads.

17.5.1.2 *Lap joint*

In the case of lap joints, the minimum lap should not be less than four times the thickness of the thinner part joined or 40 mm, whichever is more. Single end fillet should be used only when lapped parts are restrained from openings. When end of an element is connected only by parallel longitudinal fillet welds, the length of the weld along either edge should not be less than the transverse spacing between longitudinal welds.

17.5.1.3 A single fillet weld should not be subjected to moment about the longitudinal axis of the weld.

17.5.2 *Size of Weld*

17.5.2.1 The size of normal fillets shall be taken as the minimum weld leg size. For deep penetration welds, where the depth of penetration beyond the root run is a minimum of 2.4 mm, the size of the fillet should be taken as the minimum leg size plus 2.4 mm.

17.5.2.2 For fillet welds made by semi-automatic or automatic processes, where the depth of penetration is considerably in excess of 2.4 mm, the size shall be taken considering actual depth of penetration subject to agreement between the purchaser and the contractor.

17.5.2.3 The size of fillet welds shall not be less than 3 mm. The minimum size of the first run or of a single run fillet weld shall be as given in Table 22, to avoid the risk of cracking in the absence of preheating.

17.5.2.4 The size of butt weld shall be specified by the effective throat thickness.

17.5.3 *Effective Throat Thickness*

17.5.3.1 The effective size of a fillet weld shall not be less than 3 mm and shall generally not exceed 0.7*t*, or 1.0*t* under special circumstances, where *t* is the thickness of the thinner plate of elements being welded.

17.5.3.2 For the purpose of stress calculation in fillet welds joining faces inclined to each other, the effective throat thickness shall be taken as *K* times the fillet size, where *K* is a constant, depending upon the angle between fusion faces, as given in Table 23.

Table 22 Minimum Size of First Run or of a Single Run Fillet Weld

(*Clause* 17.5.2.3)

Sl No.	Thickness of Thicker Part mm		Minimum Size of First run mm
	Over	Up to and Including	
(1)	(2)	(3)	(4)
i)	—	10	3
ii)	10	20	5
iii)	20	32	6

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Table 22 (*Concluded*)

Sl No.	Thickness of Thicker Part mm		Minimum Size of First run mm
	Over	Up to and Including	
(1)	(2)	(3)	(4)
iv)	32	50	8 of first run 10 for minimum size of weld

NOTES

1 When the minimum size of the fillet weld given in the table is greater than the thickness of the thinner part, the minimum size of the weld should be equal to the thickness of the thinner part. The thicker part shall be adequately preheated to prevent cracking of the weld.

2 Where the thicker part is more than 50 mm thick, special precautions like preheating should be taken.

Table 23 Values of *K* for Different Angles Between Fusion Faces

(*Clause* 17.5.3.2)

Sl No.	Angle Between Fusion Faces	Constant, *K*
(1)	(2)	(3)
i)	60° to 90°	0.70
ii)	91° to 100°	0.65
iii)	101° to 106°	0.60
iv)	107° to 113°	0.55
v)	114° to 120°	0.50

17.5.3.3 The effective throat thickness of a complete penetration butt weld shall be taken as the thickness of the thinner part joined, and that of an incomplete penetration butt weld shall be taken as the minimum thickness of the weld metal common to the parts joined, excluding reinforcements.

17.5.4 *Effective Length or Area of Weld*

17.5.4.1 The effective length of fillet weld shall be taken as only that length which is of the specified size and required throat thickness. In practice the actual length of weld is made of the effective length shown in drawing plus two times the weld size, but not less than four times the size of the weld.

17.5.4.2 The effective length of butt weld shall be taken as the length of the continuous full size weld, but not less than four times the size of the weld.

17.5.4.3 The effective area of a plug weld shall be considered as the nominal area of the hole in the plane of the faying surface. These welds shall not be designed to carry stresses.

17.5.4.4 If the maximum length l_j of the side welds transferring shear along its length exceeds 150 times the throat size of the weld, t_t, the reduction in weld strength as per the long joint (*see* **17.5.7.3**) should be considered. For flange to web connection, where the welds are loaded for the full length, the above limitation would not apply.

17.5.5 *Intermittent Welds*

17.5.5.1 Unless otherwise specified, the intermittent fillet welding shall have an effective length of not less than four times the weld size, with a minimum of 40 mm.

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17.5.5.2 The clear spacing between the effective lengths of intermittent fillet weld shall not exceed 12 and 16 times the thickness of thinner plate joined, for compression and tension joint, respectively, and in no case be more than 200 mm.

17.5.5.3 Unless otherwise specified, the intermittent butt weld shall have an effective length of not less than four times the weld size and the longitudinal space between the effective length of welds shall not be more than 16 times the thickness of the thinner part joined. The intermittent welds shall not be used in positions subject to dynamic, repetitive and alternating stresses.

17.5.6 *Weld Types and Quality*

For the purpose of this NBCS, weld shall be fillet, butt, slot or plug or compound welds. Welding electrodes shall conform to IS 814.

17.5.7 *Design Stresses in Welds*

17.5.7.1 *Shop welds*

17.5.7.1.1 *Fillet welds*

Design strength of a fillet weld, f_{wd} shall be based on its throat area and shall be given by:

$$f_{wd} = f_{wn} / \gamma_{mw}$$

where

f_u = smaller of the ultimate stress of the weld or of the parent metal;

f_{wn} = $f_u / \sqrt{3}$; and

γ_{mw} = partial safety factor (*see* Table 5).

17.5.7.1.2 *Butt welds*

Butt welds shall be treated as parent metal with a thickness equal to the throat thickness, and the stresses shall not exceed those permitted in the parent metal.

17.5.7.1.3 *Slot or plug welds*

The design shear stress on slot or plug welds shall be as per **17.5.7.1.1**.

17.5.7.2 *Site welds*

The design strength in shear and tension for site welds made during erection of structural members shall be calculated according to **17.5.7.1** but using a partial safety factor γ_{mw} of 1.5.

17.5.7.3 *Long joints*

When the length of the welded joint, l_j of a splice or end connection in a compression or tension element is greater than 150 t_t, the design capacity of weld (*see* **17.5.7.1.1**), f_{wd} shall be reduced by the factor:

$$\beta_{lw} = 1.2 - \frac{0.2 l_j}{150 t_t} \leq 1.0$$

where

l_j = length of the joint in the direction of the force transfer; and

t_t = throat size of the weld.

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17.5.8 *Fillet Weld Applied to the Edge of a Plate or Section*

17.5.8.1 Where a fillet weld is applied to the square edge of a part, the specified size of the weld should generally be at least 1.5 mm less than the edge thickness in order to avoid washing down of the exposed corner [*see* Fig. 17(A)].

17.5.8.2 Where the fillet weld is applied to the rounded toe of a rolled section, the specified size of the weld should generally not exceed 3/4 of the thickness of the section at the toe [*see* Fig. 17(B)].

17.5.8.3 Where the size specified for a fillet weld is such that the parent metal will not project beyond the weld, no melting of the outer cover or covers shall be allowed to occur to such an extent as to reduce the throat thickness (*see* Fig. 18).

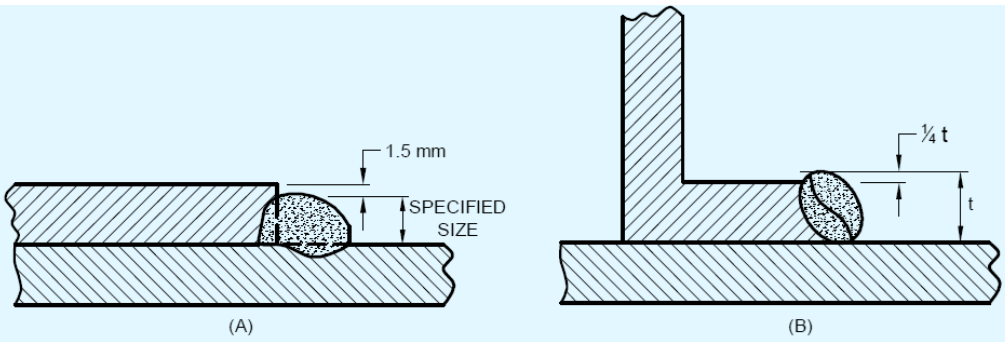

FIG. 17 FILLET WELDS ON SQUARE EDGE OF PLATE OR ROUND TOE OF ROLLED SECTION

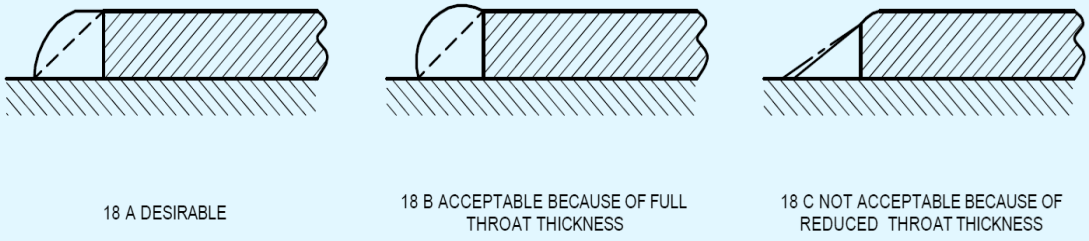

FIG. 18 FULL SIZE FILLET WELD APPLIED TO THE EDGE OF A PLATE OR SECTION

17.5.8.4 When fillet welds are applied to the edges of a plate or section in members subject to dynamic loading, the fillet weld shall be of full size with its leg length equal to the thickness of the plate or section, with the limitations specified in **17.5.8.3**.

17.5.8.5 End fillet weld, normal to the direction of force shall be of unequal size with a throat thickness not less than 0.5*t*, where *t* is the thickness of the part, as shown in Fig. 19. The difference in thickness of the welds shall be negotiated at a uniform slope.

17.5.9 *Stresses Due to Individual Forces*

When subjected to either compressive or tensile or shear force alone, the stress in the weld is given by and shall be less than or equal to design strength of the weld, f_{ws}, given in **17.5.7.1.1**:

$$f_a \text{ or } q = \frac{P}{t_t \, l_w}$$

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where

f_a = Calculated, in N/mm^2, normal stress due to axial force;

l_w = Effective, in mm, length of weld;

P = force transmitted (axial force N or the shear force Q);

q = shear, in N/mm^2, stress; and

t_t = Effective, in mm; throat thickness of weld.

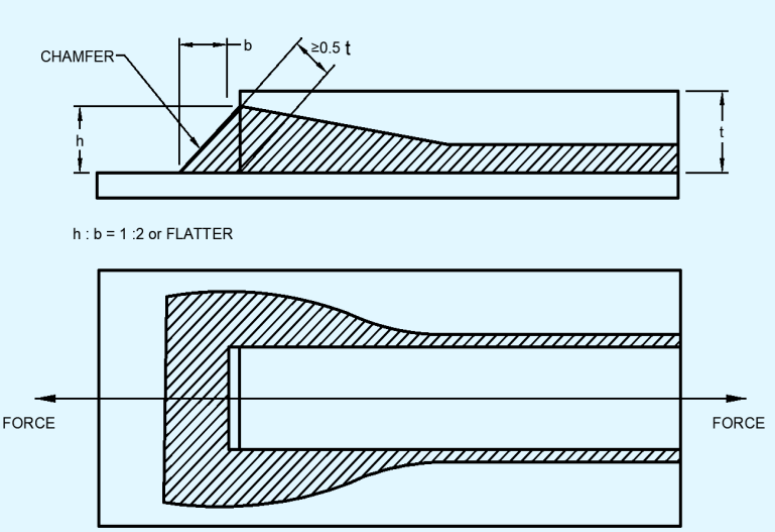

FIG. 19 END FILLET WELD NORMAL TO DIRECTION OF FORCE

17.5.10 *Combination of Stresses*

17.5.10.1 *Fillet welds*

17.5.10.1.1 When subjected to a combination of normal and shear stress, the equivalent stress f_e shall satisfy the following:

$$f_e = \sqrt{f_a^2 + 3q^2} \leq \frac{f_u}{\gamma_{mw}}$$

where

f_a = normal stresses, compression or tension, due to axial force or bending moment (*see* 17.5.9); and

q = shear stress due to shear force or tension (*see* 17.5.9).

17.5.10.1.2 Check for the combination of stresses need not be done for:

a) side fillet welds joining cover plates and flange plates; and

b) fillet welds where algebraic sum of normal and shear stresses does not exceed f_{wd} (*see* 17.5.7.1.1).

17.5.10.2 *Butt welds*

17.5.10.2.1 Check for the combination of stresses in butt welds need not be carried out provided that:

a) butt welds are axially loaded; and

b) in single and double bevel welds the algebraic sum of normal and shear stresses does not exceed the design normal stress, and the shear stress does not exceed 50 percent of the design shear stress.

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17.5.10.2.2 *Combined bearing, bending and shear*

Where bearing stress, f_{br} is combined with bending (tensile or compressive), f_b and shear stresses, q under the most unfavorable conditions of loading in butt welds, the equivalent stress, f_e as obtained from the following formula, shall not exceed the values allowed for the parent metal:

$$f_e = \sqrt{f_b{}^2 + f_{br}{}^2 - f_b f_{br} + 3q^2}$$

where

f_b = calculated, in N/mm^2; stress due to bending;

f_{br} = calculated, in N/mm^2, stress due to bearing;

f_e = equivalent stress; and

q = shear, in N/mm^2, stress.

17.5.11 Where a packing is welded between two members and is less than 6 mm thick, or is too thin to allow provision of adequate welds or to prevent buckling, the packing shall be trimmed flush with the edges of the element subject to the design action and the size of the welds along the edges shall be increased over the required size by an amount equal to the thickness of the packing. Otherwise, the packing shall extend beyond the edges and shall be fillet welded to the pieces between which it is fitted.

17.6 Design of Connections

Each element in a connection shall be designed so that the structure is capable of resisting the design actions. Connections and adjacent regions of the members shall be designed by distributing the design action effects such that the following requirements are satisfied:

a) Design action effects distributed to various elements shall be in equilibrium with the design action effects on the connection;

b) Required deformations in the elements of the connections are within their deformations capacities;

c) All elements in the connections and the adjacent areas of members shall be capable of resisting the design action effects acting on them; and

d) Connection elements shall remain stable under the design action effects and deformations.

17.6.1 Connections can be classified as rigid, semi-rigid and flexible for the purpose of analysis and design as per the recommendation in Annex F. Connections with sufficient rotational stiffness may be considered as rigid. Examples of rigid connections include flush end-plate connection and extended end-plate connections. Connections with negligible rotational stiffness may be considered as flexible (pinned). Examples of flexible connections include single and double web angle connections and header plate connections. Where a connection cannot be classified as either rigid or flexible, it shall be assumed to be semi-rigid. For Example, of semi-rigid connections include top and seat angle connection and top and seat angle with single/double web angles.

17.6.2 Design shall be on the basis of any rational method supported by experimental evidence. Residual stresses due to installation of bolts or welding normally need not be considered in statically loaded structures. Connections in cyclically loaded structures shall be designed considering fatigue as given under **19**. For earthquake load combinations, the connections shall be designed to withstand the calculated design action effects as specified in **18**.

17.6.3 Beam and column splice shall be designed in accordance with the recommendation given in **F-2** and **F-3**.

17.7 Minimum Design Action on Connection

Connections carrying design action effects, except for lacing connections, connections of sag rods, purlins and girts, shall be designed to transmit the greater of:

a) the design action in the member; and

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b) The minimum design action effects expressed either as the value or the factor times the member design capacity for the minimum size of member required by the strength limit state, specified as follows:

1) *Connections in rigid construction* — A bending moment of at least 0.5 times the member design moment capacity;

2) *Connections to beam in simple construction* — A shear force of at least 0.15 times the member design shear capacity or 40 kN, whichever is lesser;

3) *Connections at the ends of tensile or compression member* — A force of at least 0.3 times the member design capacity;

4) *Splices in members subjected to axial tension* — A force of at least 0.3 times the member design capacity in tension;

5) *Splices in members subjected to axial compression* — For ends prepared for full contact in accordance with **24.7.1**, it shall be permissible to carry compressive actions by bearing on contact surfaces. When members are prepared for full contact to bear at splices, there shall be sufficient fasteners to hold all parts securely in place. The fasteners shall be sufficient to transmit a force of at least 0.15 times the member design capacity in axial compression;

When members are not prepared for full contact, the splice material and its fasteners shall be arranged to hold all parts in line and shall be designed to transmit a force of at least 0.3 times the member design capacity in axial compression.

In addition, splices located between points of effective lateral support shall be designed for the design axial force, P_d plus a design bending moment, not less than the design bending moment $M_d = P_d l_s / 1\,000$ where, l_s is the distance between points of effective lateral support.

6) *Splices in flexural members* — A bending moment of 0.3 times the member design capacity in bending. This provision shall not apply to splices designed to transmit shear force only; and

A splice subjected to a shear force only shall be designed to transmit the design shear force together with any bending moment resulting from the eccentricity of the force with respect to the centroid of the group.

7) *Splices in members subject to combined actions* — A splice in a member subject to a combination of design axial tension or design axial compression and design bending moment shall satisfy requirements in (4), (5) and (6) above, simultaneously.

17.8 Intersections

Members or components meeting at a joint shall be arranged to transfer the design actions between the parts, wherever practicable, with their centroidal axes meeting at a point. Where there is eccentricity at joints, the members and components shall be designed for the design bending moments which result due to eccentricity.

The disposition of fillet welds to balance the design actions about the centroidal axis or axes for end connections of single angle, double angle and similar type members is not required for statically loaded members but is required for members, connection components subject to fatigue loading.

Eccentricity between the centroidal axes of angle members and the gauge lines for their bolted end connections may be neglected in statically loaded members, but shall be considered in members and connection components subject to fatigue loading.

17.9 Choice of Fasteners

Where slip in the serviceability limit state is to be avoided in a connection, high-strength bolts in a friction-type joint, fitted bolts or welds shall be used.

Where a joint is subjected to impact or vibration, either high strength bolts in a friction type joint or ordinary bolts with locking devices or welds shall be used.

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17.10 Connection Components

Connection components (cleats, gusset plates, brackets and the like) other than connectors, shall have their capacities assessed using the provisions of 12, 13, 14, 15 and 16, as applicable.

17.11 Analysis of a Bolt/Weld Group

17.11.1 *Bolt/Weld Group Subject to In-plane Loading*

17.11.1.1 *General method of analysis*

The design force in a bolt/weld or design force per unit length in a bolt/weld group subject to in-plane loading shall be determined in accordance with the following:

a) The connection plates shall be considered to be rigid and to rotate relative to each other about a point known as the instantaneous centre of rotation of the group;

b) In the case of a group subject to a pure couple only, the instantaneous centre of rotation coincides with the group centroid. In the case of in-plane shear force applied at the group centroid, the instantaneous centre of the rotation is at infinity and the design force is uniformly distributed throughout the group. In all other cases, either the results of independent analyses for a pure couple alone and for an in-plane shear force applied at the group centroid shall be superposed, or a recognized method of analysis shall be used; and

c) The design force in a bolt or design force per unit length at any point in the group shall be assumed to act at right angles to the radius from that point to the instantaneous centre, and shall be taken as proportional to that radius.

17.11.2 *Bolt/Weld Group Subject to Out-of-Plane Loading*

17.11.2.1 *General method of analysis*

The design force of a bolt in bolt group or design force per unit length in the fillet weld group subject to out-of-plane loading shall be determined in accordance with the following:

a) Design force in the bolts or per unit length in the fillet weld group resulting from any shear force or axial force shall be considered to be equally shared by all bolts in the group or uniformly distributed over the length of the fillet weld group; and

b) Design force resulting from a design bending moment shall be considered to vary linearly with distance from the relevant centroidal axes. The following shall also be considered:

   1) in bearing type of bolt group plates in the compression side of the neutral axis and only bolts in the tension side of the neutral axis may be considered for calculating the neutral axis and second moment of area;

   2) in the friction grip bolt group, only the bolts shall be considered in the calculation of neutral axis and second moment of area; and

   3) the fillet weld group shall be considered in isolation from the connected element; for the calculation of centroid and second moment of the weld length.

17.11.2.2 *Alternative analysis*

The design force per unit length in a fillet weld/bolt group may alternatively be determined by considering the fillet weld group as an extension of the connected member and distributing the design forces among the welds of the fillet weld group so as to satisfy equilibrium between the fillet weld group and the elements of the connected member.

17.11.3 *Bolt/Weld Group Subject to In-plane and Out-of-Plane Loading*

17.11.3.1 *General method of analysis*

The design force in a bolt or per unit length of the weld shall be determined by the superposition of analysis for in-plane and out-of-plane cases discussed in 17.11.1 and 17.11.2.

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17.11.3.2 *Alternative analysis*

The design force in a bolt or per unit length in the fillet weld group may alternatively be determined by considering the fillet weld group as an extension of the connected member and proportioning the design force per bolt or unit length in the weld group to satisfy equilibrium between the bolt/weld group and the elements of the connected member.

Force calculated in the most stressed bolt or highest force per unit length of the weld shall satisfy the strength requirements of **17.3**, **17.4** or **17.5**, as appropriate.

17.12 Lug Angles

17.12.1 Lug angles connecting outstanding leg of a channel-shaped member shall, as far as possible, be disposed symmetrically with respect to the section of the member.

17.12.2 In the case of angle members, the lug angles and their connections to the gusset or other supporting member shall be capable of developing a strength not less than 20 percent in excess of the force in the outstanding leg of the member, and the attachment of the lug angle to the main angle shall be capable of developing a strength not less than 40 percent in excess of the force in the outstanding leg of the angle.

17.12.3 In the case of channel members and the like, the lug angles and their connection to the gusset or other supporting member shall be capable of developing a strength of not less than 10 percent in excess of the force not accounted for by the direct connection of the member, and the attachment of the lug angles to the member shall be capable of developing 20 percent in excess of that force.

17.12.4 In no case shall fewer than two bolts, rivets or equivalent welds be used for attaching the lug angle to the gusset or other supporting member.

17.12.5 The effective connection of the lug angle shall, as far as possible terminate at the end of the member connected, and the fastening of the lug angle to the main member shall preferably start in advance of the direct connection of the member to the gusset or other supporting member.

17.12.6 Where lug angles are used to connect an angle member, the whole area of the member shall be taken as effective notwithstanding the requirements given under **13**.

18 DESIGN AND DETAILING FOR EARTHQUAKE EFFECTS

18.1 General

18.1.1 Earthquake is a ‘displacement loading’ on a structure. This implies that ‘displacements’ are ‘imposed’ under a structure during earthquake shaking and ‘forces’ are induced in it. Thus, a structure is allowed to be designed to have lesser strength capacity than that required if it were to remain elastic. And, only chosen types of damage are permitted to be accrued in select components of a structure during strong earthquake shaking.

Thus, reliance is placed on the nonlinear response of a structure, especially on its deformation and ductility capacities. These must be achieved by adopting special design and detailing practices. Therefore, steel frames shall be so designed and detailed as to give them stability, and adequate stiffness, strength, and ductility capacities to resist severe earthquake shaking without collapse in all earthquake zones mentioned in **5** of Part C ‘Structural Design/Section 1 Loads and Load Effects’ of this NBCS.

18.1.2 Steel frames required to resist earthquake effects in all earthquake zones shall satisfy the requirements specified in good practice [C-6A(42)]. Frames are classified into two types, one which have minimal inelastic deformation and ductility capacities, called ordinary frames, and one with higher deformation and ductility capacities, called special frames. For earthquake zones III and above, only special frames are permitted to be used.

18.1.3 For earthquake zone II, ordinary frames, namely ordinary moment frames (OMF) and ordinary concentrically braced frames (OCBF), are permitted to be used. In design and detailing of OMFs and OCBFs, specific requirements of this Section shall be satisfied along with other provisions of this NBCS, in addition to the earthquake demand and design criteria requirements of **5** of Part C ‘Structural design/Section 1 Loads and Load Effects’ of this NBCS.

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18.1.4 Frames, which are a part of the gravity load resisting system and are not intended to resist the lateral earthquake forces, need not satisfy the requirements of this section, provided they can accommodate the resulting earthquake shaking induced deformation without premature failure.

18.2 Loads and Load Combinations

18.2.1 Earthquake loads shall be calculated as per **5** of Part C 'Structural design/Section 1 Loads and Load Effects' of this NBCS, except that the reduction factors recommended in **18.3** may be used.

18.2.2 In the limit state design of frames resisting earthquake loads, the load combinations shall conform to Table 4.

18.2.3 In addition the following load combination shall be considered as required in **18.5.1.1**, **18.7.3.1**, **18.11.2.2** and **18.11.3.4**:

a) 1.2 Dead Load (DL) + 0.5 Imposed Load (IL) ± 2.5 Earthquake Force (E); and

b) 0.9 Dead Load (DL) ± 2.5 Earthquake Force (E).

18.3 Response Reduction Factor

18.3.1 For structures designed and detailed in accordance with the provisions of this Section, the response reduction factors specified herein may be used for calculating the design earthquake forces, in conjunction with the provisions of **5** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS.

18.3.2 For braced frame systems, the response reduction factor (R) for ordinary concentrically braced frames (OCBF), for special concentrically braced frames (SCBF) and for special eccentrically braced frames (SEBF) shall be taken from **5** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS. The response reduction factor (R) for ordinary eccentrically braced frames (OEBF) shall be taken as 4.5.

18.3.3 For moment frame systems, the response reduction factor (R) for special moment frames (SMF) shall be taken from **5** of Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS. For the Ordinary Moment Frames (OMF) designed as per **18.10**, the response reduction factor (R) shall be taken as 4.0.

18.3.4 For non-ductile frames (NDF), the response reduction factor (R) shall be taken as 1.5.

18.3.5 The braced and moment resisting frames in single storey industrial buildings can be designed as non-ductile frames (NDF), if their overall height to overall width ratio does not exceed 1.0. Such frames need not meet the ductility requirements of other types of frames as specified in **18.4** to **18.12**.

18.4 Connections, Joints and Fasteners

18.4.1 All bolts used in critical zones of frames designed to resist earthquake effects shall be fully tensioned high strength friction grip (HSFG) bolts or turned and fitted bolts. The critical zones of frames are zones of plastic hinges of moment resisting frames and beam to columns as well as bracing and their connections of braced frames. The plastic hinge zones shall be taken as a distance equal to 2 times the depth of the member from the section of maximum bending moment.

18.4.2 All welds used in frames designed to resist earthquake effects shall be complete penetration butt welds, except in column splices, which shall conform to **18.5.2**.

18.4.3 Bolted joints shall be designed not to share load in combination with welds on the same faying surface.

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18.5 Columns

18.5.1 *Column Strength*

When P_u/P_d *is* greater than 0.4, the requirements in **18.5.1.1** and **18.5.1.2** shall be met.

Where

P_u = required ultimate compressive strength of the member; and

P_d = design resistance in axial compression as obtained from **14.1.2**.

18.5.1.1 The required axial compressive and axial tensile strength in the absence of applied moment, shall be determined from the load combination in **18.2.3**.

18.5.1.2 The required strength determined in **18.5.1.1** need not exceed either of the maximum load transferred to the column considering 1.2 times the nominal strength of the connecting beam or brace element, or the resistance of the foundation to uplift.

18.5.2 *Column Splice*

18.5.2.1 A partial-joint penetration groove weld may be provided in column splice, such that the design strength of the joints shall be at least equal to 200 percent of the required strength.

18.5.2.2 The minimum required strength for each flange splice shall be 1.2 times $A_f f_y$ as shown Fig. 20, where A_f is the area of each flange in the smaller connected column.

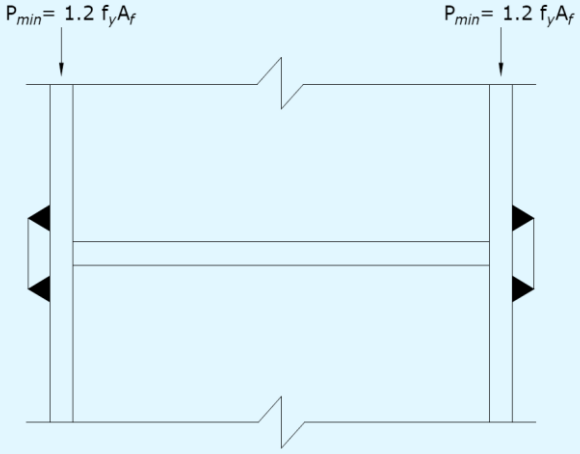

FIG. 20 PARTIAL PENETRATION GROOVE WELD IN COLUMN SPLICES

18.6 Storey Drift

The storey drift limits shall conform to **5** of Part C 'Structural design/Section 1 Loads and Load Effects' of this NBCS. The deformation compatibility of members not designed to resist seismic lateral load shall also conform to **5** of Part C 'Structural design/Section 1 Loads and Load Effects' of this NBCS.

18.7 Ordinary Concentrically Braced Frames (OCBF)

18.7.1 Ordinary concentrically braced frames (OCBF) should be shown to withstand inelastic deformation a moment of 1.2 times the full plastic moment of the corresponding to a joint rotation of at least 0.02 radians without

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degradation in strength and stiffness below the full yield value. Ordinary concentrically braced frames meeting the requirements of this section shall be deemed to satisfy the required inelastic deformation.

18.7.1.1 Ordinary concentrically braced frames shall not be used in seismic zones IV and above and for buildings with importance factor greater than unity (1 > 1.0) in
seismic zone III.

18.7.1.2 The provision in this section apply for diagonal and X-bracing only. Specialist literature may be consulted for V and inverted V-type bracing. K-bracing shall not be permitted in systems to resist earthquake.

18.7.2 *Bracing Members*

18.7.2.1 The slenderness of bracing members shall not exceed 120.

18.7.2.2 The required compressive strength of bracing member shall not exceed 0.8 times P_d, where P_d is the design strength in axial compression (*see* **14.1.2**).

18.7.2.3 Along any line of bracing, braces shall be provided such that for lateral loading in either direction, the tension braces will have to resist between 30 to 70 percent of the total lateral load.

18.7.2.4 Bracing cross-section can be plastic, compact or semi-compact, but not slender, as defined in **10.8.2**.

18.7.2.5 For all built-up braces, the spacing of tack fasteners shall be such that the unfavorable slenderness ratio of individual element, between such fasteners, shall not exceed 0.4 times the governing slenderness ratio of the brace itself. Bolted connections shall be avoided within the middle one-fourth of the clear brace length (0.25 times the length in the middle).

18.7.2.6 The bracing members shall be designed so that gross area yielding (*see* **13.2**) and not the net area rupture (*see* **13.3**) would govern the design tensile strength.

18.7.3 *Bracing Connections*

18.7.3.1 End connections in bracings shall be designed to withstand the minimum of the following:

a) A tensile force in the bracing equal to 1.2 $f_y A_g$;

b) Force in the brace due to load combinations in **18.2.3**; and

c) Maximum force that can be transferred to the brace by the system.

18.7.3.2 The connection should be checked for tension rupture and block shear under the load determined in **18.7.3.1**.

18.7.3.3 The connection shall be designed to withstand should be shown to withstand inelastic deformation a moment of 1.2 times the full plastic moment of the corresponding to a joint rotation of at least 0.02 radians braced section about the buckling axis.

18.7.3.4 Gusset plates shall be checked for buckling out of their plane.

18.8 Special Concentrically Braced Frames (SCBF)

18.8.1 Special concentrically braced frames (SCBF) should be shown to withstand inelastic deformation corresponding to a joint rotation of at least 0.04 radians without degradation in strength and stiffness below the full yield value. Special concentrically braced frames meeting the requirements of this section shall be deemed to satisfy the required inelastic deformation.

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18.8.1.1 Special concentrically braced frames (SCBF) may be used in any seismic zone as defined in **5** of Part C 'Structural design/Section 1 Loads and Load Effects' of this NBCS and for any building.

18.8.1.2 The provision in this section apply for diagonal and X-bracing only. Specialist literature may be consulted for V and inverted V-type bracing. K-bracing shall not be permitted in system to resist earthquake.

18.8.2 *Bracing Members*

18.8.2.1 Bracing members shall be made of E 250B steel of accepted standard [C-6A(6)] only.

18.8.2.2 The slenderness of bracing members shall not exceed 160 (only hangers).

18.8.2.3 The required compressive strength of bracing member shall not exceed the design strength in axial compression P_d (*see* **14.1.2**).

18.8.2.4 Along any line of bracing, braces shall be provided such that for lateral loading in either direction, the tension braces will resist between 30 percent to 70 percent of the load.

18.8.2.5 Braced cross-section shall be plastic as defined in **10.8.2**.

18.8.2.6 In built-up braces, the spacing of tack connections shall be such that the slenderness ratio individual element between such connections shall not exceed 0.4 times the governing slenderness ratio of the brace itself. Bolted connection shall be avoided.

18.8.3.4 Gusset plates shall be checked for buckling out of their plane.

18.8.4 ***Column***

18.8.4.1 The column sections used in special concentrically braced frames (SCBF) shall be plastic as defined in **10.8.2**.

18.8.4.2 Splices shall be located within the middle one-third of the column clear height. Splices shall be designed for the forces that can be transferred to it. In addition, splices in columns shall be designed to develop at least the nominal shear strength of the smaller connected section.

18.9 Eccentrically Braced Frames (EBF)

18.9.1 Ordinary eccentrically braced frames (OEBF) and Special eccentrically braced frames (SEBF) shall be designed in accordance with specialist literature.

18.10 Ordinary Moment Frames (OMFs)

Ordinary moment frames (OMFs) of structural steel shall be designed to satisfy the requirements of this section.

18.10.1 *Basis of Design*

Ordinary moment frames designed in accordance with these provisions are expected to possess limited inelastic deformation capacity through flexural yielding of the beams, panel zones, and columns.

18.10.2 *Analysis*

When two moment frames oriented in orthogonal directions intersect, the possibility of beam yielding in both orthogonal directions simultaneously shall be considered in the estimation of design demand on the common column.

18.10.3 *Member Requirements*

The requirements specified hereunder shall be satisfied by the component or member.

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18.10.3.1 *Beams*

Beam cross-section shall be plastic or compact, as defined in **10.8.2**.

18.10.3.2 *Columns*

Column cross-section shall be plastic or compact, as defined in **10.8.2**.

18.10.4 *System Requirements*

It is preferable to plan buildings to have independent lateral load resisting moment frames in the vertical plane along each principal plan direction. Also, the requirements given hereunder shall be satisfied by the building system.

18.10.4.1 *Beam-to-column connections*

Beam-to-column connections may be rigid or semi-rigid moment connections, but shall satisfy the following criteria:

a) Rigid moment connections shall be designed to withstand a moment of at least 1.2 times of either the full plastic moment of the connected beam or the maximum moment that can be delivered by the beam to the joint due to the induced weakness at the ends of the beam, whichever is less.

b) Semi-rigid connections shall be designed and to withstand either a moment of at least 0.5 times the full plastic moment of the connected beam or the maximum moment that can be delivered by the system, whichever is less. The design moment shall be achieved within a rotation of 0.01 radians. The information given in Annex F may be used for checking.

c) The stiffness and strength of semi-rigid connections shall be accounted for in the design and the overall stability of the frame shall be ensured.

d) The rigid and semi-rigid connections shall be designed to withstand a shear resulting from the load combination 1.2 DL + 0.5 LL plus the shear corresponding to the design moment defined in **(a)** and **(b)**, respectively.

e) In rigid fully welded connections, continuity plates of thickness equal to or greater than the thickness of the beam flange shall be provided and welded to the column flanges and web.

18.10.4.2 *Column to beam strength ratio*

At a beam–column joint, strong column - weak beam design shall be adopted, suitably accounting for the reduction in moment capacity of columns due to axial load.

18.11 Special Moment Frames (SMF)

18.11.1 Special moment frames (SMF) shall be made of E250B steel of accepted standard [C-6A(6)] and should be shown to withstand inelastic deformation corresponding to a joint rotation of 0.04 radians without degradation in strength and stiffness below the full yield value (M_P). Special moment frames meeting the requirements of this section shall be deemed to satisfy the required inelastic deformation.

18,11.1.1 Special moment frames (SMF) maybe used in any seismic zone and for any buildings.

18.11.2 *Beam-to-Column Joints and Connections*

18.11.2.1 All beam-to-column connections shall be rigid (*see* Annex F) and designed to withstand a moment of at least 1.2 times the full plastic moment of the connected beam. When a reduced beam section is used, its minimum flexural strength shall be at least equal to 0.8 times the full plastic moment of the unreduced section.

18.11.2.2 The connection shall be designed to withstand a shear resulting from the load combination 1.2 DL + 0.5LL plus the shear resulting from the application of $1.2M_P$ in the same direction, at each end of the beam (causing double curvature bending). The shear strength need not exceed the required value corresponding to the load combination in **18.2.3**.

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18.11.2.3 In column strong axis connections (beam and column web in the same plane), the panel zone shall be checked for shear buckling in accordance with **15.4.2** at the design shear defined in **18.11.2.2**. Column web doubler plates or diagonal stiffeners may be used to strengthen the web against shear buckling.

18.11.2.4 The individual thickness of the column webs and doubler plates (*see* Fig. 21), shall satisfy the following:

$$t \geq (d_P + b_P)/90$$

where

t = Thickness of the column web or doubler plate

d_P = Panel zone depth between continuity plates

b_P = Panel zone width between column flanges

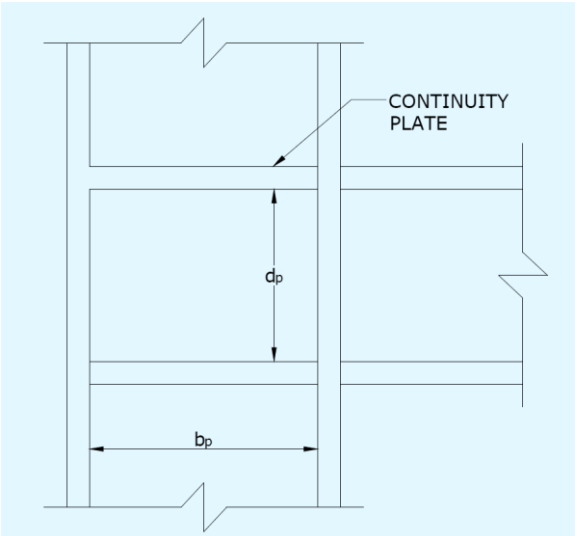

FIG. 21 CONTINUITY PLATES

18.11.2.5 Continuity plates (tension stiffener) (*see* **15.7**) shall be provided in all strong axis welded connections except in end plate connection.

18.11.3 *Beam and Column Limitation*

18.11.3.1 Beam and column sections shall be either plastic or compact as defined in **10.8.2**. At potential plastic hinge locations, they shall necessarily be plastic**.**

18.11.3.2 The section selected for beams and columns shall satisfy the following relation:

$$\frac{\sum M_{Pc}}{\sum M_{Pb}} \geq 1.2$$

where

M_{Pc}= sum of the moment capacity in the column above and below the beam centreline; and

M_{Pb} = sum of the moment capacity in the beams at the intersection of the beam and column centrelines.

In tall buildings, higher mode effects shall be accounted for in accordance with specialist literature.

18.11.3.3 Lateral support to the column at both top and bottom beam flange levels shall be provided so as to resist at least 2 percent of the beam flange strength, except for the case described in **18.11.3.4**.

18.11.3.4 A plane frame designed as non-sway in the direction perpendicular to its plane, shall be checked for buckling, under the load combination specified in **18.2.3**.

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18.12 Column Bases

18.12.1 Fixed column bases and their anchor bolts should be designed to withstand a moment of 1.2 times the full plastic moment capacity of the column section. The anchor bolts shall be designed to withstand the combined action of shear and tension as well as prying action, if any.

18.12.2 Both fixed and hinged column bases shall be designed to withstand the full shear under any load case or 1.2 times the shear capacity of the column section, whichever is higher.

18.13 Industrial Structures

18.13.1 Single storey industrial buildings with frame height to span ratio less than 1.0 and with regular configuration need not meet the ductility requirements of other types of frames specified in **18.3**, **18.4** or in good practice[C-6A(42)], provided they satisfy the requirements in **18.13.2** and **18.13.3**.

18.13.2 Braced frames or moment frames are permitted as lateral force resisting system and shall be designed using R = 1.5 as prescribed for NDF.

18.13.3 Connections in lateral force resisting systems to resist earthquake shall be designed for full member capacity except those in earthquake zone V, where the connections shall be designed for 50 percent extra force.

19 FATIGUE

19.1 General

Structure and structural elements subject to loading that could lead to fatigue failure shall be designed against fatigue as given in this section. This shall however not cover the following:

a) Corrosion fatigue;
b) Low cycle (high stress) fatigue;
c) Thermal fatigue;
d) Stress corrosion cracking;
e) Effects of high temperature (> 150 ºC); and
f) Effects of low temperature (< brittle transition temperature).

For the purpose of design against fatigue, different details (of members and connections) are classified under different fatigue class. The design stress range corresponding to various number of cycles, are given for each fatigue class. The requirements of this Section shall be satisfied with, at each critical location of the structure subjected to cyclic loading, considering relevant number of cycles and magnitudes of stress range expected to be experienced during the life of the structure.

19.2 Design

19.2.1 *Reference Design Condition*

The standard S-N curves for each detail category are given for the following conditions:

a) Detail is located in a redundant load path, wherein local failure at that detail alone will not lead to overall collapse of the structure;
b) Nominal stress history at the local point in the detail is estimated/evaluated by a conventional method without taking into account the local stress concentration effects due to the detail;
c) Load cycles are not highly irregular;
d) Details are accessible for and subject to regular inspection;
e) Structure is exposed to only mildly corrosive environment as in normal atmospheric condition and suitably protected against corrosion (pit depth < 1 mm);

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f) Structure is not subjected to temperature exceeding 150 °C; and

g) Transverse fillet or butt weld connects plates of thickness not greater than 25 mm.

19.2.1.1 *Thickness correction factor*

The values obtained from the standard S-N curve shall be modified by a capacity reduction factor μ_r, when plates greater than 25 mm in thickness are joined together by transverse fillet or butt welding, given by:

$$\mu_r = (25/t_p)^{0.25} \leq 1.0$$

where

t_p = actual thickness of the thicker plate being joined, in mm.

No thickness correction is necessary when full penetration butt weld reinforcements are machined flush and proved free of defect through nondestructive testing.

19.2.1.2 *Necessity for fatigue assessment*

a) Fatigue assessment is not normally required for building structures except as follows; and

1) Members supporting lifting or rolling loads;

2) Member subjected to repeated stress cycles from vibrating machinery;

3) Members subjected to wind induced oscillations of a large number of cycles in life; and

4) Members subjected to crowd induced oscillations of a large number of cycles in life.

b) No fatigue assessment is necessary if the following conditions are satisfied.

1) The highest normal stress range $f_{f\,max}$ satisfies:

$$f_{fmax} \leq 27\mu_r / \gamma_{mft}$$

2) The highest shear stress range τ_{fmax} satisfies:

$$\tau_{fmax} \leq 67\mu_r / \gamma_{mft}$$

3) The total number of actual stress cycles N_{SC}, satisfies:

$$N_{SC} \leq 5\times10^6 \left(\frac{27\mu_r}{\gamma_{mft}\ f_{feq}} \right)^3$$

where

f_{feq} = equivalent constant amplitude stress range, in MPa, given by.

$$f_{feq} = \left[\frac{\sum_{i=1}^{r5} n_i f_i^{\,3} + \sum_{j=r5}^{r} n_j f_j^{\,5}}{n} \right]^{1/3}$$

where

f_i, f_j = stress ranges falling above and below the ffn, the stress range corresponding to the detail at 5×10^6 number of life cycles.

$$n = \sum_{i=1}^{r} n_i$$

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19.2.2 *Design Spectrum*

19.2.2.1 *Stress evaluation*

Design stress shall be determined by elastic analysis of the structure to obtain stress resultants and the local stresses may be obtained by a conventional stress analysis method. The normal and shear stresses shall be determined considering all design actions on the members, but excluding stress concentration due to the geometry of the detail. The stress concentration effect is accounted for in detail category classification (*see* Table 26). The stress concentration, however, not characteristic of the detail shall be accounted for separately in the stress calculation.

In the fatigue design of trusses made of members with open sections, in which the end connections are not pinned, the stresses due to secondary bending moments shall be taken into account, unless the slenderness ratio (*KL*/*r*), of the member is greater than 40.

In the determination of stress range at the end connections between hollow sections, the effect of connection stiffness and eccentricities may be disregarded, provided:

a) the calculated stress range is multiplied by appropriate factor given in Table 24(A) in the case of circular hollow section connections and Table 24(B) in the case of rectangular hollow section connections; and
b) the design throat thickness of fillet welds in the joints is greater than the wall thickness of the connected member.

19.2.2.2 *Design stress spectrum*

In the case of loading events producing non-uniform stress range cycle, the stress spectrum may be obtained by a rational method, such as 'rain flow counting' or an equivalent method.

Table 24(A) Multiplying Factors for Calculated Stress Range (Circular Hollow Sections)

(*Clause* 19.2.2.1)

Sl No.	Type of Connection		Chords	Verticals	Diagonals
(1)	(2)		(3)	(4)	(5)
i)	Gap connections	K type	1.5	1.0	1.3
		N type	1.5	1.8	1.4
ii)	Overlap connections	K type	1.5	1.0	1.2
		N type	1.5	1.65	1.25

Table 24(B) Multiplying Factors for Calculated Stress Range (Rectangular Hollow Sections)

(*Clause* 19.2.2.1)

Sl No.	Types of Joint		Chords	Verticals	Diagonals
(1)	(2)		(3)	(4)	(5)
i)	Gap connections	K type	1.5	1.0	1.5
		N type	1.5	2.2	1.6
ii)	Overlap connections	K type	1.5	1.0	1.3
		N type	1.5	2.0	1.4

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19.2.3 *Partial Safety Factors*

19.2.3.1 *Partial safety factor for actions and their effects* (γ_{ff})

Unless and otherwise the uncertainty in the estimation of the applied actions and their effects demand a higher value, the partial safety factor for loads in the evaluation of stress range in fatigue design shall be taken as 1.0.

19.2.3.2 *Partial safety factor for fatigue strength* (γ_{mft})

Partial safety factor for strength is influenced by consequences of fatigue damage and level of inspection capabilities.

19.2.3.3 Based on consequences of fatigue failure, component details have been classified as given in the Table 25 and the corresponding partial safety factor for fatigue strength shall be used:

a) failsafe structural component/detail is the one where local failure of one component due to fatigue crack does not result in the failure of the structure due to availability of alternate load path (redundant system); and

b) non-failsafe structural component/detail is the one where local failure of one component leads rapidly to failure of the structure due to its non-redundant nature.

Table 25 Partial Safety Factors for Fatigue Strength (γ_{mft})

(*Clauses* 19.2.3.3, 19.5.1 *and* 19.5.2.3)

Sl No.	Inspection and Access	Consequence of Failure	
		Failsafe	Non-failsafe
(1)	(2)	(3)	(4)
i)	Periodic inspection, maintenance and accessibility to detail is good	1.00	1.25
ii)	Periodic inspection, maintenance and accessibility to detail is poor	1.15	1.35

19.3 Detail Category

Table 26(A) to Table 26 (D) indicate the classification of different details into various categories for the purpose of assessing fatigue strength. Details not classified in the table may be treated as the lowest detail category of a similar detail, unless superior fatigue strength is proved by testing and/or analysis.

Holes in members and connections subjected to fatigue loading shall not be made:

a) using punching in plates having thickness greater than 12 mm unless the holes are sub-punched and subsequently reamed to remove the affected material around the punched hole; and

b) using gas cutting unless the holes are reamed to remove the material in the heat affected zone.

19.4 Fatigue Strength

The fatigue strength of the standard detail for the normal or shear fatigue stress range, not corrected for thickness effects discussed in 19.2.1, is given below (*see* also Fig. 22 and Fig. 23):

a) Normal stress range:

1) when $N_{sc} \leq 5\times10^6$

$$f_f = f_{fn}\sqrt[3]{5\times10^6/N_{sc}}$$

2) when $5\times10^6 \leq N_{sc} \leq 10^8$

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$$f_f = f_{fn} \sqrt[5]{5 \times 10^6 / N_{sc}}$$

b) Shear stress:

$$\tau_f = \tau_{fn} \sqrt[5]{5 \times 10^6 / N_{sc}}$$

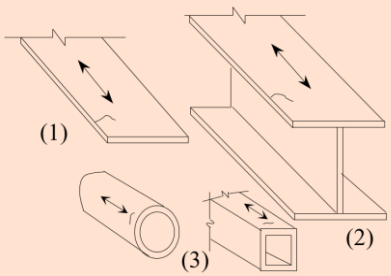
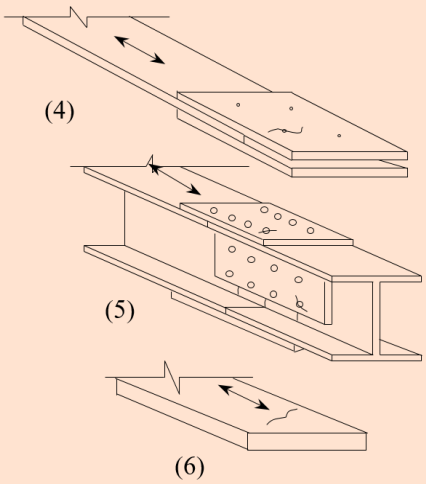
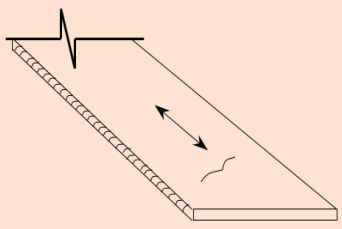
where

f_f, τ_f = design normal and shear fatigue stress range of the detail, respectively, for life cycle of N_{SC}; and

f_{fn}, τ_{fn} = normal and shear fatigue strength of the detail for 5×10^6 cycles, for the detail category (*see* Table 26).

Table 26(A) Detail Category Classification, Group 1 Non-Welded Details (f_{fn})

(*Clauses* 19.2.2.1, 19.3 *and* 19.4)

Sl No.	Detail Category	Constructional Details	
		Illustration (*see* Note)	Description
(1)	(2)	(3)	(4)
i)	118		Rolled and extruded products: a) Plates and flats (1); b) Rolled sections (2); and c) Seamless tubes (3) Sharp edges, surface and rolling flaws to be removed by grinding in the direction of applied stress
ii)	103		a) Bolted connections: (4) and (5) — Stress range calculated on the gross section and on the net section Unsupported one-sided cover plate connections shall be avoided or the effect of the eccentricity taken into account in calculating stresses b) Material with gas-cut or sheared edges with no draglines: (6) — All hardened material and visible signs of edge discontinuities to be removed by machining or grinding in the direction of applied stress
iii)	92		c) Material with machine gas-cut edges with draglines or manual gas-cut material: (7) — Corners and visible signs of edge discontinuities to be removed by grinding in the direction of the applied stress

NOTE — The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

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Table 26(B) Detail Category Classification, Group 2 Welded Details - Not in Hollow Sections (f_{fn})

(*Clauses* 19.2.2.1, 19.3 *and* 19.4)

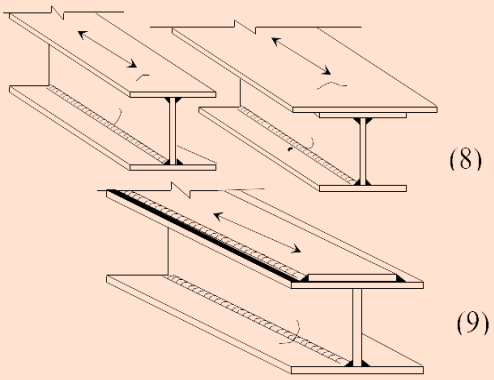
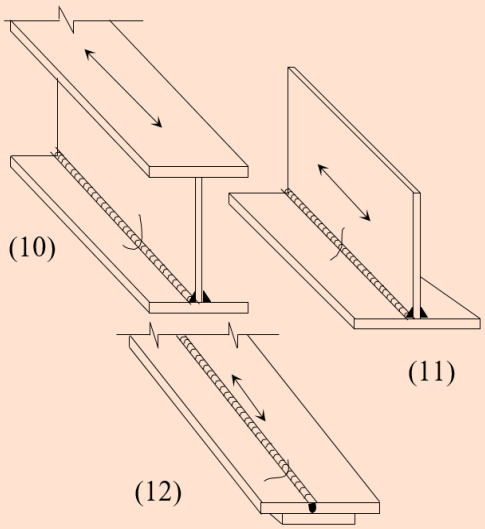
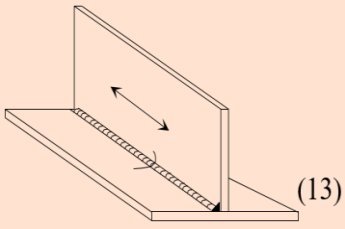
Sl No.	Detail Category	Constructional Details	
		Illustration (*see* Note)	Description
(1)	(2)	(3)	(4)
i)	92		Welded plate I-section and box girders with continuous longitudinal welds: (8) and (9) — Zones of continuous automatic longitudinal fillet or butt welds carried out from both sides and all welds not having un-repaired stop-start positions
ii)	83		Welded plate I-section and box girders with continuous longitudinal welds: (10) and (11) — Zones of continuous automatic butt welds made from one side only with a continuous backing bar and all welds not having un-repaired stop-start positions (12) — Zones of continuous longitudinal fillet or butt welds carried out from both sides but containing stop-start positions. For continuous manual longitudinal fillet or butt welds carried out from both sides, use Detail Category 92
iii)	66		Welded plate I-section and box girders with continuous longitudinal welds: (13) — Zones of continuous longitudinal welds carried out from one side only, with or without stop-start positions

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Table 26(B) (*Continued*)

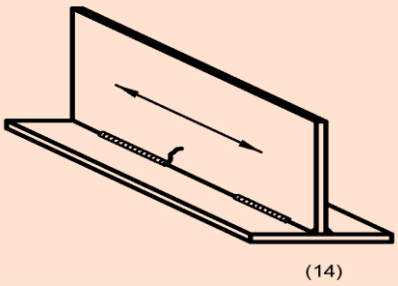
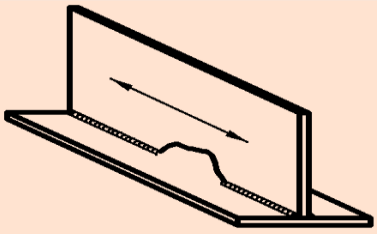
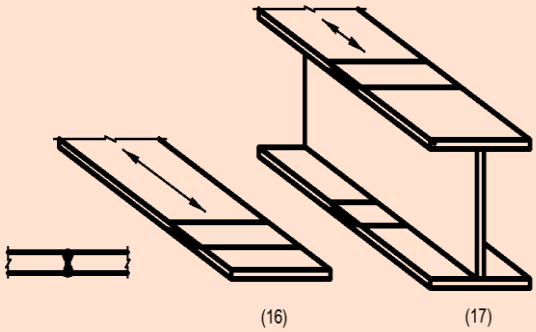
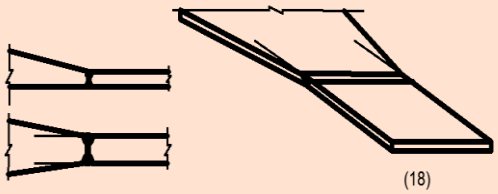
Sl No.	Detail Category	Constructional Details	
		Illustration (*see* Note)	Description
(1)	(2)	(3)	(4)
iv)	59	 (14)	Intermittent longitudinal welds: (14) — Zones of intermittent longitudinal welds
v)	52	 (15)	Intermittent longitudinal welds: (15) — Zones containing cope holes in longitudinally welded T-joints. Cope hole not to be filled with weld
vi)	83	 (16) (17)  (18)	Transverse butt welds (complete penetration): Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides (16) — Transverse splices in plates, flats and rolled sections having the weld reinforcement ground flush to plate surface. 100 percent NDT inspection, and weld surface to be free of exposed porosity in the weld metal (17) — Plate girders welded as in (16) before assembly (18) — Transverse splices as in (16) with reduced or tapered transition with taper ≤ 1 : 4

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Table 26(B) (*Continued*)

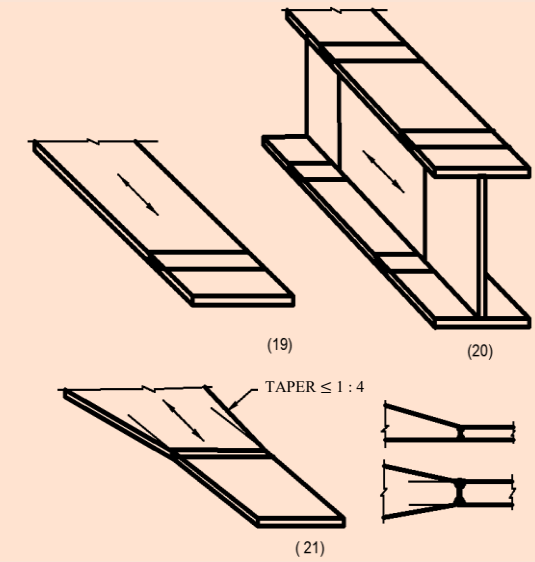
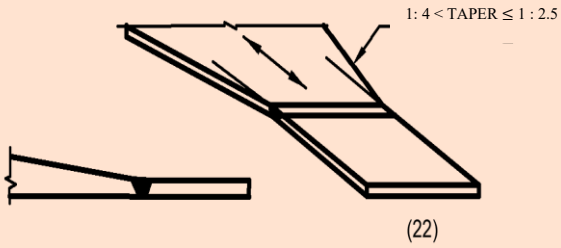
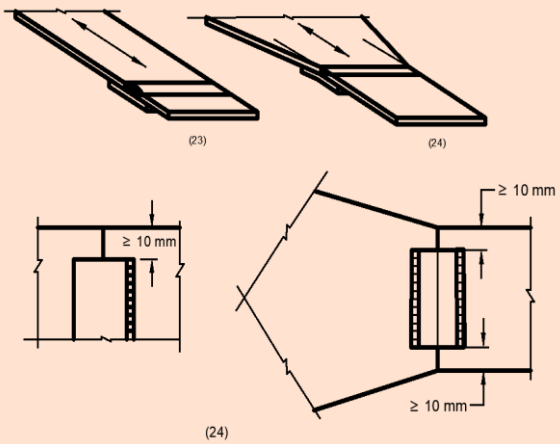
Sl No.	Detail Category	Constructional Details	
		Illustration (*see* Note)	Description
(1)	(2)	(3)	(4)
vii)	66		Transverse butt welds (complete penetration): Welds run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides (19) — Transverse splices of plates, rolled sections or plate girders (20) — Transverse splice of rolled sections or welded plate girders, without cope hole. With cope hole use Detail Category 52, as in (15) (21) — Transverse splices in plates or flats being tapered in width or in thickness where the taper is ≤ 1 : 4
viii)	59		Transverse butt welds (complete penetration): Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides (22) — Transverse splices as in (21) with taper in width or thickness >1 : 4 but ≤1 : 2.5
ix)	52		Transverse butt welds (complete penetration): (23) — Transverse butt-welded splices made on a backing bar. The end of the fillet weld of the backing strip shall stop short by more than 10 mm from the edges of the stressed plate (24) — Transverse butt welds as per (23) with taper on width or thickness < 1 : 2.5

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Table 26(B) (*Continued*)

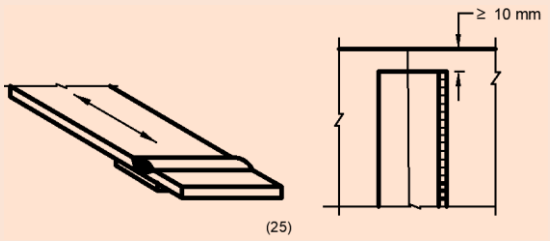
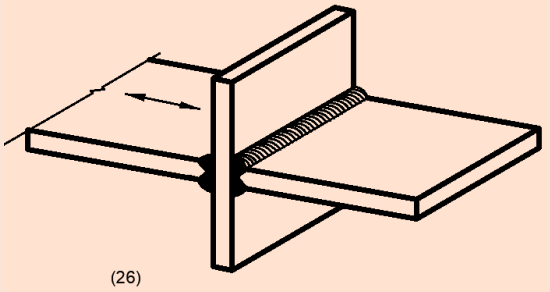
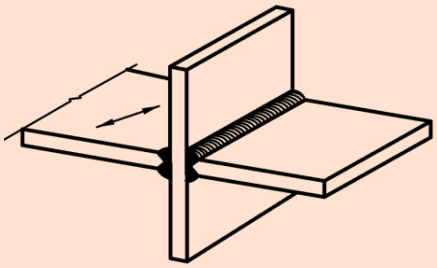
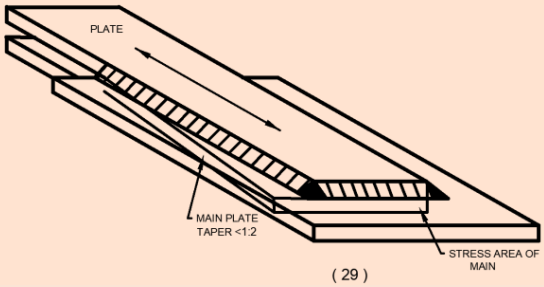
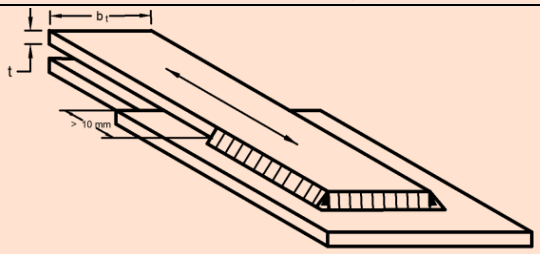
Sl No.	Detail Category		Constructional Details	
			Illustration (*see* Note)	Description
(1)	(2)		(3)	(4)
x)	37		 (25)	Transverse butt welds (complete penetration): (25) — Transverse butt welds as in (23) where fillet welds end closer than 10 mm to plate edge
xi)	52		 (26)	Cruciform joints with load-carrying welds: (26) — Full penetration welds with intermediate plate NDT inspected and free of defects. Maximum misalignment of plates either side of joint to be < 0.15 times the thickness of intermediate plate
xii)	41	(27)	 (27) and (28)	(27) — Partial penetration or fillet welds with stress range calculated on plate area
	27	(28)		(28) — Partial penetration or fillet welds with stress range calculated on throat area of weld
xiii)	46		 (29)	Overlapped welded joints: (29) — Fillet welded lap joint, with welds and overlapping elements having a design capacity greater than the main plate. Stress in the main plate to be calculated on the basis of area shown in the illustration
xiv)	41	(30)	 (30) and (31)	Overlapped weld joints: (30) — Fillet welded lap joint, with welds and main plate both having a design capacity greater than the overlapping elements
xv)	33	31		31 — Fillet welded lap joint, with main plate and overlapping elements both having a design capacity greater than the weld

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Table 26(B) (*Continued*)

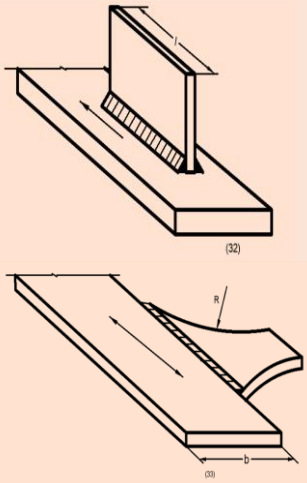
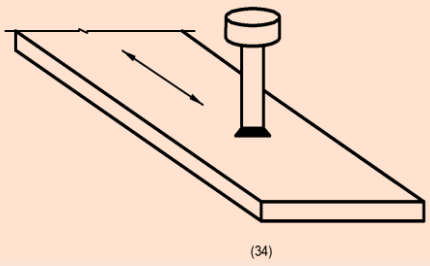
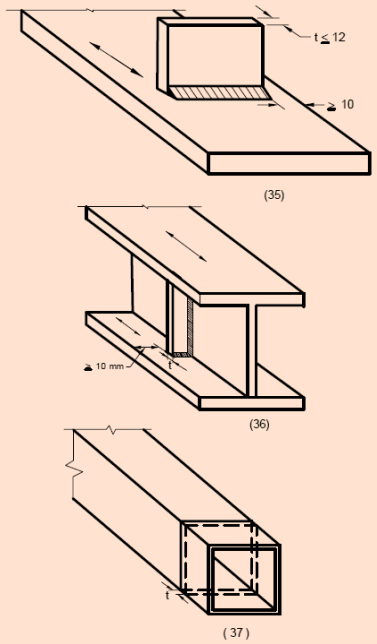
Sl No.	Detail Category	Constructional Details			
		Illustration (*see* Note)			Description
(1)	(2)	(3)			(4)
xvi)	66	(32)	(33)		Welded attachments (non-load carrying welds) — Longitudinal welds: (32) — Longitudinal fillet welds. Class of detail varies according to the length of the attachment weld as noted (33) — Gusset welded to the edge of a plate or beam flange. Smooth transition radius (r), formed by machining or flame cutting plus grinding. Class of detail varies according to r/b ratio as noted
		—	$1/3 \leq r/b$		
	59	$l \leq 50$mm	—		
	52	$50 < l \leq 100$ mm	$1/6 \leq r/b < 1/3$		
	37	100 mm $< l$	—		
	33	—	$r/b < 1/6$		
xvii)	59				Welded attachments: (34) — Shear connectors on base material (failure in base material)
xviii)	59	$t \leq 12$ mm			Transverse welds: (35) — Transverse fillet welds with the end of the weld ≥10 mm from the edge of the plate (36) — Vertical stiffeners welded to a beam or plate girder flange or web by continuous or intermittent welds. In the case of webs carrying combined bending and shear design actions, the fatigue strength shall be determined using the stress range of the principal stresses (37) — Diaphragms of box girders welded to the flange or web by continuous or intermittent welds
	52	$t > 12$ mm			

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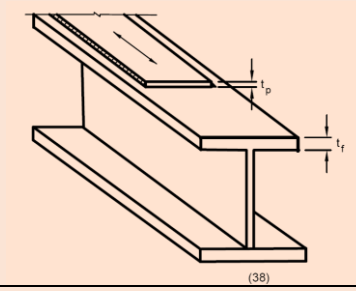
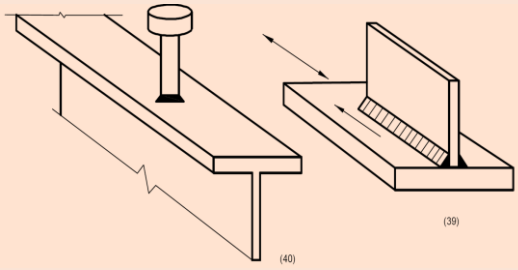
Table 26(B) (*Continued*)

Sl No.	Detail Category	Constructional Details		
		Illustration (*see* Note)		Description
(1)	(2)	(3)		(4)
xix)	37	t_f or $t_p \leq$ 25mm		Cover plates in beams and plate girders: (38) — End zones of single or multiple welded cover plates, with or without a weld across the end. For a reinforcing plate wider than the flange, an end weld is essential
	27	t_f or t_p >25 mm		
xx)	67			Welds loaded in shear: (39) — Fillet welds transmitting shear. Stress range to be calculated on weld throat area (40) — Stud welded shear connectors (failure in the weld) loaded in shear (the shear stress range to be calculated on the nominal section of the stud)

NOTE — The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

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Table 26(C) Detail Category Classification, Group 3 Bolts

(*Clauses* 19.2.2.1, 19.3 *and* 19.4)

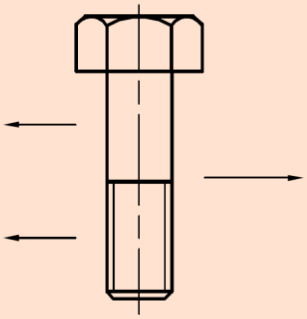
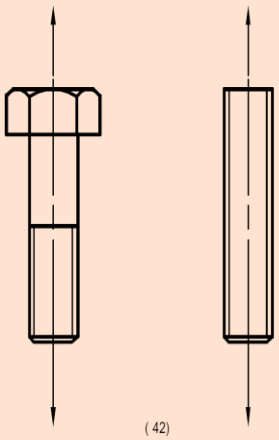
Sl No.	Detail Category	Constructional Details	
		Illustration (*see* Note)	Description
(1)	(2)	(3)	(4)
i)	83	 (41)	Bolts in shear (8.8/TB Bolting category only): (41) — Shear stress range calculated on the minor diameter area of the bolt (A_c) NOTE — If the shear on the joint is insufficient to cause slip of the joint the shear in the bolt need not be considered in fatigue
ii)	27	 (42)	Bolts and threaded rods in tension (tensile stress to be calculated on the tensile stress area, A_t): (42) — Additional forces due to prying effects shall be taken into account. For tensional bolts, the stress range depends on the connection geometry NOTE — In connections with tensioned bolts, the change in the force in the bolts is often less than the applied force, but this effect is dependent on the geometry of the connection. It is not normally required that any allowance for fatigue be made in calculating the required number of bolts in such connections
NOTE — The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.			

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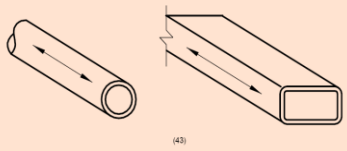
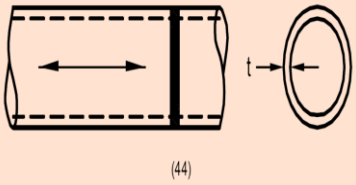
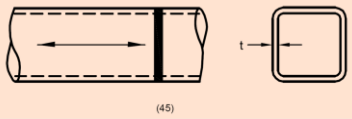
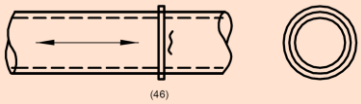
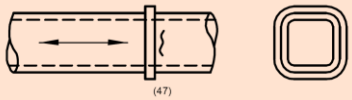
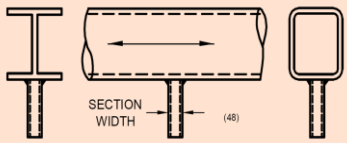
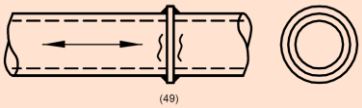
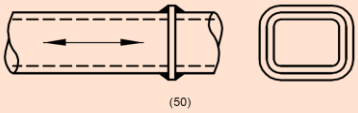
Table 26(D) Detail Category Classification, Group 4 Welded details — in Hollow Sections (f_{fn})

(*Clauses* 19.2.2.1, 19.3 *and* 19.4)

Sl No.	Detail Category	Constructional Details	
		Illustration (*see* Note)	Description
(1)	(2)	(3)	(4)
i)	103		Continuous automatic longitudinal welds: (43) — No stop-starts, or as manufactured, proven free to detachable discontinuities.
ii)	66 ($t \geq 8$ mm) 52 ($t < 8$ mm)		Transverse butt welds: (44) — Butt-welded end-to-end connection of circular hollow sections NOTE — Height of the weld reinforcement less than 10 percent of weld with smooth transition to the plate surface. Welds made in flat position and proven free to detachable discontinuities)
iii)	52 ($t \geq 8$ mm) 41 ($t < 8$ mm)		(45) — Butt-welded end-to-end connection of rectangular hollow sections
iv)	41 ($t \geq 8$ mm) 37 ($t < 8$ mm)		Butt welds to intermediate plate: (46) — Circular hollow sections, end-to-end butt-welded with an intermediate plate
v)	37 ($t \geq 8$ mm) 30 ($t < 8$ mm)		(47) — Rectangular hollow sections, end-to-end butt welded with an intermediate plate
vi)	52		Welded attachments (non-load-carrying): (48) — Circular or rectangular hollow section, fillet welded to another section. Section width parallel to stress direction ≤100mm
vii)	33 ($t < 8$ mm) 29 ($t < 8$ mm)		Fillet welds to intermediate plate: (49) — Circular hollow sections, end-to-end fillet welded with an intermediate plate
viii)	29 ($t \geq 8$ mm) 27 ($t < 8$ mm)		(50) — Rectangular hollow sections, end-to-end fillet welded with an intermediate plate

NOTE — The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

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19.5 Fatigue Assessment

19.5.1 The design fatigue strength for N_{SC} life cycles (f_{fd}, τ_{fd}) may be obtained from the standard fatigue strength for N_{SC} cycles by multiplying with correction factor, μ_r, for thickness, as mentioned in **19.2.1** and dividing by partial safety factor given in Table 25.

19.5.2 *Stress Limitations*

19.5.2.1 The maximum (absolute) value of the normal and shear stresses shall never exceed the elastic limit (f_y, τ_y) for the material under cyclic loading.

19.5.2.2 The maximum stress range shall not exceed 1.5 f_y for normal stresses and 1.5 $f_y/\sqrt{3}$ for the shear stresses under any circumstance.

19.5.2.3 *Constant stress range*

The actual normal and shear stress range f and τ at a point of the structure subjected to N_{SC} cycles in life shall satisfy:

$$f \le f_{fd} = \mu_r f_f / \gamma_{mft}$$

$$\tau \le \tau_{fd} = \mu_r \tau_f / \gamma_{mft}$$

where

f_f, τ_f = normal and shear fatigue strength ranges for the actual life cycle, N_{SC}, obtained from **19.4**;

μ_r = correction factor (*see* **19.2.1**); and

γ_{mf} = partial safety factor against fatigue failure, given in Table 25.

19.5.2.4 *Variable stress range*

Fatigue assessment at any point in a structure, wherein variable stress ranges f_i or τ_i for n_i number of cycles (i =1 to r) are encountered, shall satisfy the following:

a) For normal stress (f):

$$\frac{\sum_{i=1}^{r_5} n_i f_i^3}{5\times10^6(\mu_r f_{fn}/\gamma_{mft})^3} + \frac{\sum_{j=r_5}^{r} n_j f_j^5}{5\times10^6(\mu_r f_{fn}/\gamma_{mft})^5} \le 1.0$$

b) For shear stresses (τ):

$$\sum_{i=0}^{r} n_i \tau_i^5 \le 5\times10^6(\mu_r \tau_{fn}/\gamma_{mft})$$

Where r_5 is the summation upper limit of all the normal stress ranges (f_i) having magnitude lesser than ($\mu_r f_{fn}/\gamma_{mft}$) for that detail and the lower limit of all the normal stress ranges (f_j) having magnitude greater than ($\mu_r f_{fn}/\gamma_{mft}$) for the detail. In the above summation all normal stress ranges, f_i, and shear stress ranges τ_i having magnitude less than $0.55\mu_r f_{fn}$, and $0.55\mu_r \tau_{fn}$, respectively may be disregarded.

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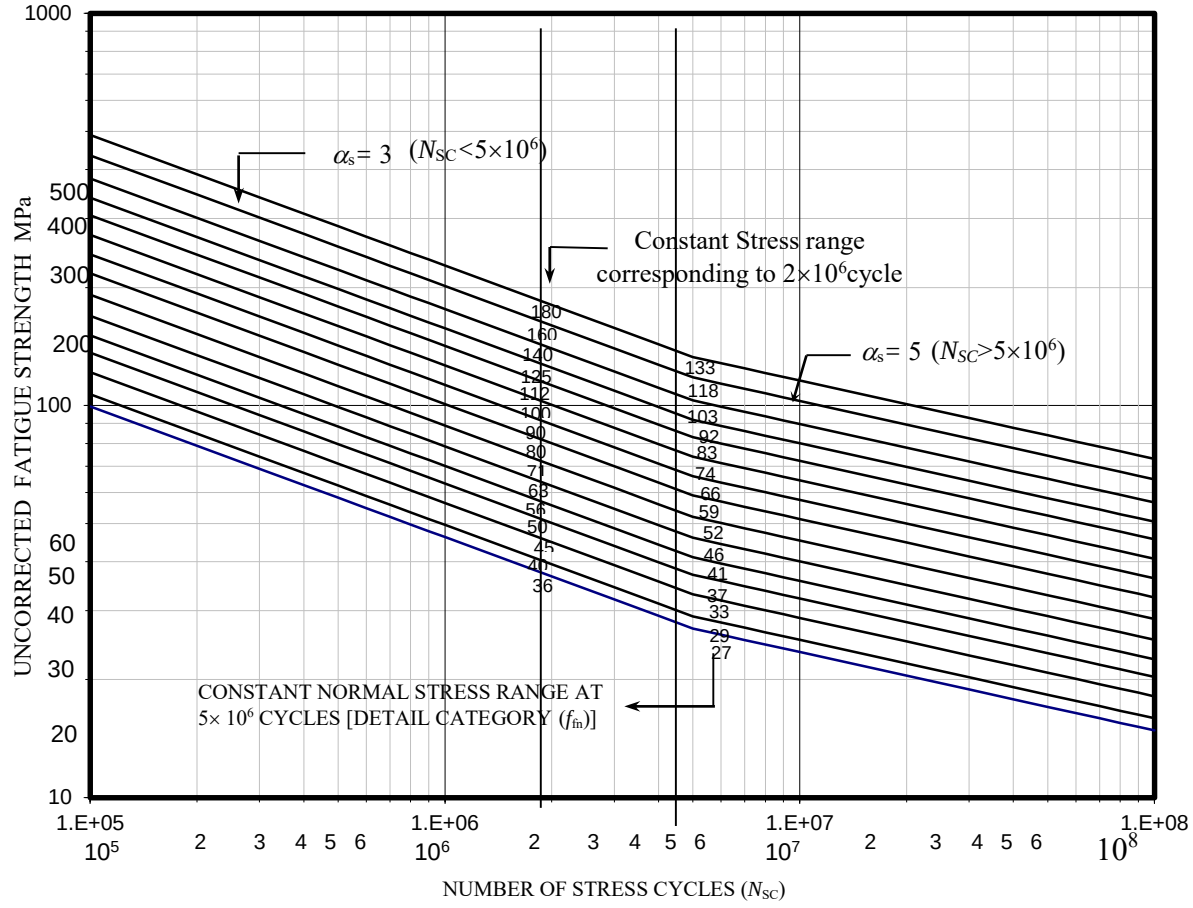

FIG. 22 S-N CURVE FOR NORMAL STRESS

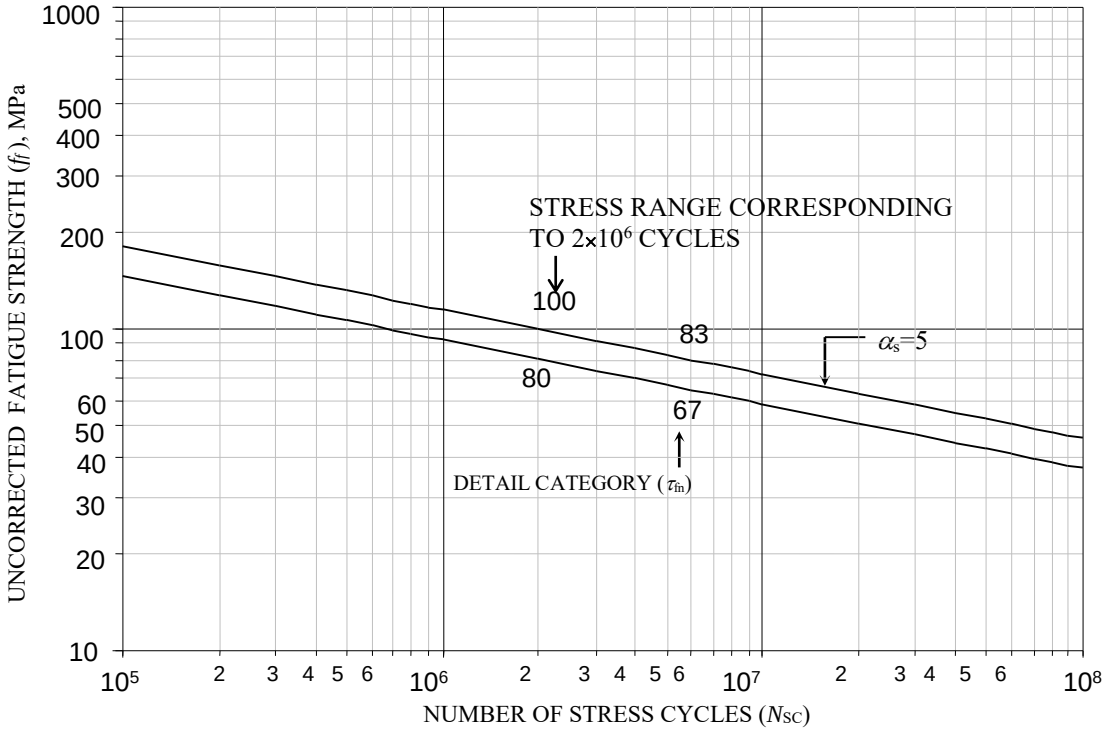

FIG. 23 S-N CURVE FOR SHEAR STRESS

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20 DESIGN ASSISTED BY TESTING

20.1 Need for Testing

Testing of structures, members or components of structures is not required when designed in accordance with this standard. Testing may be accepted as an alternative to calculations or may become necessary in special circumstances.

Testing of a structural system, member or component may be required to assist the design in the following cases:

a) when the calculation methods available are not adequate for the design of a particular structure, member or component, testing shall be undertaken in place of design by calculation or to supplement the design by calculation;

b) where rules or methods for design by calculation would lead to uneconomical design, experimental verification may be undertaken to avoid conservative design;

c) when the design or construction is not entirely in accordance with sections of this standard, experimental verification is recommended;

d) when confirmation is required on the consistency of production of material, components, members or structures originally designed by calculations or testing; and

e) when the actual performance of an existing structure capacity is in question, testing shall be used to confirm it.

20.1.1 Testing of structural system, member or component shall be of the following categories:

a) *Proof testing* — The application of test loads to a structure, sub-structure, member or connection to ascertain the structural characteristics of only that specific unit; and

b) *Prototype testing* — Testing of structures, sub-structures, members or connections is done to ascertain the structural characteristics of a class of such structures, sub-structures members or connection, which are nominally identical to the units tested.

20.2 Types of Test

20.2.1 *Acceptance Test*

This is intended as a non-destructive test for confirming structural performance. It should be recognized that the loading applied to certain structures might cause permanent distortions. Such effects do not necessarily indicate structural failure in acceptance test. However, the possibility of their occurrence should be agreed to before testing.

The load for acceptance test, $F_{test,a}$ shall be determined from:

$$F_{test,a} = (\text{Self weight}) + (1.15 \times \text{Remainder of the permanent load}) + (1.25 \times \text{Variable load})$$

The assembly shall satisfy the following criteria:

a) it shall demonstrate substantially linear behaviour under test loading; and

b) on removal of the test load, the residual deflection shall not exceed 20 percent of the maximum-recorded deflection.

If the above criteria are not satisfied the test may be repeated one more time only, when the assembly shall satisfy the following criteria:

a) it shall demonstrate substantially linear behaviour on the second application of test loading; and

b) corresponding recorded residual deflection in the second test shall not exceed 10 percent of the maximum deflection during the test.

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20.2.2 *Strength Test*

Strength test is used to confirm the calculated resistance of a structure or component. Where a number of items are to be constructed to a common design, and one or more prototypes are tested to confirm their strength, the others may be accepted without any additional test, provided they are similar in all relevant respects to the prototype.

Before carrying out the strength test, the specimen should first be subjected to and satisfy the acceptance test. Since the resistance of the assembly under test depends on the material properties, the actual yield strength of all the steel materials in the assembly shall be determined from coupon {test piece as defined in the accepted standard [C-6A(7)]} tests. The mean value of the yield strength, $f_{ym,}$ taken from such tests shall be determined with due regard to the importance of each element in the assembly. The strength test load $F_{test,\,s}$ (including self weight) shall be determined from:

$$F_{test,s} = \gamma_{mi} F_d \left(f_{ym} / f_y \right)$$

where

F_d = factored design load for the ultimate limit state;

f_y = characteristic yield stress of the material as assumed in the design; and

γ_{mi} = partial safety factor for the type of failure, as prescribed in this standard.

At this load there shall be no failure by buckling or rupture of any part of the structure or component tested. On removal of the test load, the deflection should decrease by at least 20 percent of the maximum deflection at $F_{test,s}$.

20.2.3 *Test to Failure* (*Ultimate Strength Test*)

The objective of a test to failure is to determine the design resistance from the ultimate resistance. In this situation it is still desirable to carry out the acceptance and strength tests, before test to failure.

Not less than three tests shall be carried out on nominally identical specimens. An estimate should be made of the anticipated ultimate resistance as a basis for such tests. During a test to failure, the loading shall first be applied in increments up to the strength test load. Subsequent load increments shall then be determined from consideration of the principal load deflection plot. The test load resistance, $F_{test.R}$ shall be determined as that load at which the specimen is unable to sustain any further increase in load. At this load, gross permanent distortion is likely to have occurred and in some cases such large gross deformation may define the test limit. If the deviation of any individual test result exceeds 10 percent of the mean value obtained for all the three tests, at least three more tests shall be carried out. When the deviation from the mean does not exceed 10 percent of the mean, the design resistance may be evaluated as given below:

a) When the failure is ductile, the design resistance, F_d may be determined from:

$$F_d = 0.9 F_{test,Min} (f_y / f_{ym}) / \gamma_{m0}$$

where

$F_{test,\,Min}$ = minimum test result from the tests to failure;

f_{ym} = average yield strength as obtained from the material tests; and

f_y = characteristic yield stress of the grade of steel.

In the case of a sudden (brittle) rupture type failure, the design resistance may be determined from:

$$F_d = 0.9 F_{test,Min} (f_u / f_{um}) / \gamma_{m1}$$

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where

f_u = characteristic ultimate stress of the grade of steel used; and

f_{um} = average ultimate tensile strength of the material obtained from tests.

b) In the case of a sudden (brittle) buckling type failure, the design resistance shall be determined from:

$$F_d = 0.75 F_{test,Min} (f_y / f_{ym}) / \gamma_{m0}$$

c) In ductile buckling type failure in which the relevant slenderness λ can be reliably assessed, the design resistance may be determined from:

$$F_d = 0.9 F_{test,Min} \left[(\chi f_y / \chi_m f_{ym}) / \gamma_{m0} \right]$$

where

χ = reduction factor for the relevant buckling curve; and

χ_m = value of χ when the yield strength is f_{ym}.

20.2.4 *Check Tests*

Where a component or assembly is designed on the basis of strength tests or tests to failure and a production run is carried out of such items, an appropriate number of samples (not less than two) shall be selected from each production batch at random for check tests.

20.2.4.1 The samples shall be carefully examined to ensure that they are similar in all respects to the prototype tested, particular attention being given to the following:

a) Dimensions of components and connections;

b) Tolerance and workmanship; and

c) Quality of steel used, checked with reference to mill test certificates.

20.2.4.2 Where it is not possible to determine either the variations or the effect of variations from the prototype, an acceptance test shall be carried out as a check test.

20.2.4.3 In this check test, the deflections shall be measured at the same positions as in the acceptance test of the prototype. The maximum measured deflection shall not exceed 120 percent of the deflection recorded during the acceptance test on the prototype and the residual deflection should not be more than 105 percent of that recorded for the prototype.

20.3 Test Conditions

Followings are the conditions for the test:

a) loading and measuring devices shall be calibrated in advance;

b) the design of the test rig shall be such that:

1) loading system adequately simulates the magnitude and distribution of the loading;

2) it allows the specimen to perform in a manner representative of service conditions;

3) lateral and torsional restraint, if any, should be representative of those in service;

4) specimen should be free to deflect under load according to service condition;

5) loading system shall be able to follow the movements of the specimen without interruption or abnormal restraints; and

6) inadvertent eccentricities at the point of application of the test loads and at the supports are avoided.

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c) test load shall be applied to the unit at a rate as uniform as practicable;

d) deflections should be measured at sufficient points of high movements to ensure that the maximum value is determined;

e) if the magnitude of stresses in a specimen is to be determined, the strain at the desired location may be measured and the corresponding stress calculated; and

f) prior to any test, preliminary loading (not exceeding the characteristic values of the relevant loads) may be applied and then removed, in order to set the test specimen on to the test rig.

20.4 Test Loading

20.4.1 Where the self-weight of the specimen is not representative of the actual permanent load in service, allowance for the difference shall be made in the calculation of test loads to be applied.

20.4.2 On the attainment of maximum load for either acceptance or strength tests, this load shall be maintained as given below. Reading of load and deflection shall be taken at intervals of 15 min and the loading shall be maintained constant until there is no significant increase in deflection during a 15 min period or until at least 1 h has elapsed after application of maximum load.

20.4.3 The test load shall be equal to the design load for the relevant limit state in proof testing.

20.4.4 The test load in prototype testing shall be equal to the design load for the relevant limit state as multiplied by the appropriate factor given in Table 27.

20.5 Criteria for Acceptance

20.5.1 *Acceptance for Strength*

The test structure, sub-structure, member or connection shall be deemed to comply with the requirements for strength if it is able to sustain the strength test load as required in **20.4.2**.

It shall then be inspected to determine the nature and extent of any damage incurred during the test. The effects of the damage shall be considered and if necessary appropriate repairs to the damaged parts carried out.

20.5.2 *Acceptance for Serviceability*

The maximum deformation of the structure or member under the serviceability limit state test load shall be within the serviceability limit values appropriate to the structure.

Table 27 Factors to Allow for Variability of Structural Units

(*Clause* 20.4.4)

Sl No.	No. of Similar Units to be Tested	For Strength Limit State	For Serviceability Limit State
(1)	(2)	(3)	(4)
i)	1	1.5	1.2
ii)	2	1.4	1.2
iii)	3	1.3	1.2
iv)	4	1.3	1.1
v)	5	1.3	1.1
vi)	10	1.2	1.1

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21 DURABILITY

21.1 General

A durable steel structure is one that performs satisfactorily the desired function in the working environment under the anticipated exposure condition during its service life, without deterioration of the cross sectional area and loss of strength due to corrosion.

The material used, the detailing, fabrication, erection and surface protection measures should all address the corrosion protection and durability requirements.

21.2 Requirements for Durability

21.2.1 *Shape, Size, Orientation of Members, Connections and Details*

The design, fabrication and erection details of exposed structures should be such that good drainage of water is ensured. Standing pool of water, moisture accumulation and rundown of water for extended duration shall be avoided.

The details of connections should ensure that:

a) all exposed surfaces are easily accessible for inspection and maintenance; and

b) all surfaces, not so easily accessible are completely sealed against ingress of moisture.

21.2.2 *Exposure Condition*

21.2.2.1 *General environment*

The general environment, to which a steel structure is exposed during its working life is classified into five levels of severity, as given in Table 28.

21.2.2.2 *Abrasion*

Specialist literature may be referred for durability of surfaces exposed to abrasive action as in machinery, conveyor belt support system, storage bins for grains or aggregates.

21.2.2.3 *Exposure to sulphate attack*

Appropriate coatings may be used when surfaces of structural steel are exposed to concentration of sulphates (SO_3) in soil, ground water, etc.

When exposed to very high sulphate concentrations of more than 2 percent in soil and 5 percent in water, some form of lining such as polyethylene, polychloroprene sheet or surface coating based on asphalt, chlorinated rubber, epoxy or polymethane material should be used to completely avoid access of the solution to the steel surface.

21.2.3 *Corrosion Protection Methods*

The methods of corrosion protection are governed by actual environmental conditions as specified in good practices [C-6A(43)]. The main corrosion protection methods are given below:

a) Controlling the electrode potential;

b) Inhibitors; and

c) Inorganic/metal coatings or organic/paint systems.

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Table 28 Environmental Exposure Conditions

(*Clause* 21.2.2.1)

Sl No.	Environmental Classifications	Exposure Conditions
(1)	(2)	(3)
i)	Mild	Surfaces normally protected against exposure to weather or aggressive condition as in interior of buildings, except when located in coastal areas
ii)	Moderate	Structural steel surfaces: a) exposed to condensation and rain; b) continuously under water; c) exposed to non-aggressive soil/groundwater; and d) sheltered from saturated salt air in coastal areas
iii)	Severe	Structural steel surfaces: a) exposed to severe frequent rain; b) exposed to alternate wetting and drying; c) severe condensation; d) completely immersed in sea water; and e) exposed to saturated salt air in coastal area
iv)	Very severe	Structural steel surface exposed to: a) sea water spray; b) corrosive fumes; and c) aggressive sub-soil or ground water.
v)	Extreme	Structural steel surfaces exposed to: a) tidal zones and splash zones in the sea; and b) aggressive liquid or solid chemicals

21.2.4 *Surface Protection*

21.2.4.1 In the case of mild exposure, a coat of primer after removal of any loose mill scale may be adequate. As the exposure condition becomes more critical, more elaborate surface preparations and coatings become necessary. In case of extreme environmental classification, protection shall be as per specialist literature. Table 29 gives guidance to protection of steelwork for different desired lives.

21.2.4.2 Steel surfaces shall be provided with at least one coat of primer immediately after its surface preparation, such as by sand blasting to remove all mill scale and rust and to expose the steel.

21.2.4.3 Steel without protective coating shall not be stored for long duration in outdoor environment.

21.2.4.4 Surfaces to transfer forces by friction as in HSFG connections shall not be painted. However, it shall be ensured that moisture is not trapped on such surfaces after pre-tensioning of bolts by proper protective measures.

21.2.4.5 Members to be assembled by welding shall not be pre-painted at regions adjacent to the location of such welds. However, after welding, appropriate protective coatings shall be applied in the region, as required by the exposure conditions. If the contact surfaces cannot be properly protected against ingress of moisture by surface coating, they may be completely sealed by appropriate welds.

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21.2.4.6 Pre-painted members shall be protected against abrasion of the coating during transportation, handling and erection.

21.2.5 *Special Steels*

Steels with special alloying elements and production process to obtain better corrosion resistance may be used as per specialist literature.

Table 29(A) Protection Guide for Steel Work Application — Desired Life of Coating System in Different Environment

(*Clause* 21.2.4.1)

Sl No.	Atmospheric Condition/Environmental Classification	Coating System					
		1	2	3	4	5	6
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Normal inland (rural and urban areas), mild	12 years	18 years	20 years	About 20 years	About 20 years	Above 20 years
ii)	Polluted inland (high airborne sulphur dioxide), moderate	10 years	15 years	12 years	About 18 years	15 to 20 years	Above 20 years
iii)	Normal coastal (as normal inland plus high airborne salt levels), severe	10 years	12 years	20 years	About 20 years	About 20 years	Above 20 years
iv)	Polluted coastal (as polluted inland plus high airborne salt levels), very severe or extreme	8 years	10 years	10 years	About 15 years	15 to 20 years	Above 20 years

Table 29(B) (i) Protection Guide for Steel Work Application — Specification for Different Coating System (Shop Applied Treatments)

(*Clause* 21.2.4.1)

Sl No.	Protection	Coating System					
		1	2	3	4	5	6
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Surface preparation	Blast clean	Blast clean	Blast clean	Blast clean	Girt blast	Blast clean
ii)	Pre-fabrication primer	Zinc phosphate epoxy, 20 µm	2 pack zinc-rich epoxy, 20 µm	—	2 pack zinc-rich epoxy, 20 µm	—	Ethyl zinc silicate, 20 µm
iii)	Post fabrication primer	High-build zinc phosphate modified alkyd, 60 µm	2 pack zinc-rich epoxy, 20 µm	Hot dip galvanized, 85 µm	2 pack zinc-rich epoxy, 25 µm	Sprayed zinc or sprayed aluminium	Ethyl zinc silicate, 60 µm

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Table 29(B) (i) (*Concluded*)

Sl No.	Protection	Coating System					
		1	2	3	4	5	6
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
iv)	Intermediate coat	—	High-build zinc phosphate, 25 µm	—	2 pack epoxy micaceous iron oxide	Sealer	Chlorinated rubber alkyd, 35 µm
v)	Top coat	—	—	—	2 pack epoxy micaceous iron oxide, 85 µm	Sealer	—

Table 29(B) (ii) Protection Guide for Steel Work Application — Specification for Different Coating System (Site Applied Treatments)

(*Clause* 21.2.4.1)

Sl No.	Protection	Coating System					
		1	2	3	4	5	6
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Surface preparation	As necessary	As necessary	No site treatment	As necessary	No site treatment	As necessary
ii)	Primer	Touch in	Touch in	—	—	—	Touch in
iii)	Intermediate coat	—	Modified Alkydmicaceous iron oxide, 50 µm	—	Touch In	–	High-build micaceous iron oxide chlorinated rubber micaceous, 75 µm
iv)	Top coat	High-build Alkyd finish, 60µm	Modified alkyd micaceous iron oxide, 50 µm	—	High-build chlorinated rubber	—	High-build iron oxide chlorinated rubber, 75 µm

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22 FIRE RESISTANCE

22.1 Requirements

The requirements shall apply to steel building elements designed to exhibit a required fire-resistance level (FRL) as per the relevant specifications. The provisions of this section are quite rudimentary and simplistic. Specialist literature for design of steel structures for fire resistance and design based on testing may be necessary for a more comprehensive design of steel buildings to resist fire.

22.1.1 For protected steel members and connections, the thickness of protection material (h_i) shall be greater than or equal to that required to give a period of structural adequacy (PSA) greater than or equal to the required FRL.

22.1.2 For unprotected steel members and connections, the exposed surface area to mass ratio (k_{sm}) shall be less than or equal to that required to give a PSA equal to the required FRL.

22.2 Fire Resistance Level

The required FRL shall be as prescribed in Part F 'Fire and Life Safety' of this NBCS and relative fire norms/ standards, as appropriate or in building specifications or as required by the user or the city ordinance. The FRL specified in terms of the duration (in min) of standard fire load without collapse depends upon;

a) the purpose for which structure is used; and

b) the time taken to evacuate in case of fire.

22.3 Period of Structural Adequacy (PSA)

22.3.1 The calculation of PSA involves:

a) calculation of the strength of the element as a function of temperature of the element and the determination of limiting temperature;

b) calculation of the thermal response of the element, that is calculation of the variation of the temperature of the element or the parts of the element with time, when exposed to fire; and

c) determination of PSA at which the temperature of the element or parts of the element reaches the limiting temperature.

22.3.2 *Determination of Period of Structural Adequacy*

The period of structural adequacy (PSA) shall be determined using one of the following methods:

a) *By calculation*:

   1) determining the limiting temperature of the steel (T_l) in accordance with 22.5; and

   2) determining the PSA as the time (t) (in min) from the start of the test to the time at which the limiting steel temperature is attained, in accordance with 22.6 for protected members and 22.7 for unprotected members.

b) By direct application of a single test in accordance with 22.8; or

c) By calculation of the temperature of the steel member by using a rational method of analysis confirmed by test data or by methods available in specialist literature.

22.4 Variation of Mechanical Properties of Steel with Temperature

22.4.1 *Variation of Yield Stress with Temperature*

The influence of temperature on the yield stress of steel shall be taken as follows for structures of mild steels and high strength low alloy steels:

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where

$$\frac{f_y(T)}{f_y(20)} = \frac{905 - T}{690} \leq 1.0$$

$f_y(T)$ = yield stress of steel at T °C;

$f_y(20)$ = yield stress of steel at 20 °C (room temperature); and

T = temperature of the steel, in °C.

This relationship is shown by curve 1 in Fig. 24.

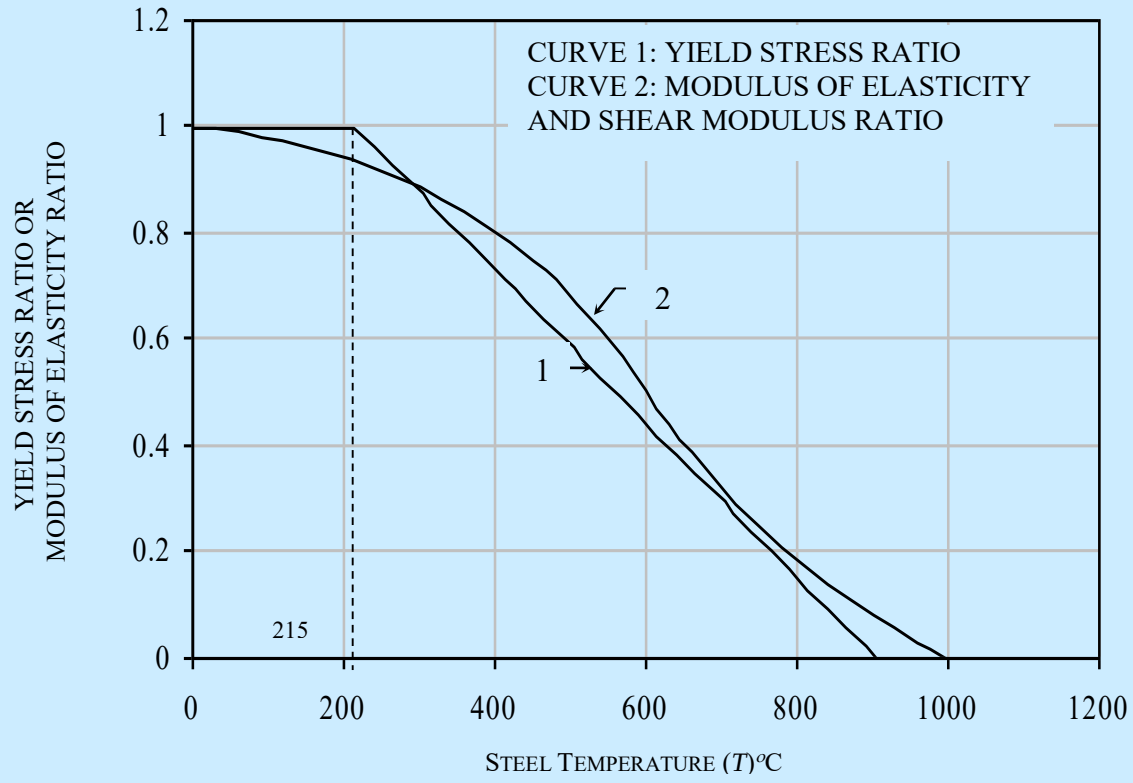

FIG. 24 VARIATION OF MECHANICAL PROPERTIES OF STEEL WITH TEMPERATURE

For temperature less than 215 °C no reduction in the yield stress need to be considered.

22.4.2 *Variation of Modulus of Elasticity and Shear Modulus with Temperature*

The influence of temperature on the modulus of elasticity shall be taken as follows for structures of mild steels and high strength low alloy steels:

$$\frac{E(T)}{E(20)} \text{ and } \frac{G(T)}{G(20)} = 1.0 + \left[\frac{T}{2000\left[ln\left(\frac{T}{1\,100}\right)\right]}\right] \text{ when } 0\,°C < T \leq 600\,°C$$

$$= \frac{690\left(1 - \frac{T}{1000}\right)}{T - 53.5} \quad \text{when} \quad 600\,°C < T \leq 1000\,°C$$

where

$E(T)$ = Modulus of elasticity of steel at T °C, and

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E (20) = Modulus of elasticity of steel at 20 °C (room temperature).

G (T) = Shear Modulus of steel at T °C, and

G (20) = Shear Modulus of steel at 20 °C (room temperature).

This relationship is shown by curve 2 in Fig. 24.

22.4.3 For special steel with higher temperature resistance, such as TMCP steels, the manufacturer's recommendation shall be used to obtain the variation of yield strength and modulus of elasticity of steel with temperature.

22.5 Limiting Steel Temperature

The limiting steel temperature (T_l), in °C, in the case of structural steels, shall be calculated by substituting f_y, f_u, E, and G values with $f_y(T)$, $f_y(T)$, $E(T)$, and $G(T)$ calculated in accordance with **22.4**, respectively in **13**, **14**, **15**, and **16** to calculate the design capacity of the member as a function of steel temperature. The limiting temperature (T_l) will be the temperature at which the design capacity is equal to the design action on the member.

The design action under fire shall consider the following:

a) Loads as per the load combinations for fire conditions given in Table 4;

b) Effects of restraint to expansion of the elements during fire; and

c) Reduced bond likely under fire.

Limiting steel temperature for special steels may be appropriately calculated using the thermal characteristics of the material obtained from the supplier of the steel.

22.6 Temperature Increase with Time in Protected Members

22.6.1 The time (t) at which the limiting temperature (T_l) is attained shall be determined by calculation on the basis of a suitable series of fire tests in accordance with **22.6.2** or from the results of a single test in accordance with **22.6.3**.

22.6.1.1 For beams and for all members with a four-sided fire exposure condition, the limiting temperature (T_l) shall be taken as the average of all of the temperatures measured at the thermocouple locations on all sides.

22.6.1.2 For columns with a three-sided fire exposure condition, the limiting temperature (T_l) shall be taken as the average of the temperatures measured at the thermocouple locations on the face farthest from the wall. Alternatively, the temperatures from members with a four-sided fire exposure condition and having the same surface area to mass ratio may be used.

22.6.2 *Temperature Based on Test Series*

Calculation of the variation of steel temperature with time shall be by interpolation of the results of a series of fire tests using the regression analysis equation specified in **22.6.2.1**, subject to the limitations and conditions of **22.6.2.3**.

22.6.2.1 *Regression analysis*

The relationship between temperature (T) and time (t) for a series of tests on a group shall be calculated by least-square regression as follows:

$$t = k_0 + k_1 h_i + k_2\left(\frac{h_i}{k_{sm}}\right) + k_3 T + k_4 h_i T + k_5\left(\frac{h_i T}{k_{sm}}\right) + k_6\left(\frac{T}{k_{sm}}\right)$$

where

h_i = thickness of fire protection material, in mm;

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k_0 to k_6 = regression coefficients from test data (*see* **22.6.2.2**);

k_{sm} = exposed surface area to mass ratio, in 10^3 mm^2/kg;

t = time from the start of the test, in min; and

T = steel temperature, in 0 °C obtained from test as given in **22.6.1**, $T > 250$ °C.

22.6.2.2 *In lieu of* test results, the values for coefficients in Table 30 may be used in the equation **22.6.2.1** when the test satisfies the conditions specified in **22.6.2.3**.

Table 30 Regression Coefficients, k

(*Clause* 22.6.2.2)

Sl No.	k_0	k_1	k_2	k_3	k_4	k_5	k_6
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	– 25.90	1.698	– 13.71	0.030 0	0.000 5	0.514 4	6.633

22.6.2.3 *Limitations and conditions on use of regression analysis*

Test data to be utilized in accordance with **22.6.2.1**, shall satisfy the following:

a) Steel members shall be protected with board, sprayed blanket or similar insulation materials having a dry density less than 1000 kg/m^3;

b) All tests shall incorporate the same fire protection system;

c) All members shall have the same fire exposure condition;

d) Test series shall include at least nine tests;

e) Test series may include prototypes which have not been loaded provided that stickability has been demonstrated; and

f) All members subject to a three-sided fire exposure condition shall be within a group in accordance with **22.9**.

The regression equation obtained for one fire protection system may be applied to another system using the same fire protection material and the same fire exposure condition provided that stickability has been demonstrated for the second system.

A regression equation obtained using prototypes with a four-sided fire exposure condition may be applied to a member with a three-sided fire exposure condition provided that stickability has been demonstrated for the three-sided case.

22.6.3 *Temperature Based on Single Test*

The variation of steel temperature with time measured in a standard fire test may be used without modification provided:

a) fire protection system is the same as the prototype;

b) fire exposure condition is the same as the prototype;

c) fire protection material thickness is equal to or greater than that of the prototype;

d) surface area to mass ratio is equal to or less than that of the prototype; and

e) where the prototype has been submitted to a standard fire test in an unloaded condition, stickability has been separately demonstrated.

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22.6.4 *Parameters of Importance in the Standard Fire Test*

a) Specimen type, loading, configuration;

b) Exposed surface area to mass ratio;

c) Insulation type, thermal properties and thickness; and

d) Moisture content of the insulation material.

22.7 Temperature Increase with Time in Unprotected Members

The time (t) at which the limiting temperature (T_l) is attained shall be calculated using the following equations:

a) Three-sided fire exposure condition:

$$t = -5.2 + 0.022\ \ 1\,T + \left(\frac{0.433T_l}{k_{sm}}\right)$$

b) Four-sided fire exposure condition:

$$t = -4.7 + 0.0263\,T + \left(\frac{0.213T_l}{k_{sm}}\right)$$

where

k_{sm} = exposed surface area to mass ratio, 2×10^3 mm^2/kg ≤ k_{sm} ≤ 35×10^3 mm^2/kg;

t = time, in min, from the start of the test; and

T = steel temperature, in ºC, 500 ºC ≤ T ≤ 750 ºC.

For temperatures below 500 ºC, linear interpolation shall be used, based on the time at 500 ºC and an initial temperature of 20 °C at t equals 0.

22.8 Determination of PSA from a Single Test

The period of structural adequacy (PSA) determined from a single test may be applied without modification provided:

a) conditions, specified in **22.6.3** are satisfied;

b) conditions of support are the same as the prototype and the restraints are not less favourable than those of the prototype; and

c) ratio of the design load for fire to the design capacity of the member is less than or equal to that of the prototype.

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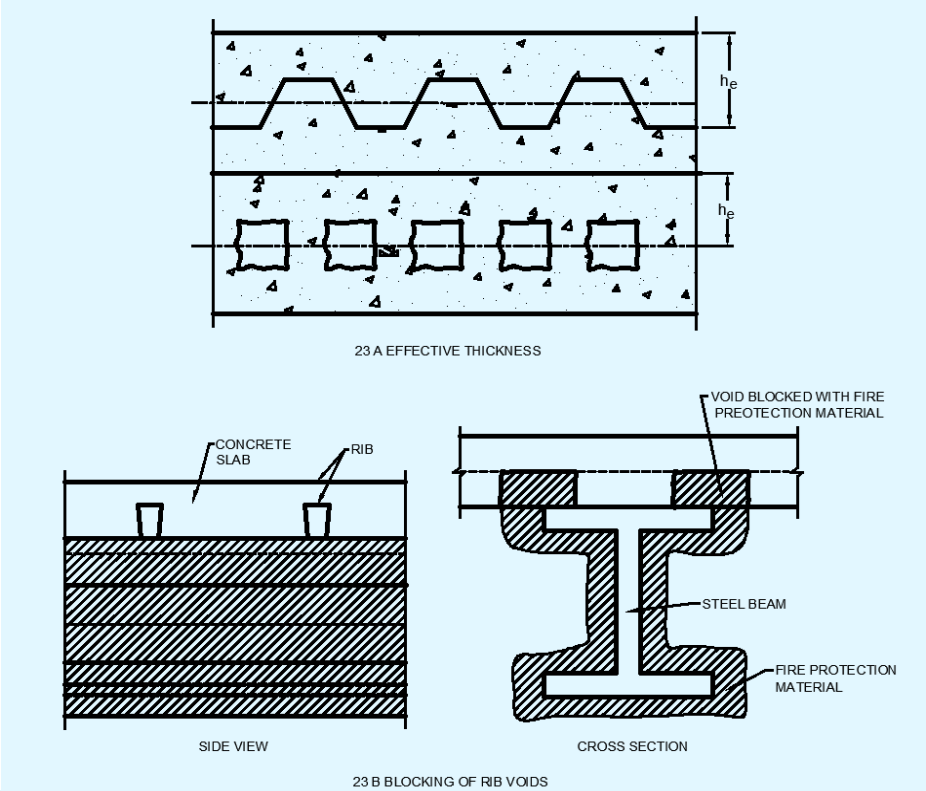

FIG. 25 THREE SIDED FIRE EXPOSURE CONDITION REQUIREMENTS

22.9 Three-sided Fire Exposure Condition

Members subject to a three-sided fire exposure condition shall be considered in separate groups unless the following conditions are satisfied:

a) the characteristics of the members of a group as given below, shall not vary from one another by more than:

1) Concrete density: $\left(\frac{\text{highest in group}}{\text{lowest in group}}\right) \leq 1.25$; and

2) Effective thickness (h_e): $\left(\frac{\text{largest in group}}{\text{smallest in group}}\right) \leq 1.25$

where the effective thickness (h_e) is equal to the cross sectional area excluding voids per unit width, as shown in Fig. 25 (a).

b) Rib voids shall either be:

1) all open; or

2) all blocked as shown in Fig. 25 (b).

c) Concrete slabs may incorporate permanent steel deck formwork.

22.10 Special Considerations

22.10.1 *Connections*

Connections shall be protected with the maximum thickness of fire protection material required for any of the members framing into the connection to achieve their respective fire-resistance levels. This thickness shall be maintained over all connection components, including bolt heads, welds and splice plates.

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22.10.2 *Web Penetrations*

The thickness of fire protection material at and adjacent to web penetrations shall be the greatest of that required, when:

a) area above the penetration is considered as a three-sided fire exposure condition (k_{sm1}) (*see* Fig. 26);

b) area below the penetration is considered as a four-sided fire exposure condition (k_{sm2}) (*see* Fig. 26); and

c) section as a whole is considered as a three-sided fire exposure condition (k_{sm}) (*see* Fig. 26).

This thickness shall be applied over the full beam depth and shall extend on each side of the penetration for a distance at least equal to the beam depth and not less than 300 mm.

22.11 Fire Resistance Rating

The fire resistance rating of various building components such as walls, columns, beams, and floors are given in Table 31 and Table 32. Fire damage assessment of various structural elements of the building and adequacy of the structural repairs can be done by the fire resistance rating for encased steel column and beam (*see* Table 31 and Table 32).

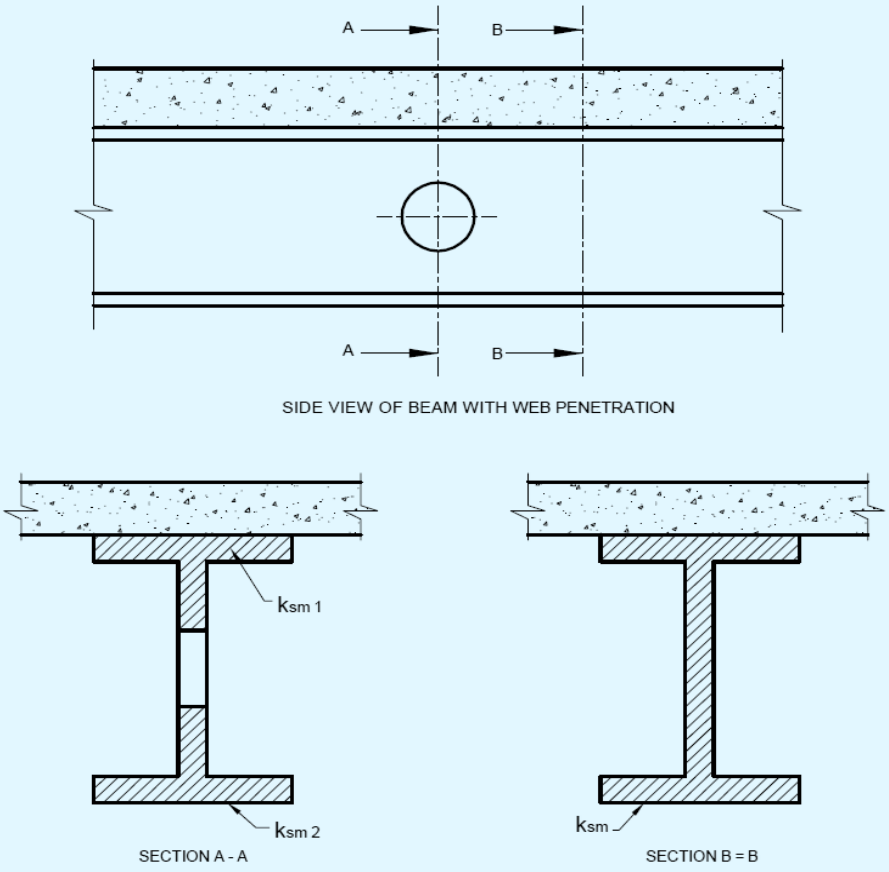

FIG. 26 WEB PENETRATION

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Table 31 Encased Steel Columns, 203 mm × 203 mm (Protection Applied on Four Sides)

(*Clause* 22.11)

Sl No.	Nature of Construction and Materials	Minimum Dimensions (in mm), Excluding Any Finish, for a Fire Resistance of				
		1 h	1 ½ h	2 h	3 h	4 h
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	Hollow protection (without an air cavity over the flanges):					
	a) Metal lathing with trowelled lightweight aggregate gypsum plaster [1)]	13	15	20	32	—
	b) Plasterboard with 1.6 mm wire binding at 100 mm pitch, finished with lightweight aggregate gypsum plaster less than the thickness specified:					
	1) 9.5 mm plaster board; and	10	15	—	—	—
	2) 19 mm plaster board.	10	13	20	—	—
	c) Asbestos insulating boards, thickness of board:					
	1) single thickness of board, with 6 mm cover fillets at transverse joints; and	—	19	25	—	—
	2) two layers, of total thickness.	—	—	—	38	50
	d) Solid bricks of clay, composition or sand lime, reinforced in every horizontal joint, unplastered	50	50	50	75	100
	e) Aerated concrete blocks	60	60	60	—	—
	f) Solid blocks of lightweight concrete hollow protection (with an air cavity over the flanges)	50	50	50	60	75
ii)	Asbestos insulating board screwed to 25 mm asbestos battens	12	19	—	—	—
iii)	Solid protections:					
	a) Concrete, not leaner than 1:2:4 mix (unplastered):					
	1) Concrete not assumed to be load bearing, reinforced[2)]; and	25	25	25	50	75
	2) Concrete assumed to be load bearing	50	50	50	75	75
	b) Lightweight concrete, not leaner than 1 : 2 : 4 mix (unplastered) concrete not assumed to be load bearing, reinforced [2)]	25	25	25	25	25

[1)] So fixed or designed, as to allow full penetration for mechanical bond.

[2)] Reinforcement shall consist of steel binding wire not less than 2.3 mm diameter, or a steel mesh weighing not less than 0.5 kg/m2. In concrete protection, the spacing of that reinforcement shall not exceed 200 mm in any direction.

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Table 32 Encased Steel Beams, 406 mm × 176 mm (Protection Applied on Three Sides)

(*Clause* 22.11)

Sl No.	Nature of Construction and Materials	Minimum Thickness (in mm) of Protection for a Fire Resistance of					
		½ h	1 h	1½ h	2 h	3 h	4 h
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Hollow protection (without an air cavity beneath the lower flanges):						
	a) Metal lathing with trowelled lightweight aggregate gypsum plaster[1)]	13	13	15	20	25	—
	b) Plasterboard with 1.6 mm wire binding at 100 mm pitch, finished with lightweight aggregate gypsum plaster less than the thickness specified[2)]:						
	1) 9.5 mm plaster board; and	10	10	15	—	—	—
	2) 19 mm plaster board	10	10	13	20	—	—
	c) Asbestos insulating boards, thickness of board:						
	1) single thickness of board, with 6 mm cover fillets at transverse joints	—	—	19	25	—	—
	2) two layers, of total thickness	—	—	—	—	38	50
ii)	Hollow protection (with an air cavity below the lower flange):						
	a) Asbestos insulating board screwed to 25 mm asbestos battens	9	12	—	—	—	—
iii)	Solid protections:						
	a) Concrete, not leaner than 1 : 2 : 4 mix (unplastered):						
	1) concrete not assumed to be load bearing, reinforced [3)]; and	25	25	25	25	50	75
	2) concrete assumed to be load bearing	50	50	50	50	75	75
	b) Lightweight concrete, not leaner than 1 : 2 : 4 mix (unplastered) [4)]	25	25	25	25	40	60

1 So fixed or designed, as to allow full penetration for mechanical bond.

2 Where wire binding cannot be used, expert advice should be sought regarding alternative methods of support to enable the lower edges of the plasterboard to be fixed together and to the lower flange, and for the top edge of the plasterboard to be held in position.

3 Reinforcement shall consist of steel binding wire not less than 2.3 mm in diameter, or a steel mesh weighing not less than 0.5 kg/m^2. In concrete protection, the spacing of that reinforcement shall not exceed 200 mm in any direction.

4 Concrete not assumed to be load bearing, reinforced.

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23 SUSTAINABLE STEEL CONSTRUCTION

23.1 General

This Chapter specifies general requirements and practices for achieving environmentally sustainable steelwork building construction. This supplements the requirements of the other chapters of this specification.

23.2 Principles of Sustainable Steel Construction

a) The structural steel design should provide a structure capable of fulfilling its intended function while sustaining the specified loads for its intended life with due regard to sustainability, robustness, economy, health and safety. Any expected change in the intended function during its life time should also be considered;

b) In environmentally sustainable construction all stake holders should consider the broader and holistic implications of the decision, which include the whole life cycle sustainability, impacts on other parts of the building and the project, externalities, maintenance, societal cost and consequences, without impinging upon safety and efficiency of the structure;

c) All parties in the supply chain for structural steelwork should take practical steps to reduce the greenhouse gas emissions of the processes over which they have control, thus contributing to sustainable construction. Further, construction operations should avoid/minimize polluting the environment with dust, wastewater, chemicals and leaching debris;

d) Steelmaking should make full use of available secondary material (scrap) globally;

e) Structural steelwork should be designed, detailed, and constructed to minimize material use and wastage of material and facilitate its recovery for reuse at its end-of-life, as a part of the circular economy, to the extent that this is reasonably practicable; and

f) The design should consider practical aspects of construction to avoid repair and rework.

23.3 Specification Requirements from Supply Chain Members

23.3.1 *Requirements from Employer*

a) Appointment of key contractors (Architects, contractors for Steel work, cladding, glazing, Mechanical, Electrical, coating systems, etc) as early as practical to ensure decisions are fully coordinated;

b) Maximization of structural zones that facilitate lean and efficient designs; and

c) Flexibility in design, if use of reclaim materials is considered.

23.3.2 *Requirements from Main Contractor*

a) Early engagement of supply chain and subcontractors with emphasis on sustainability; and

b) Judicious site planning to receive material, use of labour and location of machinery for efficient handling and construction.

23.3.3 *Requirements from the Engineering Group*

a) Design for material efficiency, equipment efficiency, and minimal embodied energy and longevity;

b) Consider holistically the environmental impact implication of temporary works and supports;

c) Design with inbuilt flexibility and adaptability;

d) Consider the economic and environmental implications of the type of connections chosen; and

e) Assess the requirement of protective treatment against corrosion and fire to minimize the life time maintenance and avoid over-specification.

23.3.4 *Requirements from Steelwork Contractor*

Establish a carbon reduction plan in steelwork. This may include use of low carbon emission transport, maximize the steelwork loads, optimizing haulage routes, minimizing empty runs, etc.

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23.4 Sustainable Design

23.4.1 *General*

The environmental impacts associated with steelwork building construction be greatly reduced during the design process by:

a) reducing the material in the structure;

b) minimizing fabrication, and avoidance of maintenance during service life; and

c) Identifying environmental improvements at the conceptual design stage requires the involvement of all the stakeholders such as architects, steelwork contractor during all design stages. Collaborative design processes result in:

   1) materials being used more efficiently;

   2) fewer on-site changes; and

   3) reduced environmental impacts.

Greenhouse gas or carbon emissions currently being the priority, action is required by all parts of the supply chain for the sector to decarbonize in line with global targets. This includes supply-side measures, in particularly decarbonizing steel-making and also demand-side measures including more efficient design and design for longevity, deconstruction and reuse.

Greenhouse gas emission should be based on life cycle assessment, which should include emissions in material manufacturing, transport, fabrication, erection, finishing, commissioning, maintenance during life and finally at the end of useful life deconstruction and disposal of the materials as waste or through recycling and reuse.

Further, environmentally friendly designs should be implemented in the construction projects and pollution of the environment during the process of construction should be avoided, if not minimized.

23.4.2 *Minimizing Material Consumption through Design*

Minimizing material consumption through design may be achieved by:

a) Avoiding over-specification of design loads;

b) Optimizing spans and grids: and adapting the building layout to take full advantage of the structural steelwork’s attributes;

c) Minimizing complex load paths and transfers in order to maximize direct load paths to ground;

d) Ensuring that structural deflection and vibration criteria, coordination zones and member depths are appropriate and allow for efficient structural solutions that avoid excessive material use;

e) Aiming to design for utilizations as close to 1.0 as practically possible, with due regard to permanent and temporary conditions;

f) Specifying higher strength steel grades in members or part of members not governed by serviceability criteria;

g) Considering key load paths, anticipated connection types and accurate loading criteria, in order to facilitate lean design principles;

h) Avoiding the over rationalization of steel member sizes to ensure efficient use of material, with due consideration of minimum economic quantities for procurement;

j) Reviewing section sizes and minimizing weight, whilst complying with safe methods of erection and fire protection requirements; and

k) Semi-continuous design, which assumes that the joints transfer moments from the beam to the column and allow rotation between the connected members as the load is resisted.

NOTE — Historically, steelwork design and fabrication has mainly been driven by cost however, sustainability and particularly the climate emergency, are likely to change this with more clients willing or mandated to pay more for sustainable and low carbon buildings. Reducing the carbon impact of steel structures through lean and less conservative design is technically achievable but can require more design effort and more complex fabrication.

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23.4.3 *Design for Adaptability and Deconstruction*

Design engineer should review design options with the steelwork contractor that may contribute to future flexibility, adaptability, deconstruction and reuse of the building structure, such as:

a) how the building design and information records about the materials and members can facilitate future, remodelling, deconstruction and recovery for reuse and recycling;

b) ensuring accessibility to members and connections to allow for deconstruction and for ease of strengthening in the future; and

c) greater standardization of sections sizes, lengths and connections, if appropriate from adaptability and deconstruction.

23.5 Sustainable Fabrication

23.5.1 *Information Records*

A non-exhaustive checklist of information to be communicated in sustainable fabrication are given below:

a) Sustainability information to be provided to the steelwork contractor:
   1) any part of the steelwork that is to incorporate reclaimed steel products;
   2) any part of the steelwork that is to be left untreated;
   3) any details of the fire design, namely the design utilization per section size in the fire condition;
   4) any workshop waste targets;
   5) a life cycle assessment of the whole building; and
   6) any specific material sourcing requirements.

b) Sustainability information to be provided by the steelwork contractor:
   1) All relevant EPDs for the steel and other products;
   2) The embodied carbon of the fabricated steelwork;
   3) Digital records of the ‘As erected’ structure, which should include all condition of structures, survey, metadata, etc;
   4) Material quality and quantity data, including batch traceability for main structural members to facilitate future reuse (to be included in the operation and maintenance manual; and
   5) Documentation of evidence of an active environmental management system.

23.5.2 *Fabrication Waste Management*

Strategies for fabrication waste management include:

a) ordering directly from mills to minimize offcuts;

b) using stock lengths as ordered efficiently to minimize waste. Where possible, splice locations in steel members should be coordinated to fall within standard stock length sizes;

c) recycling and reusing steel fabrication waste, for example, off-cuts, swarf, etc;

d) considering use off-cuts of material, particularly from plate, elsewhere, for example, temporary works, connection fittings, shims, packs, rather than simply scrapping for recycling. Always prioritized reuse over recycling;

e) considering use of machines that facilitate common line cutting of fittings;

f) minimizing coating waste and coating packaging waste, by using intermediate bulk; and

g) Minimizing/ avoiding overspray of coatings should ensure operators are properly trained and the appropriate and properly maintained equipment is used.

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23.5.3 *Sustainable Welding*

Strategies for sustainable welding include:

a) Avoiding low productivity processes and using automatic and semi-automatic processes with continuous wire feed, thus using material efficiently and avoiding energy waste;

b) Adopting energy-saving processes such as, inverter-based power sources instead of older regulator/rectifier-based sources, electrically powered pre-heating with temperature control instead of gas flame;

c) Minimizing pre-heat in the welding procedure specification and welding continuously to avoid the need to re-heat; and

d) Periodically reviewing welding techniques and weld procedures to make the best use of the latest advancements in welding equipment and consumables, including the selection of joint set-ups and consumables that allow for deeper penetration fillet welds to be achieved, thus potentially reducing the number of weld runs required.

23.5.4 *Sustainable Fire and Corrosion Protection Systems*

23.5.4.1 *Untreated steelwork*

Any part of the steelwork such as steelwork in a less corrosivity category, that is to be left untreated shall be identified in the Project Specification.

23.5.4.2 *Paint systems*

The protective coating system shall satisfy the requirements given in **21**.

23.5.4.3 *Intumescent systems*

For the same fire resistance water-based acrylic intumescent likely to have a lower environmental impact, when compared to solvent-based acrylic intumescent.

23.5.4.4 *Hot-dip galvanizing*

Requirements for thicker coatings, for example, by prior grit blasting, should be avoided by prior assessment of the predicted galvanized coating life and the minimum thickness required to avoid over-specification. For certain steel members (for example, with higher reactivity during galvanizing and/or larger section thickness), thicker coatings may also be achieved without grit blasting. Design and fabrication considerations for hot dip galvanizing should ensure adequate drainage and optimize material efficiency. Plan should optimize transport distances and provide direct delivery to the construction works, where feasible.

24 FABRICATION AND ERECTION

24.1 General

Tolerances for fabrication of steel structures shall conform to the accepted standard [C-6A(44)]. Tolerances for erection of steel structures shall conform to the accepted standard [C-6A(45)]. For general guidance on fabrication by welding, reference may be made to good practice [C-6A(46)].

In addition to the above, the following requirements shall also be considered:

a) The designer shall, in conjunction with the floor framing and vertical framing/stacking arrangements of the project, provide suitable tolerance mitigation measures. Such measures may include the use of slotted or oversized holes in beams and columns, use of HSFG bolts, or other appropriate detailing. These provisions need not be adopted for all members; however, they shall be judiciously incorporated in selected connections and splices to achieve the desired tolerance accommodation.

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b) The structural designer shall coordinate with other disciplines, such as façade, building services, and transportation systems, and ensure that the specified tolerances for steel structures are compatible with their requirements. Where the design necessitates tolerances more stringent than those specified in the relevant codes, such requirements shall be clearly indicated in the drawings and specifications. However, in no case shall the specified tolerances be more stringent than the applicable mill rolling tolerances.

c) The designer shall specify the permissible overall horizontal and vertical tolerances for the completed building structure, in accordance with the design intent. While [C-6A(45)] specifies tolerances for erected elements, it does not address the overall geometrical accuracy of the completed structure. Therefore, the structural designer shall, in coordination with the architect and other relevant disciplines, define the acceptable limits of overall as-built accuracy. These limits shall be clearly specified so as to enable proper planning and execution by the contractor at all stages of construction.

24.2 Fabrication Procedures

24.2.1 *Straightening*

Material shall be straightened or formed to the specified configuration by methods that will not reduce the properties of the material below the values used in design. Local application of pressure at room or at elevated temperature or other thermal means may be used for straightening, provided the above is satisfied.

24.2.2 *Clearances*

The erection clearance for cleated ends of members, connecting steel to steel should preferably be not greater than 2.0 mm at each end. The erection clearance at ends of beams without web cleats should be not more than 3 mm at each end. Where for practical reasons, greater clearance is necessary, suitably designed seating should be provided.

24.2.2.1 The holes for bolts may be made as given in Table 20 of **17.2.1**, unless otherwise specified by the engineer. The hole diameter in base plates shall not exceed the anchor bolt diameter by more than 6 mm.

24.2.2.2 In friction type of connection clearance may be maintained, unless specified otherwise in the design document.

24.2.3 *Cutting*

Cutting shall be effected by sawing, shearing, cropping, machining or thermal cutting process. Shearing, cropping and gas cutting shall be clean, reasonably square, and free from any distortion. Should the inspector find it necessary, the edges shall be ground after cutting. Planning or finishing of sheared or gas-cut edges of plates or shapes shall not be required, unless specially noted on drawing or included in stipulated edge preparation for welding or when specifically required in the following section.

Re-entrant corners shall be free from notches and shall have largest practical radii with a minimum radius of 15 mm.

24.2.3.1 *Shearing*

Shearing of items over 16 mm thick to be galvanized and subject to tensile force or bending moment shall not be carried out, unless the item is stress relieved subsequently.

The use of sheared edges in the tension area shall be avoided in location subject to plastic hinge rotation at factored loading.

24.2.3.2 *Thermal cutting*

Gas cutting of high tensile steel by mechanically controlled torch may be permitted, provided special care is taken to leave sufficient metal to be removed by machining, so that all metal that has been hardened by flame is removed. Hand flame cutting may be permitted only subject to the approval of the inspecting authority.

Except where the material is subsequently joined by welding, no load shall be transmitted through a gas cut surface.

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Thermally cut free edges, which shall be subject to calculated static tensile stress shall be free from round bottom gouges greater than 5 mm deep. Gouges greater than 5 mm deep and notches shall be removed by grinding.

24.2.4 *Holing*

24.2.4.1 Holes through more than one thickness of material for members, such as compound stanchion and girder flanges, shall be where possible, drilled after the members are assembled and tightly clamped or bolted together. A round hole for a bolt shall either be machine flame cut, or drilled full size, or sub-punched 3 mm undersize and reamed to size or punched full size.

Hand flame cutting of a bolt hole shall not be permitted except as a site rectification measure for holes in column base plates.

24.2.4.2 *Punching*

A punched hole shall be permitted only in material whose yield stress (f_y) does not exceed 360 MPa and where thickness does not exceed ($5600/f_y$) mm. In cyclically loaded details, punching shall be avoided in plates with thickness greater than 12 mm.

For greater thickness and cyclically loaded details, holes shall be either drilled from the solid or sub-punched or sub drilled and reamed.

The die for all sub-punched holes or the drill for all sub-drilled holes shall be at least 3 mm smaller than the required diameter of finished hole.

24.2.4.3 *Oversize holes*

A special plate washer of minimum thickness 4 mm shall be used under the nut, if the hole diameter is larger than the bolt diameter by 3 mm or more.

Oversize hole shall not exceed $1.25d$ or $(d + 8)$ mm in diameter, were d is the nominal bolt diameter, in mm.

A short slotted hole shall not exceed the appropriate hole size in width and $1.33d$, in length. A long slotted hole shall not exceed the appropriate hole size in width and $2.5d$, in length. If the slot length is larger than those specified, shear transfer in the direction of slot is not admissible even in friction type of connection.

Slotted holes shall be punched either in one operation or else formed by punching or drilling two round holes apart and completed by high quality mechanically controlled flame cutting and dressing to ensure that bolt can freely travel the full length of the slot.

24.2.4.4 *Fitted bolt holes*

Holes for turned and fitted bolts shall be drilled to a diameter equal to the nominal diameter of the shank or barrel subject to tolerance specified in the accepted standards [C-6A(47)]. Preferably, parts to be connected with close tolerance or barrel bolts shall be firmly held together by tacking bolts or clamps and the holes drilled through all the thicknesses at one operation and subsequently reamed to size. All holes not drilled through all thicknesses at one operation shall be drilled to a smaller size and reamed out after assembly. Where this is not practicable, the parts shall be drilled and reamed separately through hard bushed steel jigs.

24.2.4.5 Holes for rivets or bolts shall not be formed generally by gas cutting process. However, advanced gas cutting processes such as plasma cutting may be used to make holes in statically loaded members only. In cyclically loaded members subjected to tensile stresses which are vulnerable under fatigue, gas cutting shall not be used unless subsequent reaming is done to remove the material in the heat affected zone around the hole.

24.3 Assembly

All parts of bolted members shall be pinned or bolted and rigidly held together during assembly.

The component parts shall be assembled and aligned in such a manner that they are neither twisted nor otherwise damaged, and shall be so prepared that the specified camber, if any, is provided.

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24.3.1 *Holes in Assembly*

When holes are drilled in one operation through two or more separable parts, these parts, when so specified by the engineer, shall be separated after drilling and the burrs removed.

Matching holes for rivets and black bolts shall register with each other so that a gauge of 1.5 mm or 2.0 mm (as the case may be, depending on whether the diameter of the rivet or bolt is less than or more than 25 mm) less in diameter than the diameter of the hole will pass freely through the assembled members in the direction at right angle to such members.

Drilling done during assembly to align holes shall not distort the metal or enlarge the holes.

Holes in adjacent part shall match sufficiently well to permit easy entry of bolts. If necessary, holes except oversize or slotted holes may be enlarged to admit bolts, by moderate amount of reaming.

24.3.2 *Thread Length*

When design is based on bolts with unthreaded shanks in the shear plane, appropriate measures shall be specified to ensure that, after allowing for tolerance, neither the threads nor the thread run-out be in the shear plane.

The length of bolt shall be such that at least one clear thread shows above the nut and at least one thread plus the thread run out is clear beneath the nut after tightening. One washer shall be provided under the rotated part.

24.3.3 *Assembly Subjected to Vibration*

When non-preloaded bolts are used in a structure subject to vibration, the nuts shall be secured by locking devices or other mechanical means. The nuts of preloaded bolts may be assumed to be sufficiently secured by the normal tightening procedure.

24.3.4 *Washers*

Washers are not normally required on non-preloaded bolts, unless specified otherwise. Tapered washers shall be used where the surface is inclined at more than 3° to a plane perpendicular to the bolt axis.

Hardened washer shall be used for preloaded bolts or the nut, whichever is to be rotated.

All material within the grip of the bolt shall be steel and no compressible material shall be permitted in the grip.

24.4 Riveting

24.4.1 Rivets shall be heated uniformly throughout their length, without burning or excessive scaling, and shall be of sufficient length to provide a head of standard dimensions. These shall, when driven, completely fill the holes and, if countersunk, the countersinking shall be fully filled by the rivet. If required, any protrusion of the countersunk head shall be dressed off flush.

24.4.2 Riveted member shall have all parts firmly drawn and held together before and during riveting, and special care shall be taken in this respect for all single-riveted connections. For multiple riveted connections, a service bolt shall be provided in every third or fourth hole.

24.4.3 Wherever practicable, machine riveting shall be carried out by using machines of the steady pressure type.

24.4.4 All loose, burned or otherwise defective rivets shall be cut out and replaced before the structure is loaded, and special care shall be taken to inspect all single riveted connections.

24.4.5 Special care shall be taken in heating and driving long rivets.

24.5 Bolting

24.5.1 In all cases where the full bearing area of the bolt is to be developed, the bolt shall be provided with a washer of sufficient thickness under the nut to avoid any threaded portion of the bolt being within the thickness or the parts bolted together, unless accounted for in design.

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24.5.2 Pre-tensioned bolts shall be subjected to initial tension (the proof stress) by an appropriate pre-calibrated method.

24.6 Welding

24.6.1 Welding shall be in accordance with good practices [C-6A(48)], as appropriate.

24.6.2 For welding of any particular type of joint, welders shall give evidence acceptable to the purchaser of having satisfactorily completed appropriate tests as prescribed in good practices [C-6A(49)], as relevant.

24.6.3 Assembly and welding shall be carried out in such a way to minimize distortion and residual stress and that the final dimensions are within appropriate tolerances.

24.7 Machining of Butts, Caps and Bases

24.7.1 Column splices and butt joints of struts and compression members, depending on contact for stress transmission, shall be accurately machined and close-butted over the whole section with a clearance not exceeding 0.2 mm locally, at any place. Sum of all such clearance shall not be more than 30 percent of the contact area for stress transmission. In column caps and bases, the ends of shafts together with the attached gussets, angles, channels, etc; after connecting together should be accurately machined so that clearance between the contact surfaces shall not exceed 2 mm locally, subject further to the condition that sum total of all such clearance shall not exceed 30 percent of the total contact area for stress transmission. Care should be taken that these gussets, connecting angles or channels are fixed with such accuracy that they are not reduced in thickness by machining by more than 2.0 mm.

24.7.2 Where sufficient gussets and rivets or welds are provided to transmit the entire loading (*see* provisions under **11**), the column ends need not be machined.

24.7.3 *Slab Bases and Caps*

Slab bases and slab caps, except when cut from material with true surfaces, shall be accurately machined over the bearing surfaces and shall be in effective contact with the end of the stanchion, the bearing face which is to be grouted to fit tightly at both top and bottom, unless welds are provided to transmit the entire column face.

24.7.4 To facilitate grouting, sufficient gap shall be left between the base plates and top of pedestal and holes shall be provided where necessary in stanchion bases for the escape of air.

24.8 Painting

24.8.1 Painting shall be done in accordance with the good practice good practices [C-6A(50)].

22.8.2 All surfaces, which are to be painted, oiled or otherwise treated, shall be dry and thoroughly cleaned to remove all loose scale and loose rust.

24.8.3 Shop contact surfaces need not be painted unless specified. If so specified, they shall be brought together while the paint is still wet.

24.8.4 Surfaces not in contact, but inaccessible after shop assembly, shall receive the full specified protective treatment before assembly. This does not apply to the interior of sealed hollow sections.

24.8.5 Chequered plates shall be painted but the details of painting shall be specified by the purchaser.

24.8.6 In case of surfaces to be welded, the steel shall not be painted or metal coated within a suitable distance of any edge to be welded, if the paint specified or the metal coating is likely to be harmful to welders or impair the quality of the welds.

24.8.7 Welds and adjacent parent metal shall not be painted prior to de-slagging, inspection and approval.

24.8.8 Parts to be encased in concrete shall not be painted or oiled.

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24.8.9 Contact surface in friction type connection shall not be painted in advance.

24.9 Marking

Each piece of steel work shall be distinctly marked before dispatch, in accordance with a marking diagram and shall bear such other marks as will facilitate erection.

24.10 Shop Erection

24.10.1 The steel work shall be temporarily shop erected complete or as arranged with the inspection agency so that accuracy of fit may be checked before dispatch. The parts shall be shop assembled with sufficient numbers of parallel drifts to bring and keep the parts in place.

24.10.2 In the case of parts drilled or punched, through steel jigs with bushes resulting in all similar parts being interchangeable, the steelwork may be shop erected in such position as arranged with the inspection agency.

24.10.3 In case of shop fabrication using numerically controlled machine data generated by computer software (like CAD), the shop erection may be dispensed with at the discretion of the inspector.

24.11 Packing

All projecting plates or bars and all ends of members at joints shall be stiffened, all straight bars and plates shall be bundled, all screwed ends and machined surfaces shall be suitably packed and all rivets, bolts, nuts, washers and small and loose parts shall be packed separately in cases, so as to prevent damage or distortion during transit.

24.12 Inspection and Testing

24.12.1 The inspecting authority shall have free access at all reasonable times to those parts of the manufacturer's works which are concerned with the fabrication of the steelwork and shall be afforded all reasonable facilities for satisfying himself that the fabrication is being undertaken in accordance with the provisions of this section.

24.12.2 Unless specified otherwise, inspection shall be made at the place of manufacture prior to dispatch and shall be conducted so as not to interfere unnecessarily with the operation of the work.

24.12.3 The manufacturer shall guarantee compliance with the provisions of this standard, if required to do so by the purchaser.

24.12.4 Should any structure or part of a structure be found not to comply with any of the provisions of this section, it shall be liable to rejection. No structure or part of the structure, once rejected shall be resubmitted for test, except in cases where the purchaser or his authorized representative considers the defect as rectifiable.

24.12.5 Defects, which may appear during fabrication, shall be made good with the consent of and according to the procedure laid down by the inspecting authority.

24.12.6 All gauges and templates necessary to satisfy the inspection authority shall be supplied by the manufacturer. The inspecting authority may, at his discretion, check the test results obtained at the manufacturer's works by independent testing at outside laboratory, and should the material so tested be found to be unsatisfactory, the cost of such tests shall be borne by the manufacturer, and if found satisfactory, the cost shall be borne by the purchaser.

24.13 Site Erection

24.13.1 *Plant and Equipment*

The suitability and capacity of all plant and equipment used for erection shall be to the satisfaction of the engineer.

24.13.2 *Storing and Handling*

All structural steel should be so stored and handled at the site that the members are not subjected to excessive stresses and damage by corrosion due to exposure to environment.

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24.13.3 *Setting Out*

The positioning and leveling of all steelwork, the plumbing of stanchions and the placing of every part of the structure with accuracy shall be in accordance with the approved drawings and to the satisfaction of the engineer in accordance with the deviation permitted below.

24.13.3.1 *Erection tolerances*

Unloaded steel structure, as erected, shall satisfy the tolerance criteria specified in the accepted standards [C-6A(47)].

24.13.4 *Safety during Fabrication and Erection*

24.13.4.1 All steel materials including fabricated structures, either at fabrication shop or at erection site, shall be handled only by a worker skilled in such jobs; where necessary with load tested lifting devices, having tested wire rope slings of correct size. The devices should be well maintained and operated by experienced operators.

24.13.4.2 Oxygen and acetylene cylinders and their hoses shall have distinctive colours. Cylinders should be stored in upright position in well-ventilated rooms or in open air, not exposed to flames, naked lights or extreme heat and should also be in upright position when they are being used. All gas cutting works shall be done only by experienced skilled gas cutters, equipped with gloves, boots, aprons, goggles and good cutting sets of approved make.

24.13.4.3 While doing any welding work, it should be ensured that the welding machine is earthed and the welding cables are free from damage. The welder and his assistant shall use a face shield or head shield with a welding lens and clear cover glass and their hands, legs and bodies shall be well protected by leather gloves, shoes and aprons. Combustible materials should be kept away from the sparks and globules of molten metals generated in any arc welding. In case of welding in a confined place, it should be provided with an exhaust system to take care of the harmful gases, fumes and dusts generated.

24.13.4.4 In addition to precautions against all the hazards mentioned above, erection workers shall also be protected in the following manner:

a) all workers shall wear helmets and shall also be provided with gloves and shoes. In addition those working at heights shall use safety belts;

b) all structures shall be so braced/guyed during erection that there is no possibility of collapse before erection work is completed; and

c) warning signs such as ‘DANGER’, ‘CAUTION’, ‘440 V’, ‘DO NOT SMOKE’, ‘LOOK AHEAD’, etc; should be displayed at appropriate places.

24.13.4.5 For detailed safety precautions during erection, reference shall be made to the good practice [C-6A(51)].

24.13.5 *Field Connections*

24.13.5.1 *Field riveting*

Rivets driven at the site shall be heated and driven with the same care as those driven in the shop.

24.13.5.2 *Field bolting*

Field bolting shall be carried out with the same care as required for shop bolting.

24.13.5.3 *Fillet welding*

Field assembly and welding shall be executed in accordance with the requirements for shop fabrications excepting such as manifestly apply to shop conditions only. Where the steel has been delivered painted, the paint shall be removed for a distance of at least 50 mm on either side of the joint.

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24.14 Painting after Erection

24.14.1 Before painting of such steel which is delivered unpainted is commenced, all surfaces to be painted shall be dry and thoroughly cleaned from all loose scale and rust, as required by the surface protection specification.

24.14.2 The specified protective treatment shall be completed after erection. All rivet and bolt heads and the site welds after de-slagging shall be cleaned. Damaged or deteriorated paint surfaces shall first be made good with the same type of paint as the shop coat. Where specified, surfaces which will be in contact after site assembly, shall receive a coat of paint (in addition to any shop priming) and shall be brought together while the paint is still wet. No painting shall be used on contact surfaces in the friction connection, unless specified otherwise by the design document.

24.14.3 Where the steel has received a metal coating in the shop, this coating shall be completed on site so as to be continuous over any welds and site rivets or bolts, subject to the approval of the engineer. Painting on site may complete protection. Bolts, which have been galvanized or similarly treated, are exempted from this requirement.

24.14.4 Surface, which will be inaccessible after site assembly, shall receive the full-specified protective treatment before assembly.

24.14.5 Site painting should not be done in frosty or foggy weather, or when humidity is such as to cause condensation on the surfaces to be painted.

24.15 Bedding Requirement

24.15.1 Bedding shall be carried out with Portland cement grout or mortar, as described under **24.15.4** or fine cement concrete in accordance with good practice [C-6A(52)].

24.15.2 For multistoried buildings, this operation shall not be carried out until a sufficient number of bottom lengths of stanchions have been properly lined, levelled and plumbed and sufficient floor beams are in position.

24.15.3 Whatever method is employed, the operation shall not be carried out until the steelwork has been finally levelled and plumbed, stanchion bases being supported meanwhile by steel wedges or nuts; and immediately before grouting, the space under the steel shall be thoroughly cleaned.

24.15.4 Bedding of structure shall be carried out with grout or mortar, which shall be of adequate strength and shall completely fill the space to be grouted and shall either be placed under pressure or by ramming against fixed supports. The grouts or mortar used shall be non-shrinking variety.

24.16 Steelwork Tenders and Contracts

A few recommendations are given in Annex H for general information.

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ANNEX A

(*Clause* 4)

SYMBOLS

Symbols used in this section shall have the following meanings with respect to the structure or member or condition, unless otherwise defined elsewhere in this Section:

A	=	area of cross section;
A_c	=	area at root of threads;
A_e	=	effective cross sectional area;
A_{ef}	=	reduced effective flange area;
A_f	=	total flange area;
A_g	=	gross cross sectional area;
A_{gf}	=	gross cross sectional area of flange;
A_{go}	=	gross cross sectional area of outstanding (not connected) leg of a member;
A_n	=	net area of the total cross section;
A_{nb}	=	net tensile cross sectional area of bolt;
A_{nc}	=	net cross sectional area of the connected leg of a member;
A_{nf}	=	net cross sectional area of each flange;
A_{no}	=	net cross sectional area of outstanding (not connected) leg of a member;
A_{pb}	=	nominal bearing area of bolt on any plate;
A_q	=	cross sectional area of a bearing (load carrying) stiffener in contact with the flange;
A_s	=	tensile stress area;
A_{sb}	=	gross cross sectional area of a bolt at the shank;
A_{tg}	=	gross sectional area in tension from the centre of the hole to the toe of the angle section/channel section, etc (block shear failure, *see* **13.4**) perpendicular to the line of force;
A_{tn}	=	net sectional area in tension from the centre of the hole to the toe of the angle perpendicular to the line of force block shear failure (*see* **13.4**);
A_v	=	shear area;
A_{vg}	=	gross cross sectional area in shear along the line of transmitted force (*see* **13.4**);
A_{vn}	=	net cross sectional area in shear along the line of transmitted force (*see* **13.4**);
a, b	=	larger and smaller projection of the slab base beyond the rectangle circumscribing the column, respectively (*see* **14.4**);
a_o	=	peak acceleration;
a_1	=	unsupported length of individual elements being laced between lacing points;
B	=	length of side of cap or base plate of a column;
b	=	outstand/width of the element;
b_1	=	stiff bearing length;
b_e	=	effective width of flange between pair of bolts;
b_f	=	width of the flange;
b_i	=	width of flange as an internal element;
b_o	=	width of flange outstand;

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b_p = panel zone width between column flanges at beam-column junction;

b_s = shear lag distance;

b_t = width of tension field;

b_w = width of outstanding leg;

C = centre to centre longitudinal distance of battens;

C_m = coefficient of thermal expansion;

c = spacing of transverse stiffener;

c_b = moment amplification factor for braced member;

c_m = moment reduction factor for lateral torsional buckling strength calculation;

c_s = moment amplification factor for sway frame;

D = overall depth/diameter of the cross section;

d = depth of web, Nominal diameter;

d_2 = twice the clear distance from the compression flange angles, plates or tongue plates to the neutral axis;

d_h = diameter of a bolt/rivet hole;

d_o = nominal diameter of the pipe column or the dimensions of the column in the depth direction of the base plate;

d_p = panel zone depth in the beam-column junction;

E = modulus of elasticity for steel;

E (T) = modulus of elasticity of steel at T ºC;

E (20) = modulus of elasticity of steel at 20 ºC;

E_p = modulus of elasticity of the panel material;

F_{cdw} = buckling strength of un-stiffened beam web under concentrated load;

F_d = factored design load;

F_n = normal force;

F_o = minimum proof pretension in high strength friction grip bolts;

F_{psd} = bearing capacity of load carrying stiffener;

F_q = stiffener force;

F_{qd} = stiffener buckling resistance;

F_{test} = test load;

$F_{test,a}$ = load for acceptance test;

$F_{test, Min}$ = minimum test load from the test to failure;

$F_{test, R}$ = test load resistance;

$F_{test, S}$ = strength test load;

F_w = design capacity of the web in bearing;

F_x = external load, force or reaction;

F_{xd} = buckling resistance of load carrying web stiffener;

f = actual normal stress range for the detail category;

f_1 = frequency for a simply supported one way system;`

f_2 = frequency of floor supported on steel girder perpendicular to the joist;

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f_a = calculated stress due to axial force at service load;

f_{abc} = permissible bending stress in compression at service load;

f_{ac} = permissible compressive stress at service load;

f_{abt} = permissible bending stress in tension at service load;

f_{apb} = permissible bearing stress of the bolt at service load;

f_{asb} = permissible stress of the bolt in shear at service load;

f_{at} = permissible tensile stress at service load;

f_{atb} = permissible tensile stress of the bolt at service load;

f_{aw} = permissible stress of the weld at service load;

f_b = actual bending stress at service load;

f_{bc} = actual bending stress in compression at service load;

f_{bd} = design bending compressive stress corresponding to lateral buckling;

f_{br} = actual bearing stress due to bending at service load;

f_{bt} = actual bending stress in tension at service load;

f_{bs} = permissible bending stress in column base at service load;

f_c = actual axial compressive stress at service load;

f_{cr} = elastic buckling stress of a column, Euler buckling stress;

f_{cd} = design compressive stress;

$f_{cr,b}$ = extreme fibre compressive stress corresponding to elastic lateral buckling moment;

f_e = equivalent stress at service load;

f_f = fatigue stress range corresponding to 5×10^6 cycles of loading;

f_{feq} = equivalent constant amplitude stress;

f_{fd} = design normal fatigue strength;

$f_{f, Max}$ = highest normal stress range;

f_{fn} = normal fatigue stress range;

f_{nw} = normal stress in weld at service load;

f_o = proof stress;

f_p = actual bearing stress at service load;

f_{pb} = actual bearing stress in bending at service load;

f_{psd} = bearing strength of the stiffeners;

f_r = frequency;

f_{sb} = actual shear stress in bolt at service load;

f_t = actual tensile stress at service load;

f_{tb} = actual tensile stress of the bolt at service load;

f_u = characteristic ultimate tensile stress;

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f_{ub} = characteristic ultimate tensile stress of the bolt;

f_{um} = average ultimate stress of the material as obtained from test;

f_{up} = characteristic ultimate tensile stress of the connected plate;

f_v = applied shear stress in the panel designed utilizing tension field action;

f_w = actual stress of weld at service load;

f_{wd} = design stress of weld at service load;

f_{wn} = nominal strength of fillet weld;

f_x = maximum longitudinal stress under combined axial force and bending;

f_y = characteristic yield stress;

$f_y(T)$ = yield stress of steel at T ºC;

$f_y(20)$ = yield stress of steel at 20 $^{\circ}$C ;

f_{yb} = characteristic yield stress of bolt;

f_{yf} = characteristic yield stress of flange;

f_{ym} = average yield stress as obtained from test;

f_{yp} = characteristic yield stress of connected plate;

f_{yq} = characteristic yield stress of stiffener material;

f_{yw} = characteristic yield stress of the web material;

G = modulus of rigidity for steel;

g = gauge length between centres of the holes perpendicular to the load direction, acceleration due to gravity;

h = depth of the section;

h_b = total height from the base to the floor level concerned;

h_c = height of the column;

h_e = effective thickness;

h_f = centre to centre distance of flanges;

h_i = thickness of fire protection material;

h_L = height of the lip;

h_s = storey height;

h_y = distance between shear centre of the two flanges of a cross section;

I = moment of inertia of the member about an axis perpendicular to the plane of the frame;

I_{fc} = moment of inertia of the compression flange of the beam about the axis parallel to the web;

I_f = moment of inertia of the tension flange of the beam about minor axis;

I_q = moment of inertia of a pair of stiffener about the centre of the web, or a single stiffener about the face of the web;

I_s = second moment of inertia;

I_{so} = second moment of inertia of the stiffener about the face of the element perpendicular to the web;

I_T = transformed moment of inertia of the one way system (in terms of equivalent steel, assuming the concrete flange of width equal to the spacing of the beam to be effective);

I_t = St. Venant’s torsion constant;

I_w = warping constant;

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I_y = moment of inertia about the minor axis of the cross section;

I_z = moment of inertia about the major axis of the cross section;

K_b = effective stiffness of the beam and column;

K_h = reduction factor to account for the high strength friction grip connection bolts in oversized and slotted holes Effective length of the member;

KL = appropriate effective slenderness ratio of the section;

KL/r = effective slenderness ratio of the section about the minor axis of the section;

KL/r = effective slenderness ratio of the section about the major axis of the section;

$\left(\frac{KL}{r}\right)_o$ = actual maximum effective slenderness ratio of the laced column;;

$\left(\frac{KL}{r}\right)_e$ = effective slenderness ratio of the laced column accounting for shear deformation;

K_v = shear buckling co-efficient;

K_w = warping restraint factor;

k = regression coefficient;

k_{sm} = exposed surface area to mass ratio;

K_y, K_z, K_{LT} = moment amplification factors;

L = actual length, unsupported length, length centre to centre distance of the intersecting members, cantilever length;

L_c = length of end connection in bolted and welded members, taken as the distance between outermost fasteners in the end connection, or the length of the end weld, measured along the length of the member;

L_{LT} = effective length for lateral torsional buckling;

L_m = maximum distance from the restraint to the compression flange at the plastic hinge to an adjacent restraint (limiting distance);

L_o = length between points of zero moment (inflection) in the span;

l = centre to centre length of the supporting member;

l_e = distance between prying force and bolt centre line;

l_g = grip length of bolts in a connection;

l_j = length of the joint;

l_s = length between points of lateral support to the compression flange in a beam;

l_v = distance from bolt centre line to the toe of fillet weld or to half the root radius for a rolled section;

l_w = length of weld;

M = bending moment;

M_a = applied bending moment;

M_{cr} = elastic critical moment corresponding to lateral torsional buckling of the beam;

M_d = design flexural strength;

M_{dv} = moment capacity of the section under high shear;

M_{dy} = design bending strength about the minor axis of the cross section;

M_{dz} = design bending strength about the major axis of the cross section;

M_{eff} = reduced effective moment;

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M_{fr} = reduced plastic moment capacity of the flange plate;

M_{fd} = design plastic resistance of the flange alone;

M_{nd} = design bending strength under combined axial force and uniaxial moment;

M_{ndy}, M_{ndz} = design bending strength under combined axial force and the respective uniaxial moment acting alone;

M_p = plastic moment capacity of the section;

M_{pb} = moment in the beam at the intersection of the beam and column centre lines;

M_{pc} = moments in the column above and below the beam surfaces;

M_{pd} = plastic design strength;

M_{pdf} = plastic design strength of flanges only;

M_q = applied moment on the stiffener;

M_s = Moment at service (working) load

M_{tf} = moment resistance of tension flange;

M_y = factored applied moment about the minor axis of the cross section;

M_{yq} = moment capacity of the stiffener based on its elastic modulus;

M_z = factored applied moment about the major axis of the cross section;

N = number of parallel planes of battens;

N_d = design strength in tension or in compression;

N_f = axial force in the flange;

N_{SC} = number of stress cycles;

n = number of bolts in the bolt group/critical section;

n_e = number of effective interfaces offering frictional resistance to slip;

n_n = number of shear planes with the threads intercepting the shear plane in the bolted connection;

n_s = number of shear planes without threads intercepting the shear plane in the bolted connection;

P = factored applied axial force;

P_{cr} = elastic buckling load

P_d = design axial compressive strength;

P_{dy}, P_{dz} = design compression strength as governed by flexural buckling about the respective axis;

P_e = elastic Euler buckling load;

P_{min} = minimum required strength for each flange splice;

P_r = required compressive strength;

P_s = actual compression at service load;

P_y = yield strength of the cross section under axial compression;

p = pitch length between centres of holes parallel to the direction of the load;

p_s = staggered pitch length along the direction of the load between lines of the bolt holes (*see* Fig. 5);

Q = prying force;

Q_a = accidental load (Action);

Q_c = characteristic loads (Action);

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Q_d = design load (Action);

Q_p = permanent loads (Action);

Q_v = variable loads (Action);

q = shear stress at service load;

R = ratio of the mean compressive stress in the web (equal to stress at mid depth) to yield stress of the web; reaction of the beam at support ;

R_d = design strength of the member at room temperature ;

R_i = net shear in bolt group at bolt 'i';

R_r = response reduction factor;

R_{tf} = flange shear resistance;

R = ultimate strength of the member at room temperature;

$_u$ = appropriate radius of gyration;

r = minimum radius of gyration of the individual element being laced together;

r_1 = ratio of the design action on the member under fire to the design capacity;

r_f = radius of gyration about the minor axis (v-v) of angle section;

r_{vv} = radius of gyration about the minor axis;

r_y = radius of gyration about the major axis;

r_z = minimum transverse distance between the centroid of the rivet or bolt group or weld group;

S = constant stress range;

S_c = design strength;

S_d = original cross sectional area of the test specimen;

S_o = spring stiffness;

S_p = ultimate strength;

S_u = anchorage length of tension field along the compression flange;

s_c = anchorage length of tension field along the tension flange;

s_t = actual stiffener spacing;

s_a = ratio of the mean compressive stress in the web (equal to stress at mid depth) to yield stress of the web; reaction of the beam at support;

T = temperature in degree Celsius; Factored tension;

T_b = applied tension in bolt;

T_{cf} = thickness of compression flange;

T_d = design strength under axial tension;

T_{dg} = yielding strength of gross section under axial tension;

T_{dn} = rupture strength of net section under axial tension;

T_{db} = design strength of bolt under axial tension, Block shear strength at end connection;

T_e = externally applied tension;

T_f = factored tension force of friction type bolt;

T_l = limiting temperature of the steel;

T_{nb} = nominal strength of bolt under axial tension;

T_{nd} = design tension capacity;

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T_{ndf} = design tension capacity of friction type bolt;

T_{nf} = nominal tensile strength of friction type bolt;

T_s = actual tension under service load;

t = thickness of element/angle, time in minutes;

t_f = thickness of flange;

t_p = thickness of plate;

t_{pk} = thickness of packing;

t_q = thickness of stiffener;

t_s = thickness of base slab;

t_t = effective throat thickness of welds;

t_w = thickness of web;

V = factored applied shear force;

V_b = shear in batten plate;

V_{bf} = factored frictional shear force in friction type connection;

V_{cr} = critical shear strength corresponding to web buckling;

V_d = design shear strength;

V_{db} = block shear strength;

V_n = nominal shear strength of bolt;

V_{nbf} = bearing capacity of bolt for friction type connection;

V_p = plastic shear resistance under pure shear;

V_n = nominal shear strength;

V_{npb} = nominal bearing strength of bolt;

V_{nsb} = nominal shear capacity of a bolt;

V_{nsf} = nominal shear capacity of bolt as governed by slip in friction type connection;

V_s = transverse shear at service load;

V_{sb} = factored shear force in the bolt;

V_{sd} = design shear capacity;

V_{sdf} = design shear strength in friction type bolt;

V_{sf} = factored design shear force of friction bolts;

V_t = applied transverse shear;

V_{tf} = shear resistance in tension field;

W = total load;

w = uniform pressure from below on the slab base due to axial compression under the factored load;

w_{tf} = width of tension field;

x_t = torsional index;

Z_e = elastic section modulus;

Z_{ec} = elastic section modulus of the member with respect to extreme compression fibre;

Z_{et} = elastic section modulus of the member with respect to extreme tension fibre;

Z_p = plastic section modulus;

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Z_v = contribution to the plastic section modulus of the total shear area of the cross section;

y_g = distance between point of application of the load and shear centre of the cross section;

y_s = co-ordinate of the shear centre in respect to centroid;

α = imperfection factor for buckling strength in columns and beams;

α_t = coefficient of thermal expansion;

β_M = ratio of smaller to the larger bending moment at the ends of a beam column;

β_{My}, β_{Mz} = equivalent uniform moment factor for flexural buckling for y-y and axes, respectively;

β_{MLT} = equivalent uniform moment factor for lateral torsional buckling;

χ = strength reduction factor to account for buckling under compression;

χ_m = strength reduction factor, χ, at $f_{ym;}$

χ_{LT} = strength reduction factor to account for lateral torsional buckling of beams;

δ = storey deflection;

δ_L = horizontal deflection of the bottom of storey due to combined gravity and notional load;

δ_p = load amplification factor;

δ_u = horizontal deflection of the top of storey due to combined gravity and notional load;

ϕ = inclination of the tension field stress in web;

γ = unit weight of steel;

γ_L = partial safety factor for load;

γ_m = partial safety factor for material;

γ_{m0} = partial safety factor against yield stress and buckling;

γ_{m1} = partial safety factor against ultimate stress;

γ_{mb} = Partial safety factor for bolted connection with bearing type bolts;

γ_{mf} = partial safety factor for bolted connection with high strength friction grip bolts;

γ_{fft} = partial safety factor for fatigue load;

γ_{mft} = partial safety factor for fatigue strength;

γ_{mv} = partial safety factor against shear failure;

γ_{mw} = partial safety factor for strength of weld;

ε = yield stress ratio, $(250 / f_y)^{½}$;

λ = Non dimensional slenderness ratio = $\sqrt{f_y (KL/r)^2 / \pi^2 E} = \sqrt{f_y / f_{cr}} = \sqrt{P_y / P_{cr}}$

λ_{cr} = elastic buckling load factor;

λ_e = equivalent slenderness ratio;

λ_{LT} = non-dimensional slenderness ratio in lateral bending;

λ_{scr} = elastic buckling load factor of each storey;

μ = poisson's ratio;

μ_c = correction factor;

μ_f = coefficient of friction (slip factor);

μ_r = capacity reduction factor;

θ = ratio of the rotation at the hinge point to the relative elastic rotation of the far end of the beam segment containing plastic hinge;

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ρ	=	unit mass of steel;
τ	=	actual shear stress range for the detail category;
τ_b	=	buckling shear stress;
τ_{ab}	=	permissible shear stress at the service load;
$\tau_{cr,e}$	=	elastic critical shear stress;;
τ_f	=	fatigue shear stress range;
$\tau_{f, Max}$	=	highest shear stress range;
τ_{fd}	=	design shear fatigue strength;
τ_{fn}	=	fatigue shear stress range at N_{SC} cycle for the detail category;
τ_v	=	actual shear stress at service load;
ψ	=	ratio of the moments at the ends of the laterally unsupported length of a beam; and
Γ	=	frame buckling load factor.

NOTE — The subscripts y, z denote the y-y and z-z axes of the section, respectively. For symmetrical sections, y-y denotes the minor principal axis whilst z-z denotes the major principal axis (*see* **8**).

ANNEX B

[*Clauses* 11.1.1(c) *and* 11.7.2.1(b)]

ANALYSIS AND DESIGN METHODS

B-1 ADVANCED STRUCTURAL ANALYSIS AND DESIGN

B-1.1 Analysis

For a frame, comprising members of compact section with full lateral restraint, an advanced structural analysis may be carried out, provided the analysis can be shown to accurately model the actual behaviour of that class of frames. The analysis shall take into account the relevant material properties, residual stresses, geometrical imperfections, reduction in stiffness due to axial compression, second-order effects, section strength and ductility, erection procedures and interaction with the foundations.

Advanced structural analysis for earthquake loads shall take into account provisions of **5** of Part C 'Strucutral design/Section 1 Loads and Load Effects' of this NBCS.

B-1.2 Design

For the strength limit state, it shall be sufficient to satisfy the section capacity requirements given under **15** for the members subjected to bending, under **14** for axial members, under **16** for combined forces and under **17** for connections. Effect of moment magnification given in **16**, instability given in **14** and lateral buckling given in **15** need not be considered while designing the member, since advanced analysis methods directly consider these.

B-2 SECOND ORDER ELASTIC ANALYSIS AND DESIGN

B-2.1 Analysis

In a second-order elastic analysis, the members shall be assumed to remain elastic, and changes in frame geometry under the design load and changes in the effective stiffness of the members due to axial forces shall be accounted for. In a frame where the elastic buckling load factor (λ_{cr}) of the frame as determined in accordance with **11.6** is greater than 5, the changes in the effective stiffness of the members due to axial forces may be neglected.

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B-2.2 Design Bending Moment

The design bending moment under factored load shall be taken as the maximum bending moment in the length of the member. It shall be determined either:

a) directly from the second-order analysis; or

b) approximately, if the member is divided into a sufficient number of elements, as the greatest of the element end bending moments, or;

c) by amplifying the calculated design bending moment, taken as the maximum bending moment along the length of a member as obtained by superposition of the simple beam bending moments determined by the analysis.

For a member with zero axial force or a member subject to axial tension, the factored design bending moment shall be calculated as the moment obtained from second-order analysis without any amplification.

For a member with a design axial compressive force as determined from the analysis, the factored design bending moment shall be calculated as follows:

$$M = \delta_b M_m$$

where

δ_b = moment amplification factor for a braced member determined in accordance with **16**.

B-3 FRAME INSTABILITY ANALYSIS

B-3.1 Analysis

Frame instability, as treated here, is related to the design of multi-storey rigid-jointed frames subject to side sway. The elastic critical load factor, λ_{cr} may be determined using the deflection method as given in **B-3.2** or any other recognized method. This is used to calculate the amplified sway moments for elastic designs and to check frame stability in plastic designs. The elastic critical load factor, λ_{cr} of a frame is the ratio by which each of the factored loads would have to be increased to cause elastic instability.

B-3.2 Deflection Method

An accurate method of analysis (ordinary linear elastic analysis) should be used to determine the horizontal deflections of the frame due to horizontal forces applied at each floor level, which is equal to the notional horizontal load in **11.3.6**. Allowance should be made for the degree of rigidity of the base as given in **B-3.2** in this deflection calculation.

The base stiffness should be determined by reference to **11.3.4**.
The elastic critical load factor, λ_{cr} is calculated as:

$$\lambda_{cr} = \frac{1}{200\,\phi_{s,Max}}$$

where

$\phi_{s,Max}$ = largest value of the sway index where, ϕ_s given by:

$$\phi_{si} = \frac{\delta_{ui} - \delta_{Li}}{h_i}$$

where

h_i = storey height;

δ_{ui} = horizontal deflection of the top of the storey due to the combined gravity and notional loads; and

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δ_{Li} = horizontal deflection of the bottom of the storey due to gravity and notional load.

B-3.3 Partial Sway Bracing

In any storey the stiffening effect of infill wall panels may be allowed for by introducing a diagonal strut in that storey of area *A*, given by:

$$A = \frac{k_3 \sum K_c}{h(h/b)}\left[1+(h/b)^2\right]^{3/2}$$

where

h = storey height;

b = width of the braced bay;

$\sum K_c$ = sum of the stiffness I/L, of the columns in that storey;

$k_3 = \dfrac{h^2 \sum S_p}{80E\sum K_C} \le 2$; and

$\sum S_p$ = Sum of spring stiffness (horizontal force per unit horizontal deflection of all the panels in that storey determined from:

$$S_p = \frac{0.6h/b}{\left[1+(h/b)^2\right]^2}\, t_p E_p$$

where

t_p = thickness of the wall panel; and

E_p = modulus of elasticity of the panel material.

ANNEX C

[*Clauses* 12.2.2.2 *and* 12.6.2]

DESIGN AGAINST FLOOR VIBRATION

C-1 GENERAL

Floor with longer spans of lighter construction and less inherent damping are vulnerable to vibrations under normal human activity. Natural frequency of the floor system corresponding to the lowest mode of vibration, damping characteristics, are important characteristics in floor vibration. Open web steel joists (trusses) or steel beams on the concrete deck may experience walking vibration problem. Fatigue, overloading of floor systems and vibrations due to rhythmic activities such as aerobic or dance classes are not within the scope of this annex.

C-2 ANNOYANCE CRITERIA

In the frequency range of 2 to 8 Hz in which people are most sensitive to vibration, the threshold level corresponds approximately to 0.5 percent *g*, where *g* is the acceleration due to gravity. Continuous vibration is generally more annoying than decaying vibration due to damping. Floor systems with the natural frequency less than 8 Hz in the case of floors supporting rhythmic activity and 5 Hz in the case of floors supporting normal human activity should be avoided. However, if it is unavoidable, recommendations of specialist literature may be followed.

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C-3 FLOOR FREQUENCY

The fundamental natural frequency can be estimated by assuming full composite action, even in non-composite construction. This frequency, f_1 for a simply supported one way system is given by:

$$f_1 = 156\sqrt{EI_T / WL^4}$$

where

E = modulus of elasticity of steel, in MPa;

I_T = transformed moment of inertia of the one way system (in terms of equivalent steel) assuming the concrete flange of width equal to the spacing of the beam to be effective, in mm^4;

L = span length, in mm; and

W = dead load of the one way joist, in N/mm.

If the one way joist system is supported by a flexible beam running perpendicular with the natural frequency f_2, the floor frequency may be reduced to f_r, given by:

$$\frac{1}{f_r^2} = \frac{1}{f_1^2} + \frac{1}{f_2^2}$$

C-4 DAMPING

The percentage of critical damping may be assumed approximately as given below:

Sl No.	*System*	*Critical Damping* Percent
(1)	(2)	(3)
i)	Fully composite construction	2
ii)	Bare steel beam and concrete deck	3 to 4
iii)	Floor with finishes, false ceiling, fire proofing, ducts, furniture	6
iv)	Partitions not located along a support or not spaced farther apart than 6 m and partitions oriented in orthogonal directions	Up to 12

C-5 ACCELERATION

The peak acceleration a_o, from heel impact for floors of spans greater than 7m and natural frequency f_1, less than 10 Hz may be calculated as:

$$a_o/g = 600\, f_r / W$$

where

W = total weight of floors plus contents over the span length and equivalent floor width (b), in N;

$b = 40t_s$ (20 t_s when overhang is only on one side of the beam);

t_s = equivalent thickness of the slab, averaging concrete in slab and ribs; and

g = acceleration due to gravity.

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ANNEX D

(*Clause* 14.2.2)

DETERMINATION OF EFFECTIVE LENGTH OF COLUMNS

D-1 METHOD FOR DETERMINING EFFECTIVE LENGTH OF COLUMNS IN FRAMES

In the absence of a more exact analysis, the effective length of columns in framed structures may be obtained by multiplying the actual length of the column between the centres of laterally supporting members (beams) given in Fig. 27 and Fig. 28 with the effective length factor *K*, calculated by using the equations given below, provided the connection between beam and column is rigid type:

a) *Non-sway frames* (*braced frame*) [(*see* **11.1.2** (a)]

A frame is designated as non-sway frame if the relative displacement between the two adjacent floors is restrained by bracings or shear walls (*see* **11.1.2**). The effective length factor, *K*, of column in non-sway frames is given by (*see* Fig. 27):

$$K = \frac{\left[1+0.145(\beta_1+\beta_2)-0.265\beta_1\beta_2\right]}{\left[2-0.364(\beta_1+\beta_2)-0.247\beta_1\beta_2\right]}$$

b) *Sway frames* (*moment resisting frames*) [*see* **11.1.2** (b)]

The effective length factor *K*, of column in sway frames is given by (*see* Fig. 28):

$$K = \left[\frac{1-0.2(\beta_1+\beta_2)-0.12\beta_1\beta_2}{1-0.8(\beta_1+\beta_2)+0.6\beta_1\beta_2}\right]^{0.5}$$

where

β_1, β_2 are given, $\beta = \frac{\Sigma K_c}{\Sigma K_c + \Sigma K_b}$

K_c, K_b = Effective flexural stiffness of the columns and beams meeting at the joint at the two ends of the columns and rigidly connected at the joints, and these are calculated by:

$$K_c \text{ and } K_b = C\,(I/L)$$

I = moment of inertia of the member about an axis perpendicular to the plan of the frame,

L = length of the member equal to centre to centre distance of the intersecting member, and

C = correction factor as shown in Table 33.

Table 33 Correction Factors for Effective Flexural Stiffness

(*Clause* D-1)

Sl No.	Far End Condition	Correction Factor, C	
		Braced Frame	Unbraced Frame
(1)	(2)	(3)	(4)
i)	Pinned	$0.67(1-\bar{n})$	$0.75(1-\bar{n})$
ii)	Rigidly connected to column	$0.5(1-\bar{n})$	$1.5\,(1-0.2\,\bar{n})$
iii)	Fixed	$1.0(1-0.4\,\bar{n})$	$1.0\,(1-0.4\,\bar{n})$

NOTE — $\bar{n} = \frac{P}{P_e}$

where

P_e = elastic buckling load, and

P = applied load.

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D-2 METHOD FOR DETERMINING EFFECTIVE LENGTH FOR STEPPED COLUMNS (*see* 14.2.2)

D-2.1 Single Stepped Columns

Effective length in the plane of stepping (bending about axis *z-z*) for bottom and top parts for single stepped column shall be taken as given in Table 34.

NOTE — The provisions of D-2.1 are applicable to intermediate columns as well with stepping on either side, provided appropriate values of I_1 and I_2 are taken.

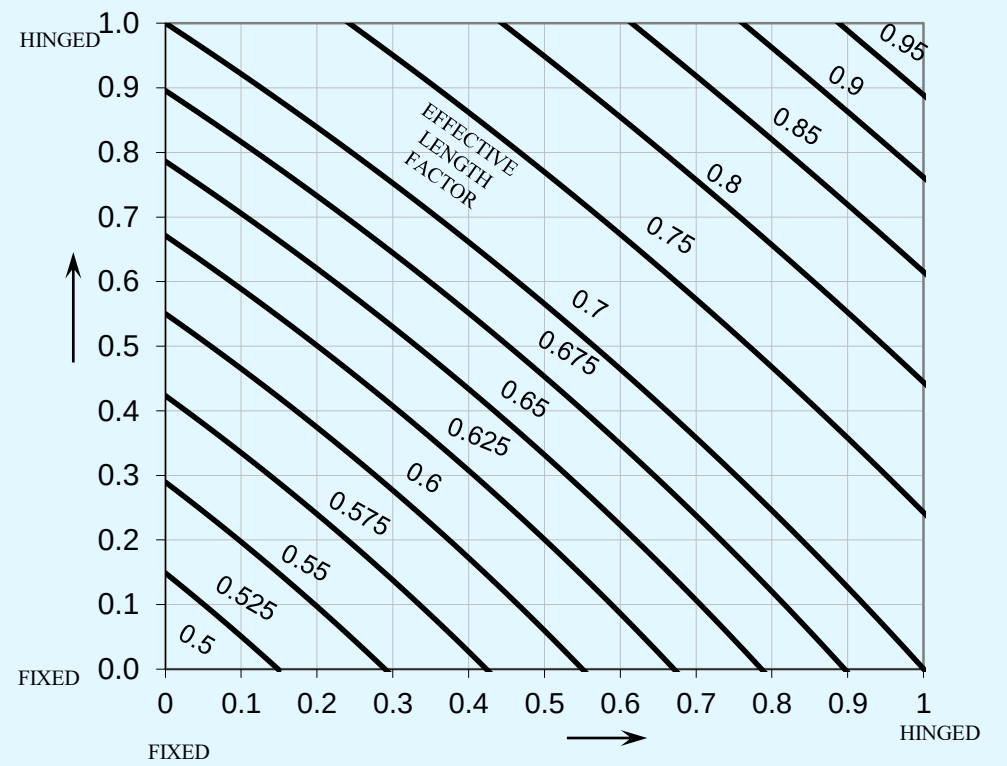

FIG. 27 COLUMN EFFECTIVE LENGTH FACTOR — NON SWAY FRAME

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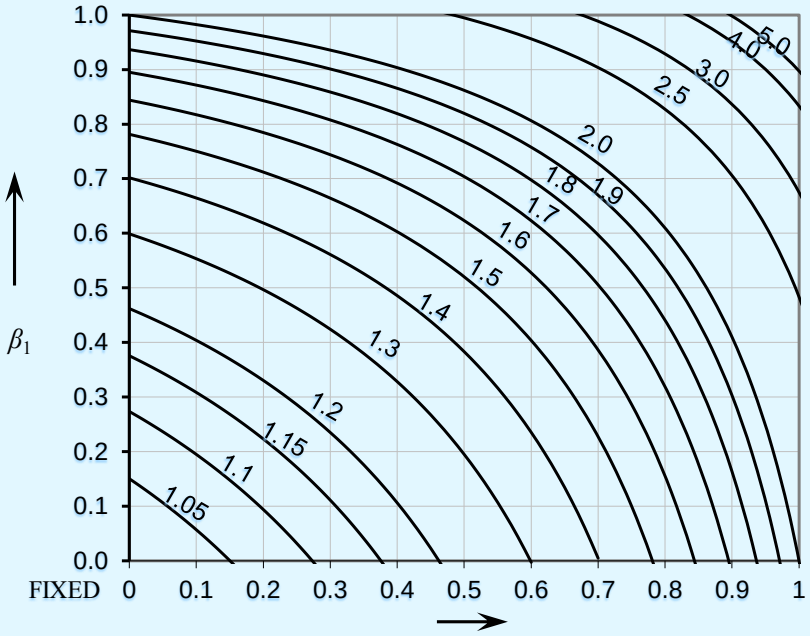


FIG. 28 COLUMN EFFECTIVE LENGTH FACTOR — SWAY FRAME

Table 34 Effective Length of Singled Stepped Columns

(*Clause* D-2.1)

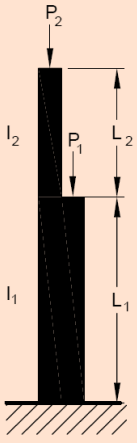
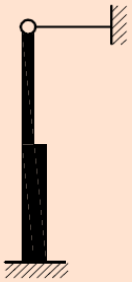
Sl No.	Degree of End Restraint	Sketch	Effective Length Coefficients	Column Parameters for all Cases
(1)	(2)	(3)	(4)	(5)
i)	Effectively held in position and restrained against rotation at both ends		$K_1 = \sqrt{\frac{K_{12}^2 + K_{11}^2(\alpha - 1)}{\alpha}}$ $K_2 = \frac{K_1}{C_1} \leq 3$ where K_{12} and K_{11} are to be taken as per Table 35	
ii)	Effectively held in position at both ends and restrained against rotation at bottom end only		$K_1 = \sqrt{\frac{K_{12}^2 + K_{11}^2(\alpha - 1)}{\alpha}}$ $K_2 = \frac{K_1}{C_1} \leq 3$ where K_{12} and K_{11} are to be taken as per Table 36	

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Table 34 (*Concluded*)

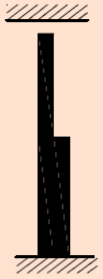
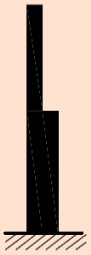
Sl No.	Degree of End Restraint	Sketch	Effective Length Coefficients	Column Parameters for all Cases
(1)	(2)	(3)	(4)	(5)
iii)	Effectively held in position and restrained against rotation at bottom end, and top end held against rotation but not held in position		K_1 to be taken as per Table 37 $K_2 = \frac{K_1}{C_1} \leq 3$	$\alpha = \frac{P_1 + P_2}{P_2}$ $C_1 = \frac{L_2}{L_1}\sqrt{\frac{I_1}{I_2 \alpha}}$ $\frac{i_2}{i_1} = \frac{I_2}{L_2} \times \frac{L_1}{I_1}$ Effective length of bottom part of column in plane of stepping = $K_1 L_1$
iv)	Effectively held in position and restrained against rotation at bottom end, and top end neither held against rotation nor held in position		K_1 to be taken as per Table 38 $K_2 = \frac{K_1}{C_1} \leq 3$	Effective length of top part of column in plane of stepping = $K_2 L_2$

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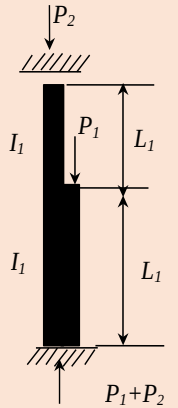
Table 35 Coefficients of Effective Lengths K_{12} And K_{11} for Columns with Both Ends Effectively Held in Position and Restrained Against Rotation

(*Table 34*)

	Coefficients K_{12} and K_{11} for L_2/L_1 Equal to														
I_2/I_1	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
	Coefficient K_{12} ($P_1 = 0$)														
0.05	0.74	0.94	1.38	1.60	1.87	2.07	2.23	2.39	2.52	2.67	3.03	3.44	3.85	4.34	4.77
0.1	0.67	0.76	1.00	1.20	1.42	1.61	1.78	1.92	2.04	2.20	2.40	2.60	2.86	3.18	3.41
0.2	0.64	0.70	0.79	0.93	1.07	1.23	1.41	1.50	1.60	1.72	1.92	2.11	2.28	2.45	2.64
0.3	0.62	0.68	0.74	0.85	0.95	1.06	1.18	1.28	1.39	1.48	1.67	1.82	1.96	2.12	2.20
0.4	0.60	0.66	0.71	0.77	0.82	0.93	0.99	1.08	1.17	1.23	1.39	1.53	1.66	1.79	1.92
0.5	0.59	0.65	0.70	0.77	0.82	0.93	0.99	1.08	1.17	1.23	1.39	1.53	1.66	1.79	1.92
1.0	0.55	0.60	0.65	0.70	0.75	0.80	0.85	0.90	0.95	1.00	1.10	1.20	1.30	1.40	1.50
	Coefficient K_{11} ($P_2 = 0$)														
0.05	0.65	0.67	0.71	0.85	1.01	1.17	1.31	1.41	1.50	1.57	1.67	1.74	1.78	1.82	1.86
0.1	0.64	0.65	0.65	0.65	0.78	0.92	1.05	1.15	1.25	1.33	1.45	1.55	1.62	1.68	1.71
0.2	0.62	0.64	0.65	0.65	0.66	0.73	0.83	0.92	1.01	1.09	1.23	1.33	1.41	1.48	1.54
0.3	0.60	0.63	0.64	0.65	0.66	0.67	0.73	0.81	0.89	0.94	1.09	1.20	1.28	1.35	1.41
0.4	0.58	0.63	0.63	0.64	0.64	0.66	0.68	0.75	0.82	0.88	1.01	1.10	1.19	1.26	1.32
0.5	0.57	0.61	0.63	0.64	0.64	0.65	0.68	0.72	0.77	0.83	0.94	1.04	1.12	1.19	1.25
1.0	0.55	0.58	0.60	0.61	0.62	0.63	0.65	0.67	0.70	0.73	0.80	0.88	0.93	1.01	1.05



NOTE — Intermediate value may be obtained by interpolation

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Table 36 Coefficients of Effective Lengths K_{12} And K_{11} for Columns with Both Ends Effectively Held in Position and Restrained Against Rotation at Bottom End Only

(*Table* 34)

I_2/I_1	Coefficients K_{12} and K_{11} for L_2/L_1 Equal to														
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
	Coefficient K_{12} (P_1 = 0)														
0.05	0.97	1.66	2.10	2.43	2.72	2.92	3.08	3.24	3.48	3.73	4.22	4.85	5.36	6.00	6.58
0.1	0.83	1.21	1.57	1.95	2.14	2.33	2.46	2.60	2.76	2.91	3.28	3.61	4.03	4.43	4.85
0.3	0.78	0.90	1.09	1.27	1.44	1.60	1.74	1.86	1.98	2.11	2.35	2.51	2.76	2.99	3.25
0.5	0.78	0.86	0.99	1.10	1.22	1.35	1.47	1.57	1.67	1.76	1.96	2.15	2.34	2.50	2.76
1.0	0.78	0.85	0.92	0.99	1.06	1.13	1.20	1.27	1.34	1.41	1.54	1.68	1.82	1.97	2.10
	Coefficient K_{11} (P_2 = 0)														
0.05	0.67	0.67	0.82	1.16	1.35	1.48	1.58	1.65	1.69	1.74	1.81	1.84	1.86	1.88	1.90
0.1	0.67	0.67	0.73	0.93	1.11	1.25	1.36	1.45	1.52	1.57	1.66	1.72	1.77	1.80	1.82
0.3	0.67	0.67	0.67	0.71	0.80	0.90	0.99	1.08	1.15	1.22	1.33	1.41	1.48	1.54	1.59
0.5	0.67	0.67	0.67	0.69	0.73	0.81	0.87	0.94	1.01	1.07	1.17	1.26	1.33	1.39	1.44
1.0	0.67	0.67	0.67	0.68	0.71	0.74	0.78	0.82	0.87	0.91	0.99	1.07	1.13	1.19	1.24

NOTE — Intermediate value may be obtained by interpolation.

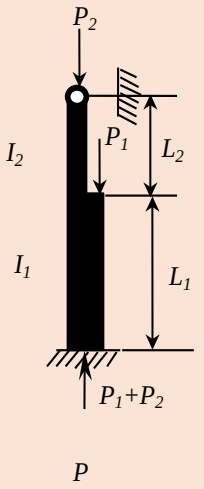

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Table 37 Coefficients of Effective Lengths K_1 for Columns Effectively Held in Position and Restrained Against Rotation at Bottom End and Top End Held Against Rotation but not Held in Position

(*Table 34*)

C_1	Coefficients K_1 for I_2 / I_1 Equal to														
	0.1	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0	2.0	1.86	1.7	1.67	1.6	1.55	1.50	1.4	1.43	1.4	1.37	1.3	1.1	1.10	1.0
0.5	2.0	1.90	1.8	1.74	1.6	1.65	1.61	1.5	1.55	1.5	---	---	---	----	----
1.0	2.0	2.00	2.0	2.00	2.0	2.00	----	----	----	----	----	----	----	----	----
1.5	2.0	2.25	2.3	2.48	----	----	----	----	----	----	----	----	----	----	----
2.0	2.0	2.66	2.91	----	----	----	----	----	----	----	----	----	----	----	----
2.5	2.5	3.17	3.50	----	----	----	----	----	----	----	----	----	----	----	----
3.0	3.0	3.70	4.12	----	----	----	----	---	----	----	----	----	----	----	----

NOTE — Intermediate values may be obtained by interpolation.

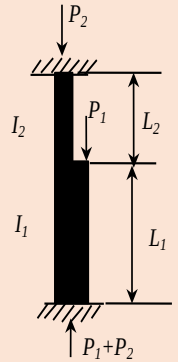
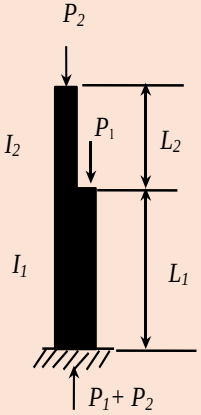


Table 38 Coefficients of Effective Lengths K_1 for Columns with Top Ends Free and Bottom End Effectively Held in Position and Restrained Against Rotation

(*Table 34*)

C_1	Coefficients K_1 for I_2/I_1 Equal to														
	0.1	0.2	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0	2.5	5.0	10	20
0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
0.5	2.0	2.14	2.24	2.36	2.47	2.57	2.67	2.76	2.85	2.94	3.02	---	---	---	----
1.0	2.0	2.73	3.13	3.44	3.74	4.00	----	----	----	----	----	----	----	----	----
1.5	3.0	3.77	4.35	4.86	----	----	----	----	----	----	----	----	----	----	----
2.0	4.0	4.90	5.67	----	----	----	----	----	----	----	----	----	----	----	----
2.5	5.0	6.08	7.00	----	----	----	----	----	----	----	----	----	----	----	----
3.0	6.0	7.25	----	----	----	----	----	----	----	----	----	----	-----	----	----

NOTE — Intermediate values may be obtained by interpolation.



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D-3 Effective Length for Double Stepped Columns

Effective lengths in the plane of steppings (bending about axis *z-z*) for bottom, middle and top parts for a double stepped column shall be taken as follows (*see also* Fig. 29):

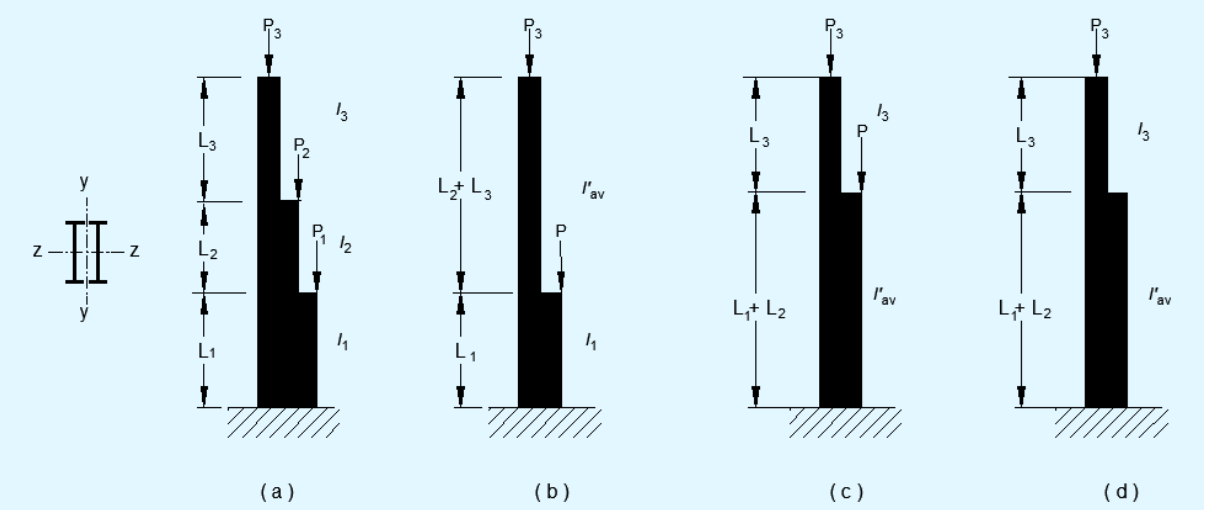

FIG. 29 EFFECTIVE LENGTH OF DOUBLE STEPPED COLUMNS

Coefficient K_1 for effective length of bottom part of double stepped column shall be taken from the formula:

$$K_1 = \sqrt{\frac{t_1 K_1^{\,2} + \left(t_2 K_2^{\,2} + K_3^{\,2}\right) \times \left(1 + n_2\right)^2 \times \frac{I_1}{I'_{av}}}{1 + t_1 + t_2}}$$

where

K_1, K_2, and K_3 are taken from Table 39.

$$t_1 = \frac{P_1}{P_3}, \qquad t_2 = \frac{P_2}{P_3},$$

I'_{av} = average value of moment of inertia for the lower and middle parts

$$= \frac{I_1 L_1 + I_2 L_2}{L_1 + L_2}$$

I'_{av} = average value of moment of inertia for the middle and top parts

$$= \frac{I_2 L_2 + I_3 L_3}{L_2 + L_3}$$

Value of coefficient K_2 for middle part of column is given by formula:

$$K_2 = \frac{K_1}{C_2}, \quad \text{and}$$

Value of coefficient K_3 for the top part of the column is given by:

$$K_3 = \frac{K_1}{C_3} \leq 3$$

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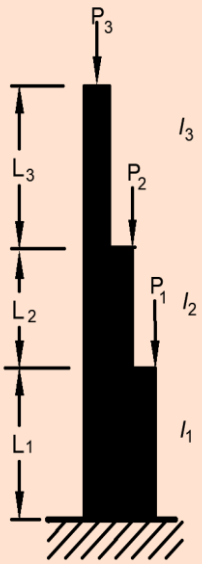
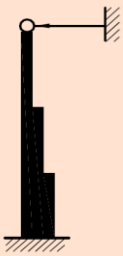
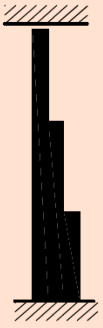
where

$$C_2 = \frac{L_2}{L_1}\sqrt{\frac{I_1(P_2+P_3)}{I_2(P_1+P_2+P_3)}}\ ,\ C_3 = \frac{L_3}{L_1}\sqrt{\frac{I_1 P_3}{I_3(P_1+P_2+P_3)}}$$

NOTE — The provisions of D-3 are applicable to intermediate columns as well with steppings on either side, provided appropriate values of I_1, I_2 and I_3 are taken.

Table 39 Values of K_1, K_2 and K_3

(*Clause* D-3)

Sl No.	Degree of End Restraint	Sketch	K_1	K_2	K_3	Column Parameters for all Cases
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	Effectively held in position and restrained against rotation at both ends		$K_1 = K_{11}$ where K_{11} is taken from Table 35	$K_2 = K_{11}$ where K_{11} is taken from Table 35	$K_3 = K_{12}$ where K_{12} is taken from Table 35	
ii)	Effectively held in position at both ends and restrained against rotation at bottom end only		$K_1 = K_{11}$ where K_{11} is taken from Table 36	$K_2 = K_{11}$ where K_{11} is taken from Table 36	$K_3 = K_{12}$ where K_{12} is taken from Table 36	
iii)	Effectively held in position and restrained against rotation at bottom end, and top end held against rotation but not held in position		$K_1 = K_1$ where K_1 is taken from Table 37 with $C_1=0$	$K_2 = K_1$ where K_1 is taken from Table 37 with $C_1=0$	$K_3 = K_1$ where K_1 is taken from Table 37 with $C_1=\frac{L_3}{L_1+L_2}\sqrt{\frac{I_{av}}{I_3}}$	
iv)	Effectively held in position and restrained against rotation at bottom end, and top end neither held against rotation nor against translation		$K_1 = 2$	$K_2 = 2$	$K_3 = K_1$ where K_1 is taken from Table 38 with $C_1=\frac{L_3}{L_1+L_2}\sqrt{\frac{I_{av}}{I_3}}$	

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ANNEX E

(*Clause* 15.2.2.1)

ELASTIC LATERAL TORSIONAL BUCKLING

E-1 ELASTIC CRITICAL MOMENT

E-1.1 General

The elastic critical moment is affected by the following:

a) Moment gradient in the unsupported length;

b) Boundary conditions at the lateral support points;

c) Non-symmetric and non-prismatic nature of the member; and

d) Location of transverse load with respect to shear centre.

The boundary conditions at the lateral supports have two components:

1) *Torsional restraint* — where the cross section is prevented from rotation about the shear centre; and

2) *Warping restraint* — where the flanges are prevented from rotating in their own plane about an axis perpendicular to the flange.

The elastic critical moment corresponding to lateral torsional buckling of a doubly symmetric prismatic beam subjected to uniform moment in the unsupported length and torsionally restraining lateral supports is given by:

$$M_{cr} = \frac{\pi^2 E I_y}{(L_{LT})^2}\left[\frac{I_w}{I_y} + \frac{G\, I_t\, (L_{LT})^2}{\pi^2 E\, I_y}\right]^{0.5}$$

where

I_y, I_w, I_t = moment of inertia about the minor axis, warping constant and St.Venant's torsion constant of the cross section, respectively (*see* accepted standards [C-6A(36)] for properties of sections);

G = modulus of rigidity; and

L_{LT} = effective length against lateral torsional buckling (*see* **15.3**).

This equation in simplified form for I-section has been presented in **15.2.2.1**.

While the simplified equation is generally on the safe side, there are many situations where this may be very conservative. More accurate calculation of the elastic critical moment for general case of unsymmetrical sections, tapered members, loading away from shear centre and beams with moment gradient can be obtained from specialist literature, by using an appropriate computer program or equations given below.

E-1.2 Elastic Critical Moment of a Section Symmetrical About Minor Axis

In case of a beam which is symmetrical only about the minor axis, and bending about major axis, the elastic critical moment for lateral torsional buckling is given by the general equation below,

$$M_{cr} = c_1 \frac{\pi^2 E I_y}{(L_{LT})^2}\left\{\left[\left(\frac{K}{K_w}\right)^2 \frac{I_w}{I_y} + \frac{G\, I_t\, (L_{LT})^2}{\pi^2 E\, I_y} + (c_2 y_g - c_3 y_j)^2\right]^{0.5} - (c_2 y_g - c_3 y_j)\right\}$$

where

c_1, c_2, c_3 = factors depending upon the loading and end restraint conditions (*see* Table 40);

K = effective length factors of the unsupported length accounting for boundary conditions at the end lateral supports. The effective length factor K varies from 0.5 for complete

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restraint against rotation about weak axis to 1.0 for free rotate about weak axis, with 0.7 for the case of one end fixed and other end free. It is analogous to the effective length factors for compression members with end rotational restraint;

K_w = warping restraint factor. Unless special provisions to restrain warping of the section at the end lateral supports are made, K_w should be taken as 1.0;

y_g = y distance between the point of application of the load and the shear centre of the cross section and is positive when the load is acting towards the shear centre from the point of application;

y_j = $y_s - 0.5\int_A \left(z^2 + y^2\right)^2 y dA / I_z$;

y_s = coordinate of the shear centre with respect to centroid, positive when the shear centre is on the compression side of the centroid; and

y, z = coordinates of the elemental area with respect to centroid of the section.

y_j can be calculated by using the following approximation:

a) *Plain flanges*:

1) $y_j = 0.8\ (\ 2\beta_f - 1)\ h_y/2.0$ (when $\beta_f > 0.5$)

2) $y_j = 1.0\ (\ 2\beta_f - 1)\ h_y/2.0$ (when $\beta_f \leq 0.5$)

b) *Lipped flanges*:

1) $y_j = 0.8\ (\ 2\beta_f - 1)\ (1+ h_L/h)\ h_y/2$ (when $\beta_f > 0.5$)

2) $y_j = (\ 2\beta_f - 1)\ (1+ h_L/h)\ h_y/2$ (when $\beta_f \leq 0.5$)

where

h_L = height of the lip,

h = overall height of the section,

h_y = distance between shear centre of the two flanges of the cross section, and

$\beta_f = I_{fc}/(I_{fc} + I_{ft})$ where I_{fc}, I_{ft} are the moment of inertia of the compression and tension flanges, respectively, about the minor axis of the entire section.

I_t = St. Venant's Torsion constant, given by:

$I_t = \sum b_i t_i^3 / 3$ (for open section)

$= 4A_e^2 / \sum (b/t)$ (for hollow section)

where

A_e = area enclosed by the section, and

b, t = breadth and thickness of the elements of the section, respectively.

I_w = The warping constant, given by:

$(1-\beta_f)\ \beta_f\ I_y\ h_y^2$ (for I-sections mono-symmetric about weak axis)= 0 (for angle, Tee, narrow rectangle section and approximately for hollow sections)

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Table 40 Constants c_1, c_2, and c_3

(*Clause* E-1.2)

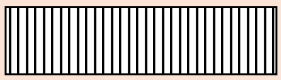
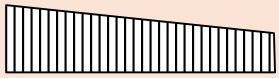
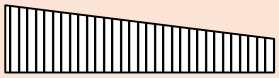
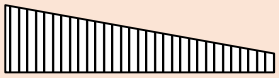
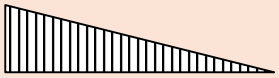
Loading and Support Conditions	Bending Moment Diagram	Value of *K*	Constants		
			c_1	c_2	c_3
(1)	(2)	(3)	(4)	(5)	(6)
	ψ = +1 	1.0 0.7 0.5	1.000 1.000 1.000	---	1.000 1.113 1.144
	ψ= + ¾ 	1.0 0.7 0.5	1.141 1.270 1.305	---	0.998 1.565 2.283
	ψ = + ½ 	1.0 0.7 0.5	1.323 1.473 1.514	---	0.992 1.556 2.271
	ψ = + ¼ 	1.0 0.7 0.5	1.563 1.739 1.788	---	0.977 1.531 2.235
	ψ = 0 	1.0 0.7 0.5	1.879 2.092 2.150	---	0.939 1.473 2.150
	ψ = - ¼	1.0 0.7 0.5	2.281 2.538 2.609	---	0.855 1.340 1.957
	ψ = - ½	1.0 0.7 0.5	2.704 3.009 3.093	---	0.676 1.059 1.546
	ψ = - ¾	1.0 0.7 0.5	2.927 3.009 3.093	---	0.366 0.575 0.837

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Table 40 (*Concluded*)

Loading and Support Conditions	Bending Moment Diagram	Value of *K*	Constants		
			c_1	c_2	c_3
(1)	(2)	(3)	(4)	(5)	(6)
	ψ = - 1	1.0	2.752	---	0.000
		0.7	3.063		0.000
		0.5	3.149		0.000
W		1.0	1.132	0.459	0.525
		0.5	0.972	0.304	0.980
W		1.0	1.285	1.562	0.753
		0.5	0.712	0.652	1.070
F		1.0	1.365	0.553	1.730
		0.5	1.070	0.432	3.050
F		1.0	1.565	1.267	2.640
		0.5	0.938	0.715	4.800
℄ F F L/4 L/4 L/4 L/4		1.0	1.046	0.430	1.120
		0.5	1.010	0.410	1.890

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ANNEX F

(*Clauses* 11.2.1.2, 15.6.4.2, 17.6.1, 18.10.4.1 *and* 18.11.2.1)

CONNECTIONS

F-1 GENERAL

The requirement for the design of splice and beam to column connection as well as recommendation for their design shall be as given below.

F-2 BEAM SPLICES

F-2.1 For rolled section beam splices located away from the point of maximum moment, it may be assumed that the flange splice carries all the moment and the web splice carries the shear (*see* Fig. 30). However, in the case of a deep girder, the total moment may be divided between the flange and the web in accordance with the stress distribution. The web connection should then be designed to resist its share of moment and shear. Even web splice is designed to carry only shear force, the moment about the centroid of the bolt group on either side of the splice should be designed for moment due to eccentricity.

F-2.2 Flange joints should preferably not be located at points of maximum stress. Where splice plates are used, their area shall not be less than 5 percent in excess of the area of the flange element spliced; and their centre of gravity shall coincide, as nearly as possible with that of the element spliced. There shall be enough fasteners on each side of the splice to develop the load in the element spliced plus 5 percent but in no case should the strength developed be less than 50 percent of the effective strength of the material spliced. Wherever possible in welded construction, flange plates shall be joined by complete penetration butt welds. These butt welds shall develop the full strength of the plates. Whenever the flange width or thickness changes at the splice location, gradual transition shall be made in the width/thickness of the larger flange.

F-2.3 When beam splice is located at the point of inflection of a continuous beam, the flange splicing requirement given above may be relaxed appropriately.

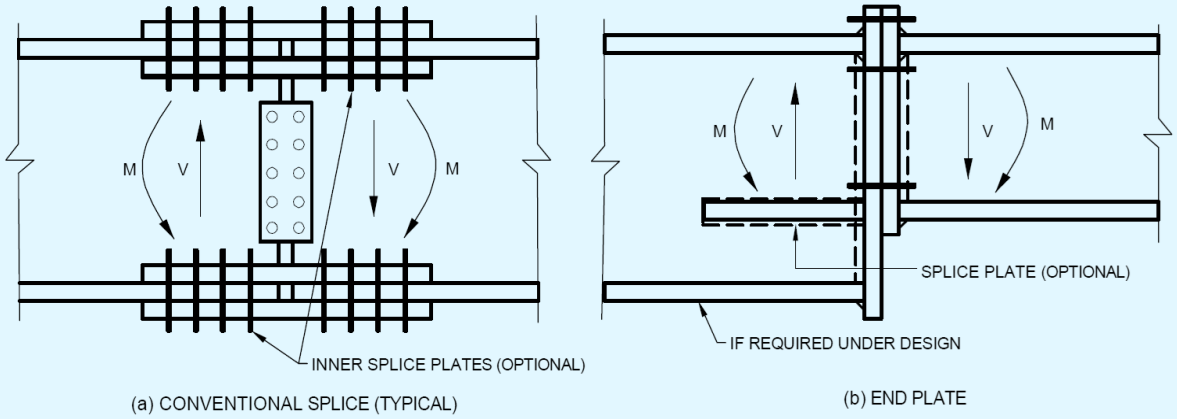

FIG. 30 BEAM SPLICES

F-3 COLUMN SPLICE

F-3.1 Where the ends of compression members are faced for bearing over the whole area, they shall be spliced to hold the connected parts aligned. The ends of compression members faced for bearing shall invariably be machined to ensure perfect contact of surfaces in bearing (*see* Fig. 31).

F-3.2 Where such members are not faced for complete bearing the splices shall be designed to transmit all the forces to which the members is subjected at the splice location.

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F-3.3 Wherever possible, splices shall be proportioned and arranged so that centroidal axis of the splice coincides as nearly as possible with the centroidal axes of the members joined, in order to avoid eccentricity; but where eccentricity is present in the joint, the resulting stress considering eccentricity shall be provided for.

F-3.4 If a column flange is subjected to significant tension or if the faces are not prepared for bearing, or if full continuity is required without slip, only HSFG bolts shall be used.

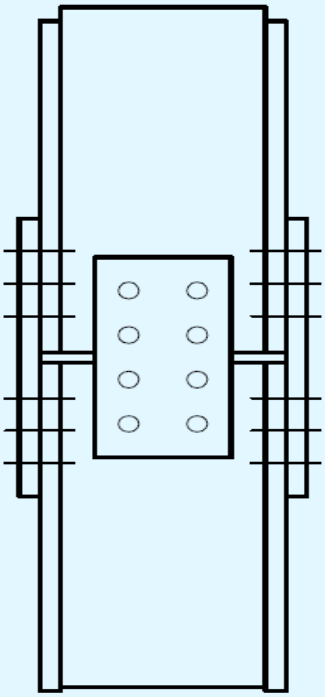

FIG. 31 COLUMN SPLICE (TYPICAL)

F-4 BEAM-TO-COLUMN CONNECTIONS

F-4.1 Simple Connections

Simple connections are assumed to transfer only shear at some nominal eccentricity and typically used in frames up to about five stories in height, where strength rather than stiffness govern the design. In such frames separate lateral load resisting system is to be provided in the form of bracings or shear walls. The connections shown in Fig. 32 (A), Fig. 32 (B), Fig. 32 (C) and Fig. 32 (D) can be assumed as simple connections in framed analysis and need to be checked only for the transfer of shear from beam to column.

F-4.2 Rigid Connections

In high-rise and slender structures, stiffness requirements may warrant the use of rigid connections. Rigid connections transfer significant moments to the columns and are assumed to undergo negligible deformations at the joint. These are necessary in sway frames for stability and also contribute in resisting lateral loads. The connections shown in Fig. 32 (E), Fig. 32 (F), Fig. 32 (G) can be assumed as rigid connection in frame analysis and need to be checked for both shear and moment transfer from beam to the column. Fully welded connections can also be considered as rigid beam to column connections.

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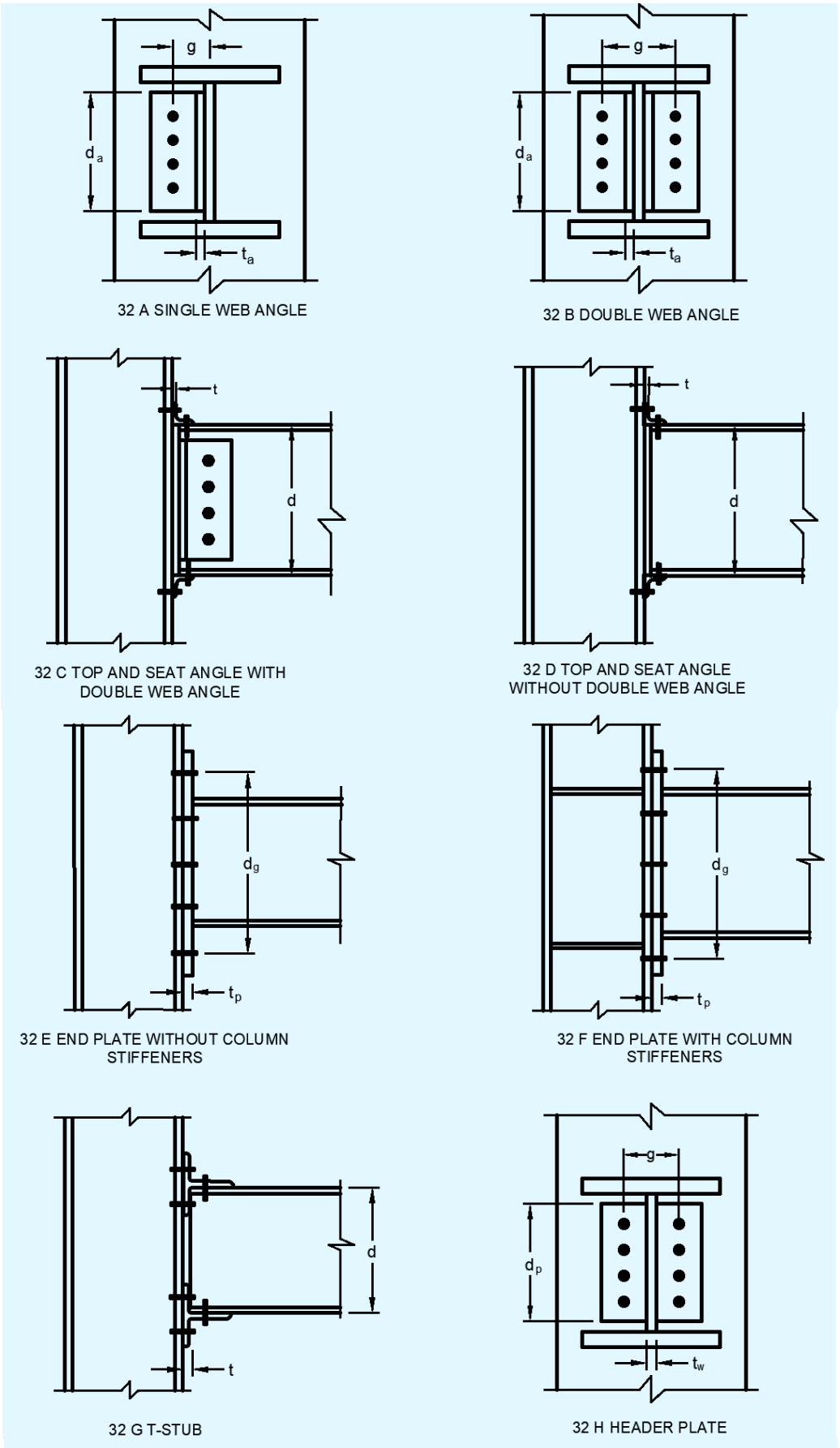

FIG. 32 SIZE PARAMETER FOR VARIOUS TYPES OF CONNECTION

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F-4.3 Semi-Rigid Connections

Semi-rigid connections fall between the two types mentioned above. The fact is that simple connections do have some degree of rotational rigidity as in the semi-rigid connections. Similarly, rigid connections do experience some degree of joint deformation and this can be utilized to reduce the joint design moments.

The moment-rotation relationship of the connections have to be determined based on experiments conducted for the specific design or based on the relationship derived from tests, presented in specialist literature. The simplest method of analysis will be to idealize the connection as an equivalent rotational spring with either a bilinear or non-linear moment-rotation characteristics. The classification proposed by Bjorhovde combined with the Frye-Morris model can be used with convenience to model semi-rigid connections, as given in the next section.

F-4.3.1 *Connection Classification*

Connections are classified according to their ultimate strength or in terms of their initial elastic stiffness and Bjorhovde's classification. It is based on the non-dimensional moment parameter ($m^1 = M_u / M_{pb}$) and the non-dimensional rotation parameter ($\theta^1 = \theta_r/\theta_p$), where θ_p is the plastic rotation. The Bjorhovde's classification is based on a reference length of the beam equal to 5 times the depth of the beam. The limits used for connection classification are shown in Table 41 and are graphically represented in Fig. 33.

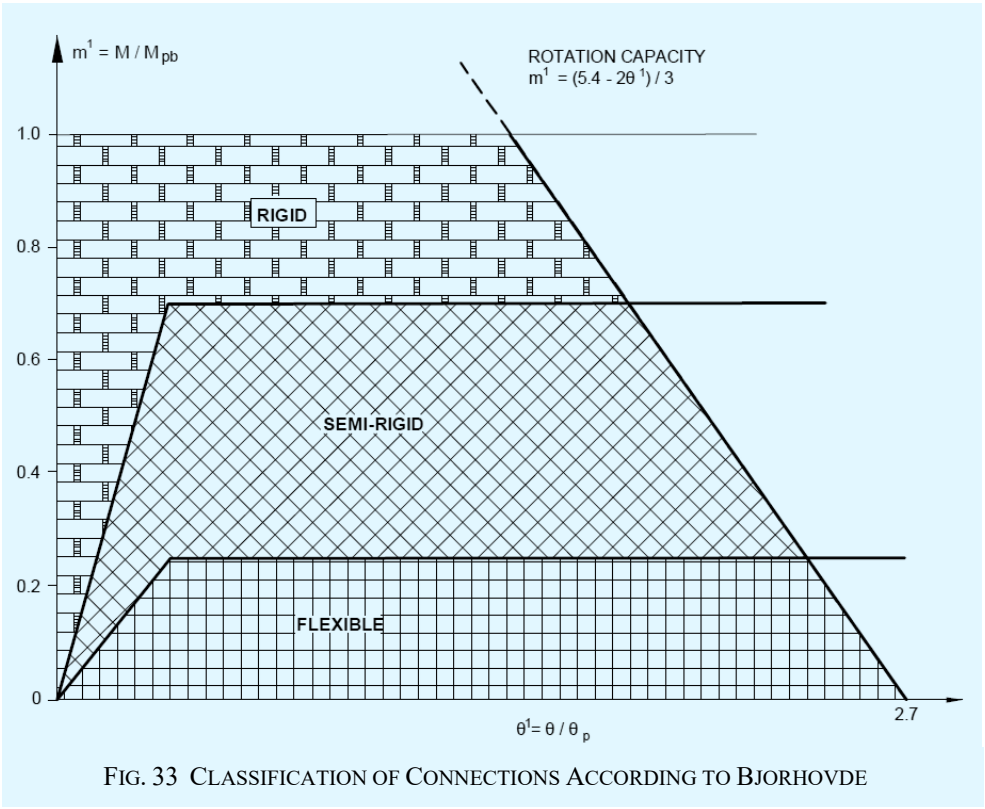

FIG. 33 CLASSIFICATION OF CONNECTIONS ACCORDING TO BJORHOVDE

F-4.3.2 *Connection Models*

Frye-Morris has derived the following polynomial model for the moment curvature relationship of semi-rigid connections:

$$\theta_r = C_1(KM)^1 + C_2 (KM)^3 + C_3 (KM)^5$$

where

M = moment at the joint, in kN.m;

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K = standardization parameter which depend on the connection type and geometry; and

C_1, C_2, C_3 = curve fitting constants Table 42 shows the curve fitting constants and standardization constants for Frye-Morris Model [All size parameters in the table are in mm, (*see* Fig. 32)].

Table 41 Connection Classification Limits

(*Clause* F-4.3.1)

Sl No.	Nature of the Connection	In Terms of Strength	In Terms of Stiffness
(1)	(2)	(3)	(4)
i)	Rigid connection	$m^1 \geq 0.7$	$m^1 \geq 2.5\theta^1$
ii)	Semi-rigid connection	$0.7 > m^1 > 0.2$	$2.5\theta^1 > m^1 > 0.5\theta^1$
iii)	Flexible connection	$m^1 \leq 0.2$	$m^1 \leq 0.5\theta^1$

Table 42 Connection Constants in Frye-Morris Model

(*Clause* F-4.3.2)

Sl No.	Type	Connection Type	Curve-Fitting Constants	Standardization Constants
(1)	(2)	(3)	(4)	(5)
i)	A	Single web angle connection	$C_1 = 1.91 \times 10^4$ $C_2 = 1.30 \times 10^{11}$ $C_3 = 2.70 \times 10^{17}$	$K = d_a^{-2.4} t_c^{-1.81} g^{0.15}$
ii)	B	Double web angle connection	$C_1 = 1.64 \times 10^3$ $C_2 = 1.03 \times 10^{14}$ $C_3 = 8.18 \times 10^{25}$	$K = d_a^{-2.4} t_c^{-1.81} g^{0.15}$
iii)	C	Top and seat angle connection with double web angle	$C_1 = 2.24 \times 10^{-1}$ $C_2 = 1.86 \times 10^4$ $C_3 = 3.23 \times 10^8$	$K = d^{-1.287} t_a^{-1.128} t_c^{-0.415}$ $l_a^{-0.694} (g - 0.5 d_b)^{1.35}$
iv)	D	Top and seat angle connection without double web angle	$C_1 = 1.63 \times 10^3$ $C_2 = 7.25 \times 10^{14}$ $C_3 = 3.31 \times 10^{23}$	$K = d^{-1.5} t_a^{-0.5} l_a^{-0.7} d_b^{-1.1}$
v)	E	End plate connection without column stiffeners	$C_1 = 1.78 \times 10^4$ $C_2 = -9.55 \times 10^{16}$ $C_3 = 5.54 \times 10^{29}$	$K = d_g^{-2.4} t_p^{-0.4} t_f^{-1.5}$
vi)	F	End plate connection with column stiffeners	$C_1 = 2.60 \times 10^2$ $C_2 = 5.37 \times 10^{11}$ $C_3 = 1.31 \times 10^{22}$	$K = d_g^{-2.4} t_p^{-0.6}$
vii)	G	T-stub connection	$C_1 = 4.05 \times 10^2$ $C_2 = 4.45 \times 10^{13}$ $C_3 = -2.03 \times 10^{23}$	$K = d^{-1.5} t_f^{-0.5} l_t^{-0.7} d_b^{-1.1}$

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Table 42 (*Concluded*)

Sl No.	Type	Connection Type	Curve-Fitting Constants	Standardization Constants
(1)	(2)	(3)	(4)	(5)
viii)	H	Header-plate connection	$C_1 = 3.87$ $C_2 = 2.71 \times 10^5$ $C_3 = 6.06 \times 10^{11}$	$K = t_p^{-1.6}\, g^{1.6}\, d_b^{-2.3}\, t_w^{-0.5}$

where

d = depth, in mm, of beam;

d_a = depth, in mm, of the angle;

d_b = diameter, in mm, of the bolt;

d_g = centre, in mm, to centre of the outermost bolt of the end plate connection;

g = gauge, in mm, distance of bolt line;

t_a = thickness, in mm, of the top angle;

t_c = thickness, in mm, of the web angle;

t_f = thickness, in mm, of flange T-stub connector;

t_w = thickness, in mm, of web of the beam in the connection;

t_p = thickness, in mm, of end plate, header plate;

l_a = length, in mm, of the angle; and

l_t = length, in mm of the T-stub connector.

NOTE — For preliminary analysis using a bilinear moment curvature relationship, the stiffness given in Table 43 may be assumed depending on the type of connection. The values are based on the secant stiffness at a rotation of 0.01 radian and typical dimension of connecting angle and other components as given in the table.

Table 43 Secant Stiffness

(*Table 42*)

SI No.	Type of Connection	Dimension mm	Secant Stiffness kNm/Radian
(1)	(2)	(3)	(4)
i)	Single web connection angle	$d_a = 250$, $t_a = 10$, $g = 35$	1 150
ii)	Double web-angle connection	$d_a = 250$, $t_a = 10$, $g = 77.5$	4 450
iii)	Top and seat angle connection without double web angle connection	d_a=300, t_a=10, l_a=140, d_b=20	2 730
iv)	Header plate	$d_p = 175$, $t_p = 10$, $g = 75$, $t_w = 7.5$	2 300

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F-5 COLUMN BASES

F-5.1 Base Plates

Columns shall be provided with base plates capable of distributing the compressive forces in the compressed parts of the column over a bearing area such that the bearing pressure on the foundation does not exceed the design strength of the point. The design strength of the joint between the base plate and the foundation shall be determined taking account of the material properties and dimensions of both the grout and the concrete foundation.

F-5.2 Holding Down Bolts (Anchor Bolts)

F-5.2.1 Holding down bolts shall be provided if necessary to resist the effects of the design loads. They shall be designed to resist tension due to uplift forces and tension due to bending moments as appropriate.

F-5.2.2 When calculating the tension forces due to bending moments, the lever arm shall not be taken as more than the distance between the centroid of the bearing area on the compression side and the centroid of the bolt group on the tension side, taking the tolerances on the positions of the holding down bolts into account.

F-5.2.3 Holding down bolts shall either be anchored into the foundation by a hook or by a washer plate or by some other appropriate load distributing member embedded in the concrete.

F-5.2.4 If no special elements for resisting the shear force are provided, such as block or bar shear connectors, it shall be demonstrated that sufficient resistance to transfer the shear force between the column and the foundation is provided by one of the following:

a) Frictional resistance of the joint between the base plate and the foundation;

b) Shear resistance of the holding down bolts;

c) Shear resistance of the surrounding part of the foundation; and

d) Shear and bearing resistance of the shear key plates welded to the base plate and embedded in the pedestal/foundation.

ANNEX G

(*Foreword*)

PLATE GIRDERS WITH CORRUGATED WEB

G-1 FLEXURAL RESISTANCE

The overhang of the flange beyond the farthest face of the web at any section may be limited to that required by class 1, 2 and 3 sections, as given in Table 2. Contribution of web in flexural resistance is disregarded as given below.

The flexural design resistance is given by

$$M_d = Min\left\{\frac{b_c t_c f_{fc}}{\gamma_{m0}}\left(h_w + \frac{t_t + t_c}{2}\right); \frac{b_t t_t f_{ft}}{\gamma_{m0}}\left(h_w + \frac{t_t + t_c}{2}\right)\right\}$$

where

The dimensional symbols are shown in Fig. 34.

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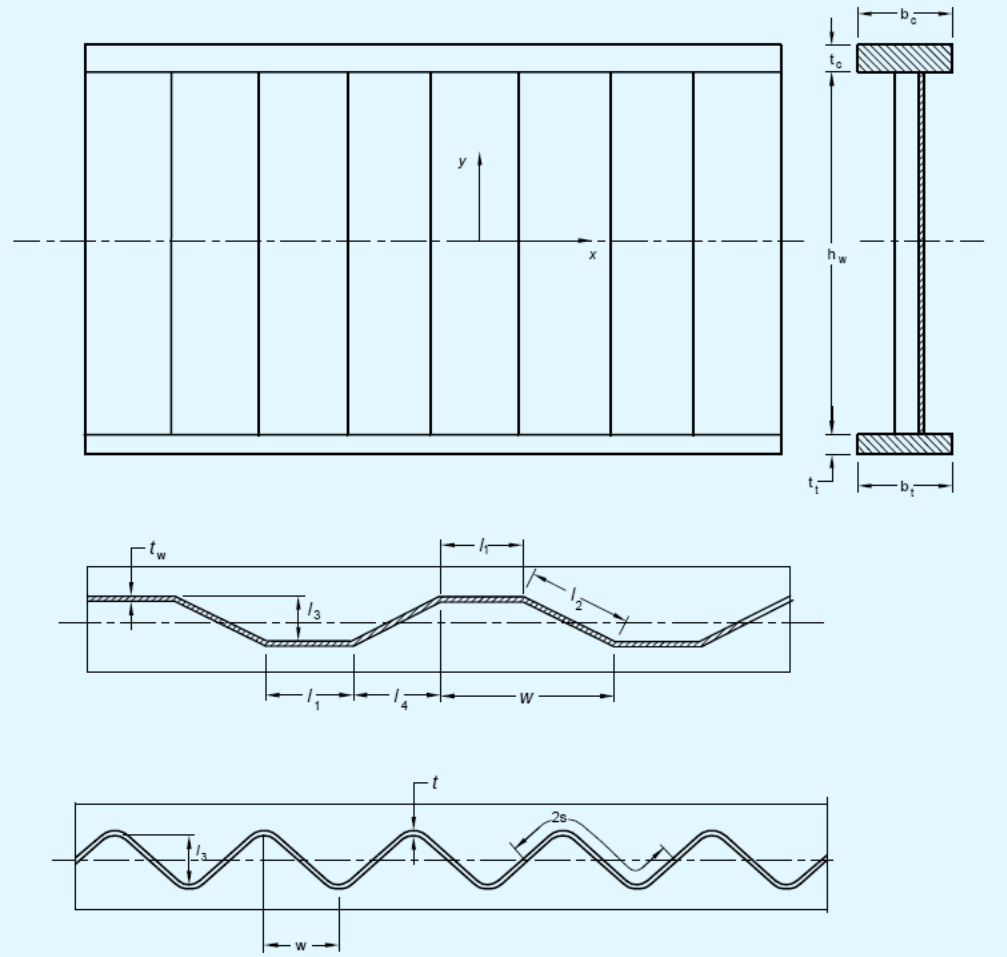

FIG. 34 ELEMENT DIMENSIONS OF PLATE GIRDER WITH CORRUGATED WEB

The design compressive stress, f_{fc}, is the minimum of yielding compressive stress after accounting for transverse (in-plane) bending of compression flange due to eccentric shear flow in the flange due to corrugated web as given below, or by lateral buckling of compression flange as given in **15.2.2**.

The yield strength after accounting for eccentric shear flow in the flanges is given by

$$f_{fc} = f_{yc}\sqrt{1 - \frac{4\,M_{vc}\gamma_{m0}}{b_c^2 t_c f_{yc}}}\,; \qquad f_{ft} = f_{yt}\sqrt{1 - \frac{M_{vt}\gamma_{m0}}{M_{Pf}}}$$

where

f_{yc}, f_{yt} = Yield strength of the material in the tension and compression flanges respectively

M_{Pf} = Ultimate design resistance of the flange under in in-plane bending

= $t\,b^3\,f_{yf}/4$ for plastic and compact flanges, $t\,b^3\,f_{yf}/6$ for semi compact flanges

M_{vc}, M_{vt} = in-plane transverse bending moment in the tension and compression flange, respectively due to eccentric shear force from the web as shown in Fig. 35, and is evaluated from

$$M_{vc} = \frac{V\,y_c b_c\,t_c\,l_1 l_3}{2I_z}; \qquad M_{vt} = \frac{V\,y_t b_t\,t_t\,l_1 l_3}{2I_z}$$

where

V = Shear force at the section of the girder;

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I_z = Moment of inertia about the major axis of the section disregarding the corrugated web; and

y_c, y_t = Distance of the centroid of the compression and tension flange, respectively, from the neutral axis of the girder.

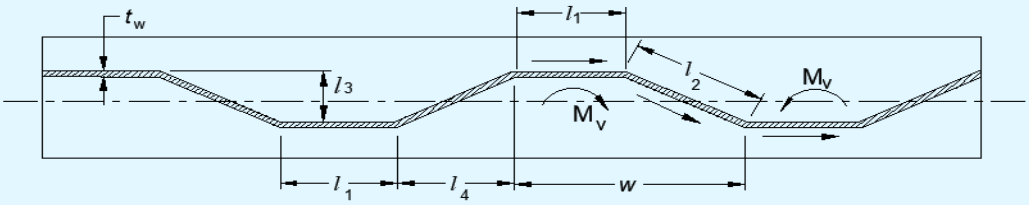

FIG. 35 IN-PLANE TRANSVERSE BENDING MOMENT IN FLANGES DUE TO ECCENTRIC SHEAR FLOW FROM WEB

G-2 SHEAR RESISTANCE

The design shear resistance of the web, V_d, is given by

$$V_d = V_n/\gamma_{m0}$$

where

$$V_n = h_w\, t_w\, \chi_v\, f_{yw}/\sqrt{3}$$

where

h_w, t_w, f_{yw} = depth, thickness and yield strength of the web;

χ_v = shear stress reduction factor to account for web buckling, and is the smallest of local buckling χ_{vl}, global buckling, χ_{vg}, or interaction buckling, χ_{vlg}, of the web under shear, evaluated as given below; and

χ_v = min (χ_{vl}, χ_{vg}, χ_{vlg}).

The shear stress reduction factor for local buckling of the web is evaluated as,

$\chi_{vl} = 1.0$ when $\lambda_{wl} \leq 0.8$,

$\chi_{vl} = [1 - 0.8(\lambda_{wl} - 0.8)]$ when $0.8 < \lambda_{wl} < 1.2$,

$\chi_{vl} = \frac{1}{\sqrt{3}\lambda_{wl}^2}$ when $\lambda_{wl} \geq 1.2$,

where

$$\lambda_{wl} = \sqrt{f_{yw}/(\sqrt{3}\tau_{cr,l})}$$

$$\tau_{cr,l} = \frac{k_v \pi^2 E}{12(1-\mu^2)\,(h_w/t_w)^2}$$

$$k_v = 4.0 + 5.35\left(\frac{c}{h_w}\right) \quad \text{when } (c/h_w) < 1.0$$

$$k_v = 5.35 + 4.0\left(\frac{c}{h_w}\right) \quad \text{when } (c/h_w) \geq 1.0$$

where

c is larger of l_1 and l_2 of corrugated web (*see* Fig. 35).

In case of sinusoidally corrugated webs $\tau_{cr,e}$ may be evaluated using:

$$\tau_{cr,l} = (5.35 + \frac{l_3\, s}{h_w t_w})\frac{\pi^2 E}{12(1-\mu^2)(s/t_w)^2}$$

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The shear stress reduction factor for global buckling of the web is evaluated as:

$$\chi_{vg} = \frac{1.5}{(0.5+\lambda_{wg}^2)} \leq 1.0$$

$$\lambda_{wg} = \sqrt{f_{\text{yw}}/(\sqrt{3}\tau_{\text{cr,g}})}$$

$$\tau_{cr,g} = \frac{32.4}{t_w h_w^2}\sqrt[4]{D_x D_z^3}$$

$D_x = \frac{Et_w^3}{12(1-\mu^2)}\frac{w}{s}$ for sinusoidal corrugation

$D_x = \frac{Et_w^3}{12(1-\mu^2)}\frac{w}{(l_1+l_2)}$ for profiled corrugation

$D_z = \frac{EI_z}{w}$

I_z = second moment of area of one corrugation of length w

The shear stress reduction factor for global buckling of the web is evaluated as:

$$\chi_{vlg} = \frac{\chi_{vl} \times \chi_{vg}}{\sqrt{{\chi_{vl}}^2 + {\chi_{vg}}^2}} \leq 1.0$$

The reduction in shear resistance of the web due to the moment at the section need not be considered, since its contribution to flexure is not considered.

G-3 RESISTANCE TO CONCENTRATED LOAD

The design for bearing resistance under concentrated load and reactions may be carried out as required in **15.7.**

ANNEX H

(*Clause* 24.16)

GENERAL RECOMMENDATIONS FOR STEELWORK TENDERS AND CONTRACTS

H-1 GENERAL

H-1.1 The recommendations given in this annex are in line with those generally adopted for steelwork construction and are meant for general information.

H-1.2 These recommendations do not form part of the requirements of the Section and compliance with these is not necessary for the purpose of complying with this Section.

H-1.3 The recommendations are unsuitable for inclusion as a block requirement in a contract, but in drawing up a contract the points mentioned should be given consideration.

H-2 EXCHANGE OF INFORMATION

Before the steelwork design is commenced, the building designer should be satisfied that the planning of the building, its dimensions and other principal factors meet the requirements of the building owner and comply with regulations of all authorities concerned. Collaboration of building designer and steelwork designer should begin at the outset of the project by joint consideration of the planning and of such questions as the stanchion spacing, materials to be used for the construction, and depth of basement.

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H-3 INFORMATION REQUIRED BY THE STEELWORK DESIGNER

H-3.1 General

Site plans showing in plan and elevation of the proposed location and main dimensions of the building or structure;

a) Ground levels, existing and proposed;

b) Particulars of buildings or other constructions which may have to remain on the actual site of the new building or structure during the erection of the steelwork;

c) Particulars of adjacent buildings affecting, or affected by, the new work;

d) Stipulation regarding the erection sequence or time schedule;

e) Conditions affecting the position or continuity of members;

f) Limits of length and weight of steel members in transit and erection;

g) Drawings of the substructure, proposed or existing, showing:

   1) levels of stanchion foundations, if already determined;

   2) any details affecting the stanchion bases or anchor bolts;

   3) permissible bearing pressure on the foundation; and

   4) provisions for grouting.

NOTE — In the case of new work, the substructure should be designed in accordance with the relevant standards dealing with foundations and substructure.

h) The maximum wind velocity appropriate to the site (*see* Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS); and

j) Environmental factors, such as proximity to sea coast, and corrosive atmosphere. Reference to bye-laws and regulations affecting the steelwork design and construction.

H-3.2 Further Information Relating to Buildings

a) Plans of the floors and roof with principal dimensions, elevations and cross sections showing heights between floor levels;

b) The occupancy of the floors and the positions of any special loads should be given;

c) The building drawings, which should be fully dimensioned, should preferably be to the scale of 1 to 100 and should show all stairs, fire escapes, lifts, etc, suspended ceilings, flues and ducts for heating and ventilating. Doors and windows should be shown, as the openings may be taken into account in the computations of dead load;

   1) Requirements should be given in respect of any maximum depth of beams or minimum head room; and

   2) Large-scale details should be given of any special features affecting the steelwork.

d) The inclusive weight per m^2 of walls, floors, roofs, suspended ceilings, stairs and partitions, or particulars of their construction and finish for the computation of dead load;

   1) The plans should indicate the floors, which are to be designed to carry partitions. Where the layout of partitions is not known, or a given layout is liable to alteration, these facts should be specially noted so that allowance may be made for partitions in any position (*see* Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS).

e) The superimposed loads on the floors appropriate to the occupancy, as given in Part C 'Structural Design/Section 1 Loads and Load Effects' of this NBCS or as otherwise required;

f) Details of special loads from cranes, runways, tips, lifts, bunkers, tanks, plant and equipment; and

g) The grade of fire resistance appropriate to the occupancy as may be required.

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H-4 INFORMATION REQUIRED BY TENDERER (IF NOT ALSO DESIGNER)

H-4.1 General

All information listed under **H-3.1**;

a) Climatic conditions at site-seasonal variations of temperature, humidity, wind velocity and direction;

b) Nature of soil. Results of the investigation of sub-soil at site of building or structure;

c) Accessibility of site and details of power supply;

d) Whether the steelwork contractor will be required to survey the site and set out or check the building or structure lines, foundations and levels;

e) Setting-out plan of foundations, stanchions and levels of bases;

f) Cross sections and elevations of the steel structure, as necessary, with large-scale details of special features;

g) Whether the connections are to be bolted, riveted or welded. Particular attention should be drawn to connections of a special nature, such as turned bolts, high strength friction grip bolts, long rivets and overhead welds;

h) Quality of steel, and provisions for identification;

j) Requirements in respect of protective paintings at works and on site, galvanizing or cement wash;

k) Approximate dates for commencement and completion of erection;

m) Details of any tests which have to be made during the course of erection or upon completion; and

n) Schedule of quantities. Where the tenderer is required to take off quantities, a list should be given of the principal items to be included in the schedule.

H-4.2 Additional Information Relating to Buildings

a) Schedule of stanchions giving sizes, lengths and typical details of brackets, joints, etc;

b) Plan of grillages showing sizes, lengths and levels of grillage beams and particulars of any stiffeners required;

c) Plans of floor beams showing sizes, lengths and levels eccentricities and end moments. The beam reactions and details of the type of connection required should be shown on the plans;

d) Plan of roof steelwork. For a flat roof, the plan should give particulars similar to those of a floor plan. Where the roof is pitched, details should be given of trusses, portals, purlins, bracing, etc;

e) The steelwork drawings should preferably be to a scale of 1 to 100 and should give identification marks against all members; and

f) Particulars of holes required for services, pipes, machinery fixings, etc Such holes should preferably be drilled at works.

H-4.3 Information Relating to Execution of Building Work

a) Supply of materials;

b) Weight of steelwork for payment;

c) Wastage of steel;

d) Insurance, freight and transport from shop to site;

e) Site facilities for erection;

f) Tools and plants;

g) Mode and terms of payment;

h) Schedules;

j) Force majeure (sections and provisions for liquidation and damages for delay in completion); and

k) Escalation sections.

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H-5 DETAILING

In addition to the number of copies of the approved drawings or details required under the contract, dimensioned shop drawings or details should be submitted in duplicate to the engineer who should retain one copy and return the other to the steel supplier or fabricators with his comments, if any.

H-6 TIME SCHEDULE

As the dates on which subsequent trades can commence, depend on the progress of erection of the steel framing, the time schedule for the latter should be carefully drawn up and agreed to by the parties concerned at a joint meeting.

H-7 PROCEDURE ON SITE

The steelwork contractor should be responsible for the positioning and levelling of all steelwork. Any checking or approval of the setting out by the general contractor or the engineer should not relieve the steelwork contractor of his responsibilities in this respect.

H-8 INSPECTION

References may be made to the good practice [C-6A(44)] for general guidance.

H-8.1 Access to Contractor's Works

The contractor should offer facilities for the inspection of the work at all stages.

H-8.2 Inspection of Fabrication

Unless otherwise, agreed, the inspection should be carried out at the place of fabrication. The contractor should be responsible for the accuracy of the work and for any error, which may be subsequently discovered.

H-8.3 Inspection on Site

To facilitate inspection, the contractor should during all working hours, have a foreman or properly accredited charge hand available on the site, together with a complete set of contract drawings and any further drawings and instructions which may have been issued from time to time.

H-9 MAINTENANCE

H-9.1 General

Where steelwork is to be encased in solid concrete, brickwork or masonry, the question of maintenance should not arise, but where steelwork is to be housed in hollow fire protection or is to be unprotected, particularly where the steelwork is exposed to a corroding agent, the question of painting or protective treatment of the steelwork should be given careful consideration at the construction stage, having regard to the special circumstances of the case.

H-9.2 Connections

Where connections are exposed to a corroding agent, they should be periodically inspected, and any corroded part should be thoroughly cleaned and painted.

Where bolted connections are not solidly encased and are subject to vibratory effects of machinery or plant, they should be periodically inspected and all bolts tightened.

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LIST OF STANDARDS

The following list records those standards which are acceptable as 'good practice' and 'accepted standards' in the fulfilment of the requirements of this NBCS. The latest version of a standard shall be adopted at the time of enforcement of this NBCS. The standards listed may be used by the Authority as a guide in conformance with the requirements of the referred clauses in this NBCS.

In the following list, the number appearing in the first column within parentheses indicates the number of the reference in this Subsection of this NBCS.

Sl No.	*IS No.*	*Title*
(1)	IS 786 : 1967	Conversion factors and conversion tables (*first revision*)
(2)	IS 808 : 2021	Hot rolled steel beam, column, channel and angle sections — Dimensions and properties (*fourth revision*)
	IS 1852 : 1985	Specification for rolling and cutting tolerance for hot rolled steel products (*fourth revision)*
	IS 12779 : 1989	Rolling and cutting tolerances for hot rolled parallel flange beam and column sections — Specification
(3)	IS 8000	Geometrical product specifications (GPS) — Geometrical tolerancing
	(Part 1) : 2019	Tolerances of form, orientation, location and run-out (*second revision*)
	(Part 2) : 2024	Maximum material requirement (MMR), least material requirement (LMR) and reciprocity requirement (RPR) (*third revision*)
	(Part 3) : 2023	Profile tolerancing (*third revision*)
	(Part 4) : 1976	Practical examples of indications on drawings
	IS 8976 : 1978	Guide for preparation and arrangement of sets of drawings and parts list
	IS 962 : 1989	Code of practice for architectural and building drawings (*second revision*)
(4)	IS 962 : 1989	Code of practice for architectural and building drawings (*second revision*)
(5)	IS 813 (Part 1) : 2018/ ISO 2553 : 2013	Welding and allied processes: Part 1 Symbolic representation on drawings — Welded Joints (*second revision*)
(6)	IS 2062 (Part 1) : 2025	Structural steel: Part 1 Hot rolled medium and high tensile steel *(eighth revision*)
(7)	IS 1608	Metallic materials — Tensile testing:
	(Part 1) : 2022/ ISO 6892-1 : 2019	Method of test at room temperature *(fifth revision)*
	(Part 2) : 2020/ ISO 6892-2 : 2018	Method of test at elevated temperature (*fourth revision*)
(8)	IS 1161 : 2014	Steel tubes for structural purposes — Specification (*fifth revision*)
(9)	IS 814 : 2004	Covered electrodes for manual metal arc welding of carbon and carbon manganese steel — Specification (*sixth revision*)
(10)	IS 1395 : 1982	Specification for low and medium alloy steel covered electrodes for manual metal arc welding (*third revision*)
(11)	IS 7557 : 1982	Specification for steel wire (up to 20 mm) for the manufacture of cold-forged rivets (*first revision*)
(12)	IS 1148 : 2009	Steel rivet bars (medium and high tensile) for structural purposes (*fourth revision*)
(13)	IS 1929 : 1982	Specification for hot forged steel rivets for hot closing (12 mm to 36 mm diameter) (*first revision*)

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Sl No.	*IS No.*	*Title*
	IS 2155 : 1982	Specification for cold forged solid steel rivets for hot closing (6 mm to 16mm diameter) (*first revision*)
(14)	IS 1363	Hexagon head bolts, screws and nuts of product grade C:
	(Part 1) : 2023/ ISO 4016 : 2022	Hexagon head bolts (size range M5 to M64) (*sixth revision*)
	(Part 2) : 2023/ ISO 4018 : 2022	Hexagon head screws (size range M5 to M64) (*sixth revision*)
	(Part 3) : 2018/ ISO 4034 : 2012	Hexagon nuts, Style 1 (size range M5 to M64) (*fifth revision*)
	IS 1364	Hexagon head bolts, screws and nuts of product grades A and B:
	(Part 1) : 2023/ ISO 4014 : 2022	Hexagon head bolts (size range M1.6 to M64) (*sixth revision*)
	(Part 2) : 2023/ ISO 4017 : 2022	Hexagon head screws (size range M1.6 to M64) (*sixth revision*)
	(Part 3) : 2018/ ISO 4032 : 2012	Hexagon nuts, Style 1 (size range M1.6 to M64) (*fifth revision*)
	(Part 4) : 2025/ ISO 4035 : 2023	Hexagon nuts , Style 0 (size range M1.6 to M64) (*fifth revision*)
	(Part 5) : 2002/ ISO 4036 : 1999	Hexagon thin nuts — Product grade B (unchamfered) (size range M1.6 to M10) (*fourth revision*)
	IS 1367	Technical supply conditions for threaded steel fasteners:
	(Part 1) : 2014/ ISO 8992 : 2005	General requirements for bolts, screws, studs and nuts (*fourth revision*)
	(Part 2) : 2002/ ISO 4759-1 : 2000	Tolerance for fasteners — Bolts, screws, studs and nuts — Product grades A, B and C (*third revision*)
	(Part 3) : 2017/ ISO 898-1 : 2013	Mechanical properties of fasteners made of carbon steel and bolts, screws and studs (*fifth revision*)
	(Part 5) : 2018/ ISO 898-5 : 2012	Mechanical properties of fasteners made of carbon steel and alloy steel — Set screws and similar threaded fasteners with specified hardness classes — Coarse thread and fine patch thread (*fourth* revision)
	(Part 6) : 2025/ ISO 898-2 : 2022	Mechanical properties of fasteners made of carbon steel and alloy steel — Nuts with specified property classes (*fifth revision*)
	(Part 7) : 1980	Mechanical properties and test methods for nuts without specified proof loads (*second revision*)
	(Part 8) : 20020/ ISO 2320 : 2015	Prevailing torque type steel nuts — functional properties (*fifth revision*)
	(Part 9)	Surface discontinuities:
	Sec 1 : 1993/ ISO 6157-1 : 1988	Bolts, screws and studs for general application (*third revision)*
	Sec 2 : 1993/ ISO 6167-3 : 1988	Bolts, screws and studs for special applications (*third revision*)

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Sl No.	*IS No.*	*Title*
	(Part 10) : 2002/ ISO 6157-2 : 1995	Surface discontinuities — Nuts (*third revision*)
	(Part 11) : 2024/ ISO 4042 : 2022	Electroplated coating systems (*fifth revision*)
	(Part 12) : 1981	Phosphate coatings on threaded fasteners (*second revision*)
	(Part 13) : 2020/ ISO 10684 : 2004	Hot dip galvanized coatings on threaded fasteners (*third revision*)
	(Part 14)	Mechanical properties of corrosion-resistant stainless-steel fasteners:
	Sec 1 : 2023/ ISO 3506-1 : 2020	Bolts, screws and studs with specified grades and property classes (*fifth revision*)
	Sec 2 : 2023/ ISO 3506-2 : 2020	Nuts with specified grades and property classes (*fifth revision*)
	Sec 3 : 2018/ ISO 3506-3 : 2009	Set screws and similar fasteners not under tensile stress (*fourth revision*)
	(Part 16) : 2002/ ISO 8991 : 1986	Designation system for fasteners (*third revision*)
	(Part 17) : 2023/ ISO 3269 : 2019	Inspection, sampling and acceptance procedure (*fifth revision*)
	(Part 18) : 1996	Packaging (*third revision*)
	(Part 19) : 1997/ ISO 3800 : 1993	Axial load fatigue testing of bolts, screws and studs
	(Part 20) : 1996/ ISO 898-7 : 1992	Torsional test and minimum torques for bolts and screws with nominal diameters 1 mm to 10 mm
	IS 3640 : 1982	Specification for hexagon fit bolts (*first revision*)
	IS 3757 : 1985	Specification for high strength structural bolts (*second revision*)
	IS 5370 : 1969	Specification for plain washers with outside diameter ≈ 3 × inside diameter
	IS 5372 : 1975	Taper washer for channels (ISMC) (*first revision*)
	IS 5374 : 1975	Taper washers for I-beams (ISMB) (*first revision*)
	IS 5624 : 2021	Foundation bolts - Specification (*second revision*)
	IS 6610 : 1972	Specification for heavy washers for steel structures
	IS 6623 : 2004	High strength structural nuts — Specification (*second revision*)
	IS 6639 : 1972	Specification for hexagon bolts for steel structures
	IS 6649 : 1985	Specification for hardened and tempered washers for high strength structural bolts and nuts (*first revision*)
(15)	IS 4000 : 1992	High strength bolts in steel structures — Code of practice (*first revision*)
(16)	IS 1030 : 1998	Carbon steel castings for general engineering purposes (*fifth revision*)
(17)	IS 2708 : 1993	1.5 percent manganese steel castings for general engineering purpose (*third revision*)
(18)	IS 1278 : 1972	Specification for filler rods for gas welding (*second revision*)

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Sl No.	*IS No.*	*Title*
(19)	IS 1387 : 1993	General requirements for the supply of metallurgical materials (*second revision*)
(20)	IS 15977 : 2013	Classification and acceptance test for bare solid wire electrodes and wire flux combination for submerged arc welding of structural steel — Specification
(21)	IS 6419 : 1996	Welding rods and bare electrodes for gas shielded arc welding of structural steels — Specification (*first revision*)
	IS 6560 : 2017/ ISO 21952 : 2012	Welding consumables — wire electrodes, wires, rods and deposits for gas shielded arc welding of creep resisting steels — Classification (*second revision*)
(22)	IS 513	Cold reduced carbon steel sheet and strip:
	(Part 1) : 2016	Cold forming and drawing purpose (*sixth revision*)
	(Part 2) : 2016	High tensile and multi-phase steel (*sixth revision*)
(23)	IS 1079 : 2017	Hot rolled carbon steel sheet, plate and strip - Specification (*seventh revision*)
(24)	IS 1367 (Part 3) : 2017/ISO 898-1 : 2013	Technical supply conditions for threaded steel fasteners: Part 3 Mechanical properties of fasteners made of carbon steel and bolts, screws and studs (*fifth revision*)
(25)	IS 1875 : 1992	Carbon steel billets, blooms, slabs and bars for forgings — Specification (*fifth revision*)
(26)	IS 1990 : 2009	Steel rivet and stay bars for boilers (*second revision*)
(27)	IS 2002 : 2024	Steel plates for pressure vessels for intermediate and high temperature service including boilers — Specification (*fourth revision*)
(28)	IS 3039 : 2024	Structural steel for ship construction — Specification (*third revision*)
(29)	IS 6240 : 2008	Hot rolled steel plate (up to 6 mm) sheet and strip for the manufacture of low pressure liquefiable gas cylinders (*fourth revision*)
(30)	IS 9295 : 2024	Steel tubes for idlers for belt conveyors — Specification (*second revision*)
(31)	IS 10748 : 2004	Hot rolled steel strip for welded tubes and pipes — Specification (*second revision*)
(32)	IS 11587 : 1986	Specification for structural weather resistant steels
(33)	IS 15103 : 2002	Fire resistant steel — Specification
(34)	IS 15962 : 2012	Structural steels for building and structures with improved seismic resistance
(35)	IS 801: 1975	Code of practice for use of cold-formed light gauge steel structural members in general building construction (*first revision*)
(36)	IS 808 : 2021	Hot-rolled steel beam, column, channel and angle sections — Dimensions and Properties (*fourth revision*)
(37)	IS 4326 : 2013	Earthquake resistance design and construction of buildings — Code of practice (*third revision*)
(38)	IS 1946 : Part 2 : 2025	Fixing Devices in Walls, Ceilings and Floors of Solid Construction: Part 2 Design of post-installed anchorage to concrete — Code of practice
(39)	IS 3757 : 1985	Specification for high strength structural bolts (*second revision*)
(40)	IS 1367	Technical supply conditions for threaded steel fasteners:
	(Part 1) : 2014/ ISO 8992 : 2005	General requirements for bolts, screws, studs and nuts (*fourth revision*)

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Sl No.	*IS No.*	*Title*
	(Part 2) : 2002/ ISO 4759-1 : 2000	Tolerance for fasteners — Bolts, screws, studs and nuts – Product grades A, B and C (*third revision*)
	(Part 3) : 2017/ ISO 898-1 : 2013	Mechanical properties of fasteners made of carbon steel and bolts, screws and studs (*fifth revision*)
	(Part 5) : 2018/ ISO 898-5 : 2012	Mechanical properties of fasteners made of carbon steel and alloy steel — Set screws and similar threaded fasteners with specified hardness classes — Coarse thread and fine patch thread (*fourth* revision)
	(Part 6) : 2025/ ISO 898-2 : 2022	Mechanical properties of fasteners made of carbon steel and alloy steel — Nuts with specified property classes (*fifth revision*)
	(Part 7) : 1980	Mechanical properties and test methods for nuts without specified proof loads (*second revision*)
	(Part 8) : 20020/ ISO 2320 : 2015	Prevailing torque type steel nuts — Functional properties (*fifth revision*)
	(Part 9)	Surface discontinuities
	Sec 1 : 1993/ ISO 6157-1: 1988	Bolts, screws and studs for general application (*third revision*)
	Sec 2 : 1993/ ISO 6167-3 : 1988	Bolts, screws and studs for special applications (*third revision*)
	(Part 10) : 2002/ ISO 6157-2 : 1985	Surface discontinuities — Nuts (*third revision*)
	(Part 11) : 2024/ ISO 4042 : 2022	Electroplated coating systems (*fifth revision*)
	(Part 12) : 1981	Phosphate coatings on threaded fasteners (*second revision*)
	(Part 13) : 2020/ ISO 10684 : 2004	Hot dip galvanized coatings on threaded fasteners (*third revision*)
	(Part 14)	Mechanical properties of corrosion-resistant stainless-steel fasteners
	Sec 1 : 2023/ ISO 3506-1 : 2020	Bolts, screws and studs with specified grades and property classes (*fifth revision*)
	Sec 2 : 2023/ ISO 3506-2 : 2020	Nuts with specified grades and property classes (*fifth revision*)
	Sec 3 : 2018/ ISO 3506-3 : 2009	Set screws and similar fasteners not under tensile stress (*fourth revision*)
	(Part 16) : 2002/ ISO 8991 : 1986	Designation system for fasteners (*third revision*)
	(Part 17) : 2023/ ISO 3269 : 2019	Inspection, sampling and acceptance procedure (*fifth revision*)
	(Part 18) : 1996	Packaging (*third revision*)
	(Part 19) : 1997/ ISO 3800 : 1993	Axial load fatigue testing of bolts, screws and studs

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Sl No.	*IS No.*	*Title*
	(Part 20) : 1996/ ISO 898-7 : 1992	Torsional test and minimum torques for bolts and screws with nominal diameters 1 mm to 10 mm
(41)	IS 816 : 1969	Code of practice for use of metal arc welding for general construction in mild steel (*first revision*)
	IS 9595:1996	Metal arc welding of carbon and carbon manganese steels — Recommendations (*first revision*)
(42)	IS 18168 : 2023	Earthquake resistant design and detailing of steel buildings — Code of practice
(43)	IS 9077 : 1979	Code of practice for corrosion protection of steel reinforcement in RB and RCC construction
	IS 9172 : 1979	Recommended design practice for corrosion prevention of steel structures
(44)	IS 7215 : 1974	Tolerances for fabrication of steel structures
(45)	IS 12843 : 1989	Tolerances for erection of steel structures
(46)	IS 9595 : 1996	Metal arc welding of carbon and carbon manganese steels-Recommendations (*first revision*)
(47)	IS 919	Geometrical product specifications (GPS) — ISO code system for tolerances on linear sizes:
	(Part 1) : 2014/ ISO 286-1 : 2010	Bases of tolerance, deviations and fits (*third revision*)
	(Part 2) : 2014/ ISO 286-2 : 2010	Tables of standard tolerance grades and limit deviations for holes and shafts (*second revision*)
(48)	IS 816 : 1969	Code of practice for use of metal arc welding for general construction in mild steel (*first revision*)
	IS 819 : 1957	Code of practice for resistance spot welding for light assemblies in mild steel
	IS 1024 : 2025	Use of welding in bridges and structures subject to dynamic loading — Code of practice (*third revision*)
	IS 1261 : 1959	Code of practice for seam welding in mild steel
	IS 1323 : 1982	Code of practice for oxy-acetylene welding for structural work in mild steels (*second revision*)
	IS 9595 : 1996	Metal arc welding of carbon and carbon manganese steels-Recommendations (*first revision*)
(49)	IS 817	Training of welders — Code of Practice
	(Part 1) : 1992	Manual metal arc welding (*second revision*)
	(Part 2) : 1996	Oxyfuel welding (*second revision*)
	IS 1393 : 1961	Code of practice for training and testing of oxy-acetylene welders
	IS 7307 (Part 1) : 1974	Approval tests for welding procedures: Part 1 Fusion welding of steel
	IS 7310 (Part 1) : 2019/ISO 9606-1 : 2012	Qualification testing of welders — Fusion welding: Part 1 Steels (*first revision*)
	IS 7318 (Part 1) : 1974	Approval tests for welders when welding procedure approval is not required: Part 1 Fusion welding of steel

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Sl No.	*IS No.*	*Title*
(50)	IS 1477	Code of practice for painting of ferrous metals in buildings:
	(Part 1) : 1971	Pre-treatment (*first revision*)
	(Part 2) : 1971	Painting (*first revision*)
(51)	IS 7205 : 1974	Safety code for erection of structural steelwork
(52)	IS 456 : 2000	Plain and reinforced concrete — Code of practice (*fourth revision*)

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 6 STEEL AND COMPOSITE CONSTRUCTION

6B COMPOSITE CONSTRUCTION IN STRUCTURAL STEEL AND CONCRETE

BUREAU OF INDIAN STANDARDS

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National Building Code Sectional Committee, CED 46

FOREWORD

This NBCS (Part C/Subsection 6B) covers the composite construction consists of the use of prefabricated steel structural units like steel beams and steel open web joists (trusses) in combination with concrete elements and often with steel reinforcements.

The design and construction should ensure monolithic action between the structural steel and concrete components so that they act as a single structural unit.

The Section 6 'Steel' was first published in 1970 and was subsequently revised in 1983, 2005 and 2016 as a chapter of the National Building Code of India. In the second revision in 2005, the detailed design provisions were not included as IS 800 : 1984 'Code of practice for general construction in steel (*second revision*)'was under revision at that time. This Section, therefore, then referred to the 1984 version of IS 800 with the remarks that the latest version would prevail, as and when published. It, however, covered the philosophical aspects of limit state design which was being introduced in the revision of IS 800. IS 800 was subsequently revised and published as IS 800 : 2007 and hence, became the prevailing version for use under this Section. In the third revision of 2016, the Section 6 was brought in line with the revised version of IS 800 : 2007 and its amendment No. 1 and to take care of further improvements and incorporations required.

In the current revision, this publication has been renamed as National Building Construction Standards (NBCS).

In this revision of Section 6, the Section has been subdivided into the following subsections with the chapter on 'composite construction in structural steel and concrete' being introduced for the first time:

a) 6A Steel; and

b) 6B Composite construction in structural steel and concrete.

The provisions of this Subsection (Part C/Subsection 6B) are largely based on IS 11384 : 2022 'Composite construction in structural steel and concrete — Code of practice (*first revision*)'. The growing use of steel-concrete composites in building construction necessitated this new chapter in this NBCS.

The provisions of this Subsection conform to the limit state design philosophy, ensuring consistency with IS 456 : 2000 'Plain and reinforced concrete — Code of practice' for reinforced concrete and Part C/Sec 6A for structural steel. The Subsection includes comprehensive provisions for the design of key structural components such as beams, slabs, and columns in composite construction. It specifically addresses two types of composite columns — Concrete-encased steel columns, which may be either fully or partially encased, and concrete-filled steel columns — Both of which enhance load-bearing capacity and fire resistance. For composite slabs, the Subsection includes two types: profiled sheeting, which serves as formwork for reinforced concrete, and embossed profile sheeting, which functions as both formwork and tension reinforcement, with or without shear connectors. It also provides guidelines for the design and testing of shear connectors to ensure efficient load transfer. Serviceability limit states regulate deflection, vibration, and fire performance. Additionally, the subsection includes specifications for light gauge steel and lightweight concrete, broadening the scope of composite construction applications. This subsection is restricted to the design of steel-concrete composite components and systems used in buildings.

Though the common methods of designs have been covered in this NBCS, special systems of design and construction, not covered by this NBCS, may be permitted on production of satisfactory evidence regarding their design adequacy and safety based on specialist literature or by analysis, test, or both.

All requirements of Part C/Sec 6A in so far as they apply, shall be deemed to form a part of this subsection, except where otherwise laid down in this subsection.

All standards, whether given herein above or cross-referred to in the main text of this Section, are subject to revision. The parties to agreement based on this Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

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As the regulation of land development and buildings is a State subject and is dealt by the States and local bodies such as urban local bodies within their jurisdiction through building regulatory documents like building regulations, building byelaws, and fire regulations, this document is voluntary in nature and is non-binding. The implementation depends on adoption by concerned parties or stipulations in a contract or by appropriate adoption by the concerned authorities.

For the purpose of deciding whether a particular requirement of this Subsection is complied with the final value, observed or calculated, expressing the result of a test or analysis shall be rounded off in accordance with IS 2 : 2022 ‘Rules for rounding off numerical values (*second revision*)’. The number of significant places retained in the rounded-off value should be the same as that of the specified value in this Subsection.

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Important Explanatory Note for Users of the NBCS

In any Part/Section of this NBCS, where reference is made to **'good practice'** in relation to design, constructional procedures or other related information, and where reference is made to **'accepted standard'** in relation to material specification, testing, or other related information, the Indian Standards listed at the end of the Part/Section shall be used as a guide to the interpretation.

At the time of publication, the editions indicated in the standards were valid. All standards are subject to revision and parties to agreements based on any Part/Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

In the list of standards given at the end of a Part/Section, the number appearing within parentheses in the first column indicates the number of the reference of the standard in the Part/Section. For example:

a) Good practices [C-6B(1)] refers to the Indian Standard(s) give at serial number (1) of the list of standards given at the end of this Part C/Subsection 6B, that is, IS 456 : 2000 'Plain and reinforced concrete — Code of practice (*fourth revision*)' and IS 800 : 2007 'General construction in steel — Code of practice (*third revision*)'; and

b) Accepted standards [C-6B(4)] refers to the Indian Standards given at serial number (4) of the list of standards given at the end of this Part C/Subsection 6B, that is, IS 2062 (Part 1) : 2025 'Structural steel: Part 1 Hot rolled medium and high tensile steel (*eighth revision*)'.

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 6 STEEL AND COMPOSITE CONSTRUCTION

6B COMPOSITE CONSTRUCTION IN STRUCTURAL STEEL AND CONCRETE

1 SCOPE

1.1 This NBCS (Part C/Subsection 6B) deals with the design and construction of composite structures made up of structural steel and cast-in-situ/precast concrete, joined together to act integrally.

1.2 This Subsection is applicable to simply supported as well as continuous beams slabs and supporting column systems. The design provisions in this code are based on the limit states method of design.

2 -

NOTE — This clause is intentionally left blank.

3 TERMINOLOGY

For the purpose of this Subsection, the definitions given in as per good practices [C-6B(1)] and the following shall apply.

3.1 Accidental Load — The load is not normally expected in design life but has a major impact if it ever occurs, such as ramming of vehicles against columns or any other member of the frame like bracings, blast loading, etc.

3.2 Accompanying Load — Imposed load acting along with leading imposed load but causing lower action and/or deflections.

3.3 Action — The primary cause for stress or deformations in a structure, such as dead, imposed, wind, seismic, or temperature loads.

3.4 Built-up Section — A member fabricated by inter-connecting more than one element to form a compound section acting as a single member.

3.5 Camber — intentionally introduced pre-curving (generally upwards) in a system, member or any portion of a member with respect to its chord. These are generally introduced to compensate for deflections at a specific level of loads.

3.6 Composite Action — Integral action of primary supporting steel member and supported concrete deck with or without limited slip at their interface to ensure greater strength and rigidity. In composite columns, it is the integral action between steel and its encasement or infill concrete. The shear transfer is to be ensured through the use of mechanical devices known as shear connectors in composite beams and columns, as required (*see* Fig. 8).

3.7 Design Load/Factored Load — A load value obtained by multiplying the characteristic load with the partial safety factor for loads.

3.8 Design Service Life — The time period during which the structure or its components should satisfy the design objectives and functions.

3.9 Detail Category — Designation given to a particular detail to indicate the S-N curve to be used in fatigue assessment.

3.10 Differential Shrinkages — It is entirely due to shrinkage of concrete from the time composite action comes into effect. When the coefficient of thermal expansion varies significantly between steel and concrete (concrete with limestone or granite aggregate), it also includes the difference in thermal strain between the steel and concrete. Differential shrinkage may lead to an increase in stresses and is more pronounced in continuous beams.

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3.11 Fatigue — Damage caused by repeated fluctuations of stress, leading to progressive cracking of a structural element.

3.12 Fatigue Limit State — The state of failure through fatigue damage due to repeated application of loads.

3.13 Fatigue Strength — Stress range that can be endured by a category of detail, depending upon the number of cycles.

3.14 Flexible Shear Connector — It resists shear forces by bending, tension or shearing in the root at the connection point of the steel member. It is subjected to plastic deformations as it reaches the ultimate strength, and the failure mode is ductile. It consists of studs, channels, etc (Fig. 8) welded to steel members to develop composite action with the concrete.

3.15 Initial Dead Load — The combination of weight of steel structure and the portion of the concrete deck that is supported by the steel structure alone before the development of full composite action with concrete reaching 75 percent of its 28 days strength.

3.16 Limit State — The load state beyond which the structure is incapable of performing its desired function.

3.17 Loads — Applied forces as per loads indicated in relevant standards (*see* Part C 'Structural Design/ Section 1 Loads and Load effects' of this NBCS) for dead, imposed, wind, snow loads, etc, and also for seismic loads that the structure is subjected to during its lifetime.

3.18 Partial Safety Factor for Loads — The factors multiplied by the loads or their combinations to obtain design loads while checking performance under various limit states.

3.19 m-k Factors — Physically, in a composite slab, 'm' is a broad measure of the mechanical interlock between the embossed profiled decking sheet and the reinforced cement concrete on top of the deck, and 'k' represents the frictional resistance between the two elements that is, steel and concrete (*see* Fig. 38).

3.20 Rigid Shear Connectors — These are shear hoops welded to bars, angles, horseshoes or tees welded to the flange of the fabricated steel units, as shown in Fig. 8(b) and Fig. 8 (c). They resist shear forces by concrete bearing on the vertical face and shearing. They exhibit negligible deformation under shear transfer (that is, brittle failure). These are not usually recommended for ultimate limit state design.

3.21 Service Limit — The loading state beyond which the structure or its components becomes incapable of performing its intended function due to excessive deformation, or deflection, or vibration.

3.22 Serviceability Loads — The loads on the structure against which the serviceability of the structure needs to be checked.

3.23 Shear Connectors — These are the mechanical attachments to steel members to transfer interface shear between steel and concrete to develop composite action and are composed of flexible shear connectors [Fig. 8 (a), Fig. 8 (c)], rigid shear connectors [Fig. 8 (b)], etc.

3.24 S-N Curve — Curve, defining the relationship between the numbers of stress cycles to failure Nsc at a constant stress range Sc, during fatigue loading on parts of a structure.

3.25 Strength Factors — The factors by which the specified strength is divided to obtain design strength; while assessing the safety under limit states of strength.

3.26 Stress Range — Algebraic difference between two extremes of stresses in a cycle of loading at a location in a member.

3.27 Superimposed Dead Load — The dead loads added subsequent to concrete hardening of concrete that is resisted by composite action.

3.28 Transient Load — The loads that are assumed to be varying over a short time interval like imposed load, loads with dynamic effect, temperature effects, wind loads on the structure, earthquake loads, accident loads, etc.

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3.29 Ultimate Limit State — The state at which the structure fails and loses its integrity leading to its collapse.

4 SYMBOLS

For the purpose of this Subsection, the symbols (other than those used for load categorization) given in Annex A are applicable.

5 MATERIALS AND WORKMANSHIP

5.1 The main materials for composite construction are structural steel, reinforcing steel, and concrete. The materials and workmanship of structural steel shall generally comply with specifications laid down in as per good practice [C-6B(3)], and that of reinforced concrete shall comply with specifications laid down in as per good practice [C-6B(2)]. However, the general properties and specifications of materials for composite construction are detailed as given in **5.2** to **5.4**.

5.2 Structural Steel

All the structural steel used in general construction, coming under the purview of this NBCS, shall before fabrication conform to the accepted standard [C-6B(4)].

Structural steel other than those complying with the accepted standard [C-6B(4)] may also be used provided that the limiting stresses and other design provisions are suitably modified and the steel is also suitable for the type of fabrication adopted.

Steel that is not supported by mill test results may be used only in unimportant members and details, where their properties such as ductility and weldability would not affect the performance requirements of the members and the structure as a whole. However, such steels may be used in the structural system after confirming their quality is as per the accepted standard [C-6B(4)], by carrying out appropriate tests in accordance with the method specified in the accepted standard [C-6B(5)].

5.2.1 *Properties of Structural Steel*

The following physical properties shall be assumed for all grades of steel for design purposes:

Unit mass of steel	= 7 850 kg/m^3
Young's modulus (modulus of elasticity)	= 2.0 × 10^5 MPa
Shear modulus	= 0.769 × 10^5 MPa
Poisson's ratio	= 0.30
Coefficient of thermal expansion	= 0.000 012/°C

The mechanical properties of steel like yield stress, ultimate stress and, elongation shall be as per values indicated in as per good practice [C-6B(3)].

5.2.2 *Specifications of Structural Steel*

Unless otherwise permitted herein, structural steel used shall, before fabrication, comply with the requirements of the accepted standards [C-6B(6)].

The use of structural steel not covered by the above standards may be permitted with the specific approval of the competent authority.

5.2.3 *Other Steels*

Except where permitted with the specific approval of the authority, steels for machined parts and for uses in features other than structural members or elements shall comply with the relevant accepted standards [C-6B(7)], as appropriate.

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5.2.4 *Castings and Forgings*

Steel casting and forgings shall comply with the requirements of the Indian Standards the accepted standard [C-6B(8)] as appropriate.

5.2.5 *Fasteners*

Bolts, nuts, washers and rivets shall comply with the following or relevant accepted standard [C-6B(9)], as appropriate.

5.2.6 *Welding Consumables and Practices*

Welding consumables and practices shall comply with the following accepted standard [C-6B(10)], as appropriate.

5.2.7 *Wire Ropes and Cables*

These shall conform to the relevant accepted standard [C-6B(11)] except where the use of other types is specifically permitted by the authority.

5.3 Concrete

5.3.1 All structural reinforced concrete shall be of minimum grade M20 and shall be in accordance with material specification and workmanship as stipulated in as per good practice [C-6B(2)]. The strengths shall be specified in terms of the characteristic compressive strengths of cubes, f_{ck}, measured at 28 days. The design provisions in this Subsection are applicable for concrete strength between M20 to M75. Specialist literature shall be adopted in composite design, while using concrete strength outside this range.

5.3.2 Concrete grade shall be designated based on its characteristic strength. The three main categories of concrete strength grade are given below, and the recommended design properties of concrete are correlated to 28 day characteristic compressive strength unless specified otherwise. The mechanical properties of concrete, namely, tensile strength and modulus of elasticity, shall be determined as per good practice [C-6B(2)]. Additional data is given in Annex F.

a) *Ordinary Concrete* — Concrete grades up to M20 are included in this type. It could be prepared by a nominal mix proportioned by the weight of its main ingredients;

b) *Standard Concrete* — This type comprises of concrete grades from M25 to M55. It is made based on a design mix proportioned by the weight of its main ingredients, along with chemical admixtures to achieve certain target values; and

c) *High Strength Concrete* — Concrete grades from M60 to M80 are included in this type. Even though usage of high strength concrete is allowed in composite construction, capacity equations specified in this provision shall not be directly used in the design. Specialist literature and experimental results are required for using this concrete.

5.3.3 *Lightweight Concrete*

Lightweight concrete may be used in composite construction, and the design provisions in this NBCS shall be used within a strength range of M20 to M60. The mean tensile strength $f_{ctm,l}$ and modulus of elasticity $E_{cm,l}$ of lightweight concrete is to be calculated as modifications over the provisions in [C-6B(2)], as follows:

$$f_{\mathrm{ctm,l}} = \eta_{\mathrm{l}} f_{\mathrm{ctm}}$$

$$E_{\mathrm{cm,l}} = \eta_{\mathrm{E}} E_{\mathrm{cm}}$$

The factors η_{l} and η_{E} are determined using the following equation:

$$\eta_{\mathrm{l}} = 0.4 + \frac{0.6\,\rho}{2\,200}$$

$$\eta_{\mathrm{E}} = \left(\frac{\rho}{2\,200}\right)^2$$

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Here, ρ is the upper limit of the oven-dry density of the relevant class of lightweight concrete, as given in Table 1.

Table 1 Density Classes and Corresponding Design Densities for Lightweight Concrete

(*Clause* 5.3.3)

Sl No.	Density Class, ρ		1.0	1.2	1.4	1.6	1.8	2.0
(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)
i)	Density range (kg/m^3)		801 to 1 000	1 001 to 1 200	1 201 to 1 400	1 401 to 1 600	1 601 to 1 800	1 801 to 2 000
ii)	Density (kg/m^3)	Plain concrete	1 050	1 250	1 450	1 650	1 850	2 050
		Reinforced concrete	1150	1 350	1 550	1 750	1 950	2 150

5.3.4 Design of concrete components of a composite structure against creep, shrinkage and temperature stress may be done as indicated in as per good practice [C-6B(2)] or as per specialist literature.

5.4 Reinforcement Steel

5.4.1 Reinforcement steel shall consist of hot rolled, thermo-mechanically treated, or heat-treated rods, de-coiled rods, or cold-worked steel of various grades as given in Table 2. The grade designations and strength properties are given in Table 3.

Table 2 Grades of Reinforcing Steel

(*Clause* 5.4.1)

Sl No.	Types of Steel	Grade/Designation	Relevant Standard
(1)	(2)	(3)	(4)
i)	Mild steel (MS)	Grade I	[C-6B(12)]
ii)	High yield strength deformed (HYSD)	Fe 415, Fe 415S, Fe 415D Fe 500, Fe 500S, Fe 500D, Fe 550, Fe 550D, Fe 600	[C-6B(13)]

5.4.2 The minimum strength of reinforcing steel as specified in as per good practice [C-6B(2)] is either the yield stress in case of mild steel or 0.2 percent proof strength in case of high yield strength steel and it is notionally taken as the characteristic strength of reinforcement f_{yk} .

5.4.3 The steel may be coated or galvanized to improve its corrosion resistance. The following corrosion resistive steel may be used as reinforcements:

a) *Galvanized Reinforcements* — The strength, elongation and bond properties are not adversely affected by galvanizing;

b) *Epoxy-coated Reinforcements* — These are reinforcements conforming to the accepted standard [C-6B(13)] coated by fusion bonding epoxy conforming to the accepted standard [C-6B(14)]. The bond of coated reinforcements is lowered by up to 20 percent compared to un-coated reinforcements. The lap length and anchorage length shall be increased by 25 percent while using these steel bars; and

Stainless Steel Reinforcements — Properties of stainless steel reinforcement shall not be inferior to carbon steel reinforcement of corresponding strength and class. These are reinforcements conforming to the accepted standard [C-6B(15)].

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Table 3 Strength of Reinforcing Steel

(*Clause* 5.4.1)

Sl No.	Type of Steel	Grade/ Designation	Minimum Yield Stress/0.2 Percent Proof Stress,	Minimum Tensile Strength, as Percent of the Actual 0.2 Percent Proof Stress/Yield Stress But Not Less Than	Minimum Percent Elongation
(1)	(2)	(3)	(4)	(5)	(6)
i)	Mild steel	Grade - I	Bars up to and including 20 mm dia = 250 MPa	410 MPa	23.0
			Bar dia > 20 mm ≤ 50 mm = 240 MPa	410 MPa	23.0
ii)	High yield strength deformed steel (HYSD)	Fe 415	415 MPa	110 Percent (≥ 485 MPa)	14.5
		Fe 415D/S		112 Percent (≥ 500 MPa)	18.0
		Fe 500	500 MPa	108 Percent (≥ 545 MPa)	12.0
		Fe 500D/S		110 Percent (≥ 565 MPa)	16.0
		Fe 550	550 MPa	110 Percent (≥ 585 MPa)	10.0
		Fe 550 D		108 Percent (≥ 600 MPa)	14.5
		Fe 600	600 MPa	106 Percent (≥ 600 MPa)	10.0

NOTES

1 Elongation on a gauge length of 5.65√A, where A is the cross-sectional area of the test piece, when tested in accordance with [C-6B(5)].

2 For Seismic Zone III, IV and V; HYSD steel bars having a minimum elongation of 14.5 percent and conforming to other requirements of [C-6B(13)] shall be used.

3 For Seismic Zone III, IV and V; Structural steel maximum yield strength shall not exceed the specified minimum value by more than 20 percent and conforming to other requirements of [C-6B(13)] shall be used.

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6 STRUCTURAL ANALYSIS

This section is applicable to composite structures in which most of the members are composite. Where the structural behaviour is essentially that of reinforced or prestressed concrete, with only a few composite members, analysis provisions of as per good practice [C-6B(2)] or as per good practice [C-6B(16)] respectively, shall be used. Similarly, where the structural behaviour is essentially that of structural steel, with only a few composite members, analysis provisions of as per good practice [C-6B(3)] shall be used.

6.1 Modelling and Basic Assumptions

6.1.1 *Local Buckling of Plate Elements of Steel Section and Section Classification*

The types of elements that are encountered in steel sections are listed below:

a) Internal elements are elements attached along both longitudinal edges to other elements or to longitudinal stiffeners connected at suitable intervals to transverse stiffeners, for example, web of I-section and flanges and web of box section;

b) Outside elements or outstands are elements attached along only one of the longitudinal edges to an adjacent element, the other edge being free to displace out of plane for example, flange overhang of an I-section, the stem of T-section and legs of an angle section; and

c) Tapered elements may be treated as a flat elements having an average thickness as defined in the accepted standard [C-6B(17)].

6.1.2 The section strength at the ultimate limit state should be considered based on their ability to resist local buckling before full plastic strength is developed. In this respect, the structural steel sections may be classified as given in as per good practice [C-6B(3)]. Guidelines for sectional classification of composite sections with and without concrete encasements are given below:

a) *Composite Sections*

1) Steel compression elements (plates) in a composite section should be classified according to the least favourable class of steel elements in compression;

2) The class of a composite section depends on the direction of the bending moment at that section;

3) A steel compression element restrained by a reinforced concrete element through shear connectors may be placed in a more favourable class, after ensuring its improved local buckling resistance due to the above connection;

4) Plastic stress distribution over the cross section should be used for section classifications 1 and 2. In classification 3, the elastic stress distribution should be used taking into account the sequence of construction and the effects of creep and shrinkage;

5) For classification, design values of the strength of materials should be taken. Concrete in tension should be neglected. The stress distribution should be established for the gross cross-section of the steel web and the effective concrete flanges;

6) The tension capacity of steel reinforcements satisfying the ductility requirements specified in 5.4, only can be used in evaluating the strength in the hogging moment regions of the composite members. When the design resisting moment of the full or partial composite members is evaluated based on either simple rectangular stress block or parabolic stress block, the minimum area of steel reinforcement, A_{st}, used within the effective width of the concrete flange should satisfy the following:

$$A_{st} = A_c \, \alpha \, \frac{1}{\varepsilon^2} \frac{f_{ctm}}{f_{yk}} \sqrt{k_c}$$

where

ε = $\sqrt{250/f_y}$;

A_c = Effective area of the concrete flange;

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f_y = Nominal value of yield strength of structural steel, in MPa;

f_{yk} = Characteristic yield strength of tension reinforcement, in MPa;

f_{ctm} = Mean tensile strength of concrete, in MPa (Annex F);

k_c = $\frac{1}{(1+d_s/(2\,z_0))} + 0.3 \leq 1.0$;

α = 1.06 for compact section, 1.17 for plastic section;

d_s = thickness of concrete slab excluding haunch if any; and

z_0 = vertical distance between the centroid of the uncracked concrete slab and the centroid of the uncracked effective composite section using the modular ratio m for short-term loading.

7) The welded mesh should not be included in the effective section unless it has sufficient ductility before fracture when embedded in concrete; and

8) Account should be taken of the class of steel section at every stage of construction in the global elastic analysis.

b) *Composite Sections without Concrete Encasements*

1) A steel compression flange that is restrained from buckling by effective attachment to a concrete flange by shear connectors may be assumed to be in class 1 if the maximum spacing of the connectors is in accordance with **11.6.3**;

2) Other steel flanges and webs in compression in the composite beams should be classified on the basis of width to thickness ratios and susceptibility to local buckling of steel only unless they are also restrained by concrete as in columns. Accordingly, sections are categorized into three groups as indicated in as per good practice [C-6B(3)]; and

3) In beams, cross-sections with webs in Class 3 and flanges in Class 1 or 2 may be treated as an effective cross-section in Class 2 with an effective web in accordance with Fig. 1. The proportion of the web in compression should be replaced by a part of $20\varepsilon t_w$ adjacent to the compression flange, with another part of 20ε *t*w adjacent to the plastic neutral axis of the effective cross-section where ε = $\sqrt{250/f_y}$ and t_w is the thickness of the web.

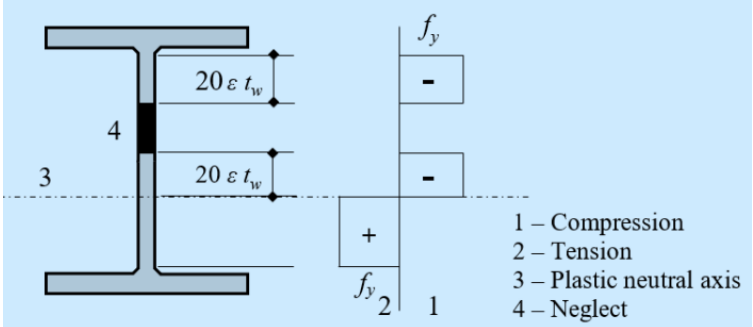

FIG. 1 EFFECTIVE CLASS 2 WEB IN BENDING

c) *Composite section with concrete encasements*

1) A steel outstand flange shall be classified as per Table 4; and

2) In partially concrete-encased I-section (Fig. 2), wherein the concrete is effectively attached to the web with stud or any reinforcement and covers on each side of the at least 80 percent of the flange overhang, the limiting overhang width of the flange to thickness ratio, b/t_f, for section classification may be taken as given in Table 4, where ε = $\sqrt{250/f_y}$.

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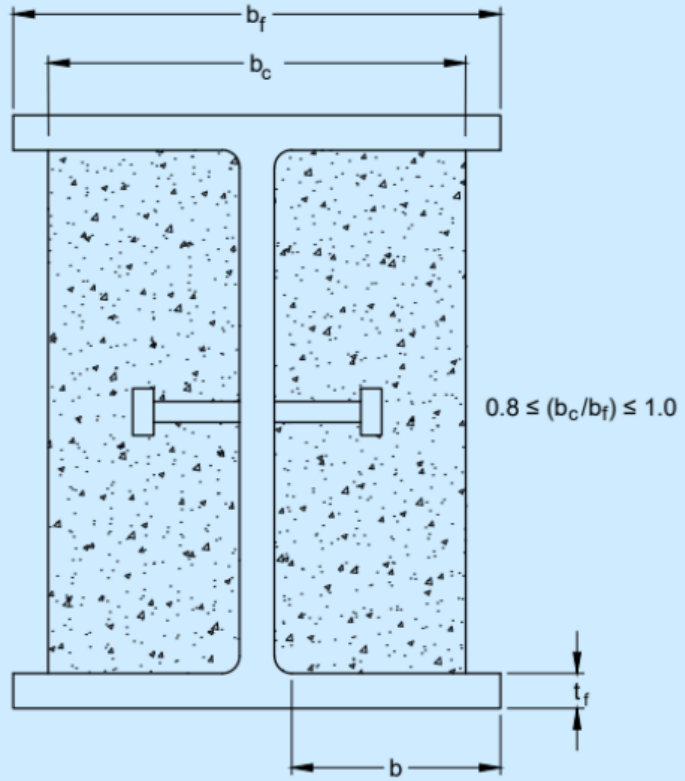

FIG. 2 PARTIALLY ENCASED I-SECTION

Table 4 Encased I Section Classification

[*Clause* 6.1.2(c)]

Sl No.	Section	Flange Overhang to Thickness Ratio (b/t_f)		
		Class 1 Plastic	Class 2 Compact	Class 3 Semi-compact
(1)	(2)	(3)	(4)	(5)
i)	Rolled section	9.4 ε	10.5 ε	15.7 ε
ii)	Welded section	8.4 ε	9.4 ε	13.6 ε

6.1.3 *Joints*

Joints in a composite structure may be modelled as:

a) Hinged, wherein the connection transfers only axial force and shear force, and negligible moment may be assumed to be transferred;

b) Rigid, wherein relative deformation between the members being joined is negligible due to the rigid nature of the joint, and the compatibility of deformation of the adjacent members at the joint may be assumed in the analysis; and

c) Semi-rigid, wherein the joint deformation under the force transferred across the joint should be modelled.

The effects of the deformation behaviour of the joints on the internal force resultants and deformations in the members and structure may be generally neglected. However, where such effects may be significant (such as in semi-rigid connections), they shall be considered as discussed in as per good practice [C-6B(3)].

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6.1.4 *Foundation Support*

Normally the support from the foundation may be assumed as hinged or rigid, depending on the soil condition and foundation type. Effects of support deformation may be considered, where significant.

6.1.5 *Deformation of the Structure*

6.1.5.1 If under the external actions, the deformations are small, then the analysis can be done by studying the equilibrium of the undeformed structure. Such an analysis is referred to as first-order analysis or linear elastic analysis.

6.1.5.2 If under the external actions, the deformations are large enough to significantly affect or modify the structural behaviour, then the analysis should be performed considering the equilibrium of the structure under deformed configuration. Such an analysis is referred to as second-order analysis.

6.1.5.3 First-order analysis is adequate if the increase in the internal forces or moments due to actions on the deformation from the first-order analysis is less than 10 percent. This may be assumed to be satisfied if the ratio of the P_{cr}/P_d is greater than or equal to 10, where P_{cr} is the load corresponding to elastic instability of the member or structure as a whole and P_d, is the factored design compressive load. In evaluating the elastic critical load, P_{cr}, effect of cracking and creep of concrete, deformation of the joint shall be considered.

6.1.6 *Imperfections*

Effects of imperfections in the member (local imperfections) and imperfections in the overall geometry of the structure should be considered if they are significant. The first order analysis does not account for these, and hence the design method used should consider these effects. The second-order analysis should include the imperfections in the modelling, if the ratio of elastic critical load to the factored design loads, $P_{cr}/P_d < 4.0$. If the second-order analysis does not include the imperfection effects (when $P_{cr}/P_d \geq 4.0$), the design method used should consider the imperfection effects. If the second-order analysis includes imperfection modelling, then the design method shall not include the effects of imperfections.

6.1.7 *Shear Lag and Effective Width of Flanges*

The effectiveness of steel and concrete flanges is reduced due to shear deformation in their plane either through rigorous analysis or by using effective width of the concrete as given in section **8.3.3** and that of steel flanges as given in as per good practice [C-6B(3)].

6.1.8 *Creep, Shrinkage and Temperature*

The effect of creep of concrete on the internal forces can be accounted by using effective modulus of elasticity of concrete, as recommended in as per good practice [C-6B(2)], by considering the creep and the age of concrete at the time of loading. The shrinkage strain may be taken as recommended in as per good practice [C-6B(2)] or as per specialized literature.

When permanent loads are imposed in several stages, one mean value of time may be used for calculating creep coefficient. Such a mean value may also be used for evaluating prestressing deformations if the age of the relevant portion of the concrete at the time of prestressing is more than 14 days.

If a prefabricated and prestressed concrete slab is made composite with structural steel subsequently, the creep and shrinkage values only after the composite action becomes effective are to be taken.

The effects of creep, shrinkage and differential temperature, in terms of internal forces, moments and deformation in determinate structure and indeterminate structures, where compatibility is not enforced, are referred to as ‘primary effects’. In indeterminate structures, wherein the compatibility of deformation is ensured in the analysis, the additional effects of enforcing compatibility are referred to as 'secondary effects'.

The effects of creep, shrinkage, and temperature are normally neglected in analysis for the ultimate limit state other than fatigue for composite members, in which steel sections used should meet either plastic or compact classification.

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The characteristic values of indirect actions due to the controlled imposed deformations (for example jacking of supports) may be calculated using the characteristic or nominal values of material properties in prestressed indeterminate composite beams and slabs unless a more accurate method is followed.

6.1.9 *Cracking of Concrete*

The effect of cracking of concrete in tension zones, as obtained from elastic analysis of the uncracked concrete model, may be considered by using effective stiffness of cracked concrete as given in as per good practice [C-6B(2)].

6.1.10 *Sequence of Construction*

Depending upon the type of construction (propped/unpropped, prestressed, precast, etc) the model for different stages of loading may differ. For the resultant internal forces, stresses and deflection at service load considering the effects of the staged construction may be obtained by superposing the analysis results for different loads obtained from different appropriate models for the stage of construction.

The effect of the sequence of construction may be neglected in structures made of plastic and compact steel sections and the model corresponding to the final stage of construction may be utilized for the ultimate limit state, other than fatigue.

6.2 Methods of Global Analysis

Different methods of global analysis, as discussed in the following sections, are permitted. The design and member strength check methods are chosen to be consistent with assumptions in analysis and actual global/member behaviour. Linear elastic first order method is most commonly used, but the second order effects and member imperfection effects, etc. Have to be taken care of in the design and member check stages.

6.2.1 *Linear Elastic Analysis*

6.2.1.1 Effect of cracking of concrete in tension, creep, shrinkage, temperature, prestressing and sequence of construction should be considered in the linear elastic analysis, as discussed in **6.1** and **7**. Forces and moments from the linear elastic analysis may be used even when the design is based on the ultimate limit state. In the case of continuous structures with hogging moments over supports, evaluation of the effective section at supports may be necessary, as mentioned in **8.3.3**. Appropriate load combinations with corresponding load factors are to be used to find out the maximum design values of moments and shears.

The stability of a structure as a whole against overturning shall be ensured under the limit state as per provisions of as per good practice [C-6B(2)]. The foundation components of the structure shall also be safe against sliding under adverse conditions of the applied characteristic loads. The following factor of safety shall be ensured:

a) *Overturning* — The stability of a structure as a whole against overturning shall be ensured so that the restoring moment shall not be less than the sum of 1.2 times the maximum overturning moment due to the characteristic dead load and 1.4 times the maximum overturning moment due to the characteristic imposed loads. In cases where dead load provides the restoring moment, only 0.9 times the characteristic dead load as per good practice [C-6B(2)] shall be considered. Restoring moments due to imposed loads shall be ignored;

b) *Sliding* — The structure shall have a factor against sliding of not less than 1.4 under the most adverse combination of the applied characteristic forces. In this case also, only 0.9 times the characteristic dead load shall be taken into account if it assists in resisting sliding; and

c) *Instability* — Linear elastic instability analysis of member and the global system is possible, wherein first order assumptions and the resulting eigenvalue problem leads to eigenvalues as the instability load,

6.2.1.2 *Redistribution of moments from first order analysis*

Bending moments obtained from the linear elastic analysis of indeterminate slabs, beams and frames in composite

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structures in buildings may be redistributed to a limited extent as given below while ensuring the equilibrium is still satisfied:

a) concrete members are subjected primarily to flexure in accordance with as per good practice [C-6B(2)];

b) steel members satisfy the plastic section requirements of as per good practice [C-6B(3)];

c) composite members have partial or full shear connection as given in **11**;

d) redistributed internal forces and moments should satisfy equilibrium;

e) inelastic behaviour of the material, and local and lateral-torsional buckling should allow such a redistribution;

f) at least one end of the beam should be connected by a rigid or full-strength joint to the adjacent member; and

g) each span is of uniform depth.

Table 5 gives the maximum percentage reduction in the hogging bending moments obtained from linear elastic global analysis, in composite beams, unless it is verified that the plastic rotation capacity permits a higher value.

Table 5 Limits to Percentage Redistribution of Hogging Bending Moments Obtained from Linear Elastic Analysis

(*Clause* 6.2.1.2)

Sl No.	Classification of the Cross-Section in the Hogging Moment Region	Plastic	Compact	Semi-compact	Slender
(1)	(2)	(3)	(4)	(5)	(6)
i)	For un-cracked section analysis moment	40	30	20	10
ii)	For cracked section analysis moment	25	15	10	0

NOTES

1 For grades of steel having yield strength greater than 360 MPa, redistribution is applicable to beams with plastic or compact cross-sections. In the redistribution, the reduction in hogging moment should not exceed the values for compact sections given in Table 5, unless it is demonstrated that rotation capacity permits a higher value.

2 The limits in Table 5 for semi-compact and slender sections relate to bending moments to be resisted by composite members and moments to be resisted by bare steel section should not be redistributed.

6.2.2 *Non-linear Global Analysis*

The non-linear analysis may be performed as specified in as per good practice [C-6B(3)]. The non-linear behaviour of the shear connection shall be taken into account. Effects of deformed geometry of the structure, imperfections, joint and foundations, creep, shrinkage, temperature, stages of loading shall be taken into account as discussed in **6.1**.

6.2.3 *Rigid Plastic Analysis*

Rigid plastic global analysis may be used in composite structures to obtain ultimate limit state design values other than fatigue if the conditions given below are satisfied. Second-order effects, imperfections, creep, shrinkage, temperature effects, etc. need not be considered in the rigid plastic analysis. The effect of alternating plasticity due to variable loading need not be normally considered in buildings:

a) All the members and joints of the frame are either composite or structural steel;

b) Material and cross-section should satisfy requirements of plastic analysis as given in [C-6B(3)];

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c) Joints are able to sustain the plastic resisting moments over a large plastic hinge rotation;

d) Critical plastic hinge location shall satisfy the following requirements:

   1) Cross-section of the structural steel section shall be symmetric about the plane of bending;

   2) restraints are provided to prevent lateral torsional buckling;

   3) lateral restraint is provided to the compression flange at all critical plastic hinge sections under any loading;

   4) The rotation capacity of the section, considering any axial compression at the section, should be adequate to permit the plastic hinge rotation to form the plastic collapse mechanism; and

   5) When rotation requirements are not calculated and ensured, all effective sections at potential plastic hinge locations should be of plastic classification;

e) The plastic hinge rotation capacity in composite sections may be assumed to be adequate where:

   1) the yield strength of structural steel does not exceed 450 MPa;

   2) the concrete slab is of normal density concrete, having a characteristic strength within the range of 25 MPa to 50 MPa;

   3) contribution of reinforced concrete encasement in compression is neglected in calculating design moment resistance;

   4) all effective steel sections at potential plastic hinge rotation should be plastic, and all the other sections should be plastic or compact;

   5) all beam connections to adjacent beams/columns should have adequate plastic rotation capacity or should have a design resisting moment of at least 1.2 times the design plastic resistance of the adjacent beam;

   6) structural steel compression flange at plastic hinge location should be laterally restrained;

   7) adjacent span lengths do not differ by more than 50 percent of the shorter span length;

   8) the end span length is not more than 115 percent of the adjacent span length;

   9) at plastic sections with the concrete slab is under compression, not more than 15 percent of the total depth of the section shall be under compression at the ultimate moment unless the plastic hinge at the section is the last to form (not requiring much plastic hinge rotation);

   10) plastic hinges shall not be located in composite columns unless plastic rotation capacity can be verified;

   11) the lateral support to the compression flange shall be located within half the depth of the steel section from the plastic hinge section; and

   12) sections away from plastic hinge regions should satisfy at least compact section requirements.

7 LIMIT STATE DESIGN

7.1 General

In the limit state design method, the structure shall be designed to withstand safely all loads likely to act on it throughout its life. The objective of the design is to arrive at a structure that will remain fit for use during its life with acceptable target reliability. In other words, the probability of a limit state being reached during its lifetime should be very low. The acceptable limit for the safety and serviceability requirements before failure occurs is called a limit state. In general, the structure shall be designed on the basis of the most critical limit state and shall be checked for other limit states.

For satisfactory functioning of a structure during its design life conditions, stipulations and directives laid down in [C-6B(1)] shall be adequately satisfied for all steel-concrete composite structures.

7.1.1 *Effect of Construction Sequence*

The linear elastic method is valid for the analysis of the structure after considering load history, sequence of

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concrete casting and development of composite strength. In the case of propped construction, most of the initial dead load is resisted through the beam-prop system and the steel beam remains basically unstressed at this stage. In the case of un-propped construction, the steel beams alone have to carry the initial dead load and construction loads. Consequently, stresses and deflections at this stage shall not exceed specified design limits. The necessary distinction has to be made in the analysis about the stage of loading and the effectiveness of the system resisting the load. In the ultimate limit state, however, this distinction is not necessary while checking for flexural resistance. For the design of steel components and concrete deck, stipulations of as per good practice [C-6B(1)] and this Subsection shall be applied.

7.1.2 The sequence of construction should be considered as an integral part of the design process, for example, when calculating the stresses under serviceability limit criteria. The sequence and method of construction need to be mentioned in the drawings. Additional information such as the position of construction joints, if any, may also be included in the drawings.

Where the composite section is loaded before the concrete has attained its 28 days characteristic cube strength, the elastic properties and limiting compressive strength of the concrete, and the nominal strengths of shear connectors should be based upon the cube strength of the concrete at the time of loading. Where a partially cast slab is assumed to act compositely, the shear connection should be designed for this condition as well as for the final condition. In calculating deflections, consideration should be given to the sequence of construction and, where appropriate, proper account should be taken of the pre-camber of steel section, the deflections of the steel section due to loads applied to it prior to the development of composite action and of partial composite action where deck slabs are cast in stages. In execution, the rate and sequence of concreting should be required to be such that partly matured concrete is not damaged as a result of limited composite action occurring from the deformation of the steel beams under subsequent concreting operations.

7.2 Limit States

A composite structure or part of it is considered unfit for use when it exceeds a particular state called the limit state, beyond which it infringes on one of the criteria governing its performance or use. The limit states can be classified into the following categories:

a) Ultimate limit state;

b) Serviceability limit state; and

c) Fatigue limit state.

7.2.1 *Ultimate Limit State*

It is the state when under the worst combination of factored loads the structure or its components either reach design strength or becomes unstable. Both stability and strength need to be checked under the ultimate limit state.

In steel-concrete composite structures used in buildings or general constructions, the significant ultimate limit states to be considered are as follows:

a) Collapse due to flexural, shear or bearing failure of one or more critical sections or components;

b) Collapse due to horizontal shear failure at the interface between the steel beam and the concrete slab or composite slab system involving concrete slab and embossed profiled sheets;

c) Collapse due to the vertical separation of the concrete/composite slab from the steel beams; and

d) Collapse due to shear failure between steel and concrete components of the composite column or due to buckling of both fully/partially concrete encased steel columns or concrete filled hollow sections used as columns.

7.2.2 *Serviceability Limit States*

It is the state at which any of the following conditions occur during the loads encountered under construction and service:

a) Stress in structural steel has reached the prescribed limit;

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b) Stress in concrete has reached the prescribed limit;

c) Deflection of a structure or its component reaches the prescribed limit;

d) Concrete crack width reaches the prescribed limit;

e) Slip at the interface between steel and concrete exceeds the prescribed limit;

f) Vibration becomes excessive, especially at overhangs;

g) Excessive corrosion affecting the durability of the structure; and

h) Unacceptable effects due to fire.

7.2.3 *Fatigue Limit State*

It is the state at which stress range due to the application of imposed loads reaches the prescribed limit, corresponding to the number of load cycles and detail configuration.

7.3 Design Philosophy

For ensuring the design objectives, the design should be based on the characteristic values for material strengths (resistance) and applied loads (actions), which take into account the probability of variations in the material strengths and in the loads to be supported. The characteristic values should be based on statistical data, if available. Where such data is not available, they should be based on experiments. The design values are derived from the characteristic values through the use of partial safety factors, both for material strengths and for loads. In the absence of special considerations, these factors should have the values given in this section according to the material, the type of load and the limit state being considered. The reliability of the design is achieved by ensuring that:

$$\text{Design Action (Load)} \leq \text{Design Strength (Resistance)}$$

Design action refers to the external actions or load which act on the structure and the design resistance refers to the maximum resistance the structure and its components provides to resist the actions without causing failure of the structure and its components or causing hindrances to the smooth operation of the structure for which it is intended.

7.4 Design Actions (Loads)

7.4.1 *Actions* (*Loads*) *are Classified into Three Main Categories*:

a) *Permanent or dead loads* — Loads due to self-weight of structural and non-structural components, fittings, ancillaries, and fixed equipment etc. Dead loads shall be calculated on the basis of unit weights which shall be established taking into consideration the materials specified for construction. Alternatively, the dead loads may be calculated on the basis of nominal dimensions and unit weights of materials given in (*see* Part C 'Structural Design/Section 1 Loads and Load effects' of this NBCS);

b) *Variable or imposed loads* — Construction and service stage loads such as imposed loads (for example, crane loads, impact, etc.), wind loads, snow loads and other loads shall be assumed in accordance with (*see* Part C 'Structural Design/Section 1 Loads and Load effects' of this NBCS) and the earthquake forces shall be calculated in accordance with the same; and

c) *Accidental loads* — Accidental loads are actions or loads expected due to explosions and the impact of vehicles, etc. The characteristic values of accidental loads generally correspond to the value specified by relevant code, standard or client. Design for the accidental load is generally not required in every building unless it is required by the client or approving authority, in which case, general recommendations given in as per good practice [C-6B(3)] or specialist literature shall be followed.

7.4.2 Other than the actions due to the externally applied loads as given in **7.4.1**, if the effects of shrinkage, creep and temperature are liable to affect material safety and serviceability of the structure, these shall be accounted for in the design calculations (*see* Part C 'Structural Design/Section 1 Loads and Load effects' of this NBCS) .

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7.4.3 *Load Combination*

The different combinations of loads considered shall be as given in (*see* Part C 'Structural Design/Section 1 Loads and Load effects' of this NBCS). For each combination, different partial safety factors for loads are assigned to different loads, γ_f to account for:

a) Possibility of unfavourable deviation of the load from the characteristic value;

b) Possibility of inaccurate assessment of the load;

c) Uncertainty in the assessment of effects of the load; and

d) Uncertainty in the assessment of the limit states being considered.

The loads or load effects shall be multiplied by the relevant γ_f factors, given in Table 6, to calculate the design loads or design load effects.

Table 6 Partial Safety Factor for Loads, γ_f for Limit States

(*Clause* 7.4.3 *and* 9.3.2.1)

Sl No.	Combination	Limit State of Strength					Limit State of Serviceability			
		DL	LL [2]		WL/EL	AL	DL	LL [2]		WL/EL
			Leading	Accompanying				Leading	Accompanying	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
i)	DL+LL+CL	1.5	1.5	1.05	–	–	1.0	1.0	1.0	–
ii)	DL+LL+CL+WL/EL	1.2	1.2	1.05	0.6	–				
		1.2	1.2	0.53	1.2		1.0	0.8	0.8	0.8
iii)	DL+WL/EL	1.5 (0.9)[1]	–	–	1.5	–	1.0	–	–	1.0
iv)	DL+ER	1.2 (0.9)[1]	1.2	–	–	–	–	–	–	–
v)	DL+LL+AL	1.0	0.35	0.35	–	1.0	–	–	–	–

[1] This value is to be considered when the dead load contribution to stability against overturning is critical or the dead load causes a reduction in stress due to other loads.

[2] When the action of different imposed loads is simultaneously considered, the leading imposed load shall be considered to be the one causing the higher load effects in the member/section.

NOTE — The effects of actions (loads) in terms of stresses or stress resultants may be obtained from an appropriate method of analysis.

Abbreviations: DL= Dead Load, LL= Imposed Load, WL= Wind Load, CL= Crane Load (Vertical / horizontal), AL=Accidental Load, ER= Erection Load, EL= Earthquake Load.

7.5 Material Strength and Partial Safety Factor

The design strength, S_d of a structural component, is obtained as given below from nominal ultimate strength, S_u, and partial safety factors for material strength, γ_m.

$$S_d = S_u/\gamma_m$$

where the partial safety factor for materials, γ_m, (*see* Table 7), account for:

a) Possibility of unfavorable deviation of material strength from the characteristic value;

[1] This value is to be considered when the dead load contribution to stability against overturning is critical or the dead load causes a reduction in stress due to other loads.

[2] When the action of different imposed loads is simultaneously considered, the leading imposed load shall be considered to be the one causing the higher load effects in the member/section.

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b) Possibility of unfavorable variation of member sizes;

c) Possibility of unfavorable reduction in member strength due to fabrication and tolerances; and

d) Uncertainty in the calculation of the strength of the members.

Table 7 Partial Safety Factor for Material γ_m

(*Clauses* 7.5, 8.3.5.1, 8.3.6.1 *and* 9.1.2)

Sl No.	Material	Item	Partial Safety Factor γ_m
(1)	(2)	(3)	(4)
i)	Structural steel, and steel sheeting	Resistance against yield stress, γ_{m0}	1.10
		Resistance of member to buckling, γ_{m0}	1.10
		Resistance against ultimate stress γ_{m1}	1.25
ii)	Reinforcement	Reinforcement resistance against yield stress,γ_k	1.15
iii)	Shear connectors	Yield stress, γ_{mv}	1.25
iv)	Connections	Bolts-friction type, γ_{mf}	1.25
		Bolts-bearing type, γ_{mb}	1.25
		Rivets, γ_{mr}	1.25
		Welds for shop fabrication, γ_{mw}	1.25
		Welds for site fabrication, γ_{mw}	1.50
v)	Concrete	For basic and seismic load combinations, γ_c	1.50

NOTE — The partial safety factor for resistance for all materials in case of accidental loads = 1.0

8 DESIGN FOR ULTIMATE LIMIT STATES

8.1 Composite Beams

Composite beams shall be checked for the following:

a) resistance to flexure;

b) resistance to lateral-torsional buckling;

c) resistance to web shear buckling;

d) resistance to longitudinal shear (between steel and concrete components).

8.1.1 *Assumptions*

Design for the limit state of collapse in flexure shall be based on the assumptions given below:

a) Plane sections normal to the neutral axis remain plane and normal after bending;

b) The maximum strain in concrete at the outermost compression fiber at collapse is taken as 0.003 5 in bending as per good practice [C-6B(2)];

c) The tensile strength of the concrete is ignored;

d) The stress-strain curve for the reinforcing steel shall be assumed to be the same as given in as per good practice [C-6B(2)];

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e) The properties of structural steel shall be taken as given in as per good practice [C-6B(3)]; and

f) The stress-strain curve for concrete may be taken to be the same as given in as per good practice [C-6B(2)]. The corresponding parabolic stress block may be used to evaluate the design bending resistance of composite beams and slabs. The bending resistance of a composite beam for the value of concrete stress using an equivalent rectangular stress block, which simplifies the design resistance equation, may be used as an alternative method. Both methods and the design resistance equations are given in Annex B.

NOTE — Critical cross-sections include the following:

a) the sections where the bending moment is maximum;

b) the section at the faces of the support;

c) sections adjacent to concentrated force or reactions; and

d) locations where a sudden change of cross-section occurs; (ratio of greater to the lesser resisting moment at adjacent sections exceeds 1.2) (changes due to cracking of concrete not to be included).

8.1.2 *Ultimate Bending Resistance*

For determining the position of the plastic neutral axis and the ultimate moment of resistance of composite beams, the guidelines given in Annex B may be used.

8.1.2.1 *General*

Composite constructions of types: (a) Top flange of steel beam supporting the concrete slab (Fig. 3) and (b) composite slab resting on structural steel beam (Fig. 4) are included in this design philosophy. A typical composite beam arrangement is shown in Fig. 3, where cast in place or precast RC slab is directly resting on the steel beam. Fig. 4 depict a composite slab consisting of RC and profiled deck sheet with or without embossments) and supported on a structural steel beam, parallel and perpendicular to the steel beam, respectively. The neutral axis may be in the concrete slab, in the top flange of the steel section, or in the web of the steel section.

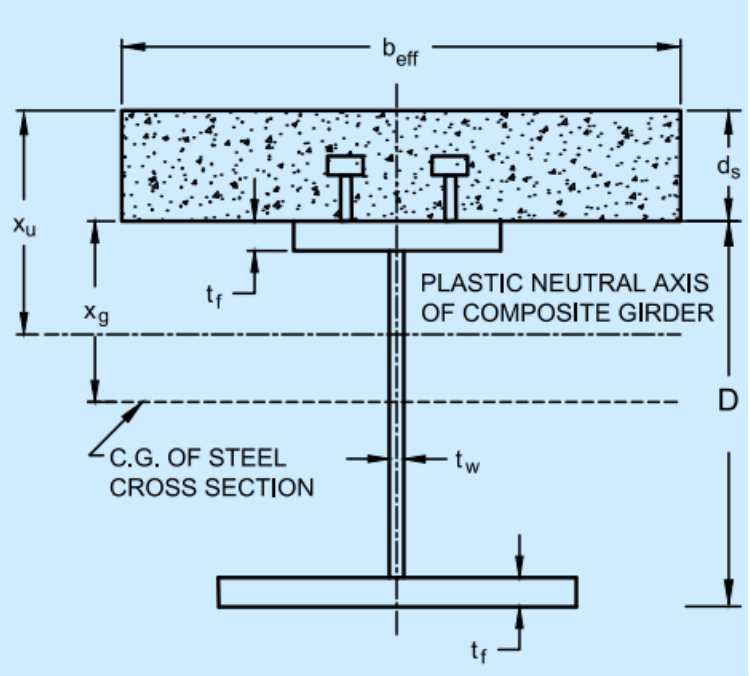

FIG. 3 TYPICAL COMPOSITE BEAMS WITHOUT PROFILED DECKING SHEET

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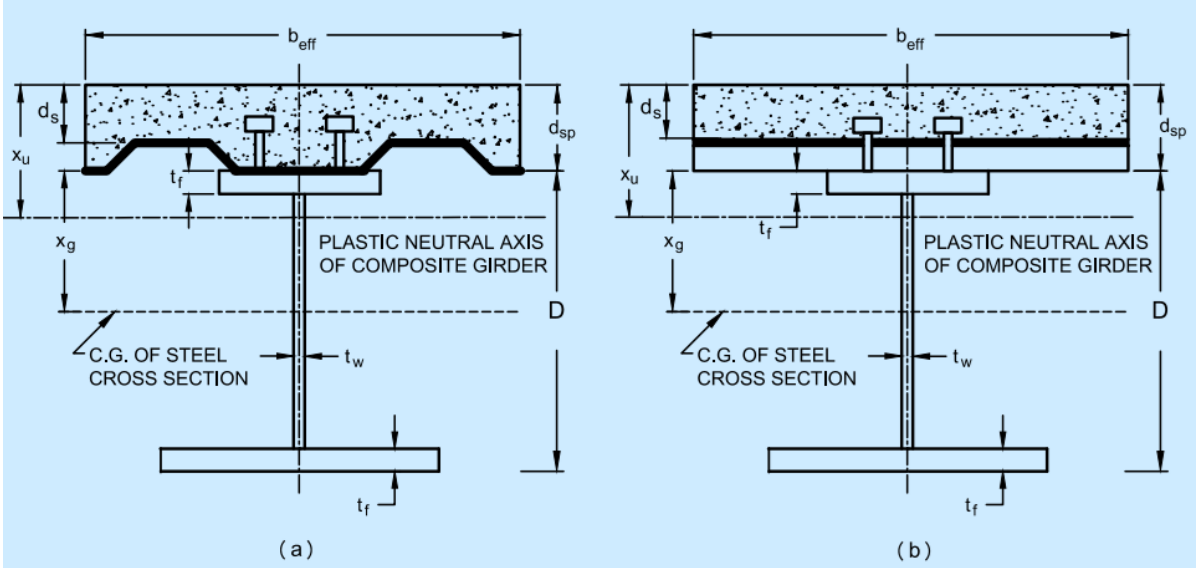

FIG. 4 TYPICAL COMPOSITE BEAMS WITH PROFILED DECKING SHEET (A) RIBS PARALLEL TO BEAMS (B) RIBS PERPENDICULAR TO BEAM

Bending moments and shears due to the application of factored loads may be analyzed in indeterminate structures by elastic theory assuming the concrete in the slab as un-cracked and unreinforced.

Hogging moments over internal supports as calculated above should be checked against section resistance assuming steel beam acting integrally with concrete (considering un-cracked and unreinforced). If the flexural tensile stress in concrete thus calculated exceeds the tensile strength of concrete, $f_{ctk,\ 0.05}$ as in Annex F then a new analysis, neglecting concrete but including reinforcements over the effective width of the slab (*see* **8.3.3**) and over 15 percent of the span on each side of the support should be done to calculate the required design resistance, provided adjacent spans do not differ appreciably.

The redistribution of elastic analysis moment of indeterminate structure, as recommended in **6.2.1.2**, is allowed to arrive at the design moments in the case of plastic beams. Design for plastic analysis moments (as given in **6.2.3**) is permitted in the case of beams classified as plastic.

The bottom flange of the beam in the hogging moment zone should be adequately braced against lateral buckling. Otherwise, lateral buckling resistance has to be evaluated as recommended in **B-4**.

8.2 Precast Slab on Steel Beam

The use of precast slab, both full depth and partial depth, is allowed for composite construction as one of the components of composite beams. Precast slabs shall be erected and connected to the steel section so as to ensure composite action along with the steel beam.

8.2.1 *Full Depth Precast Slab*

Full-depth precast concrete deck panels may be used for new construction as well as for the replacement of deteriorated concrete decks on existing steel beams so as to obtain composite action. This shall be ensured by proper shear connection during the erection of these precast panels. The typical requirements for these types of beams are as given below:

a) Panels shall either span the full width of the concreting deck or shall be in lengths that span between two or more parallel beams. The minimum thickness of the slab shall be 150 mm;

b) The panels shall be connected to the beams using shear connectors in pockets, which consist of mechanical connectors, such as shear studs encapsulated in non-shrinking grouted pockets. These connections cause the panels to develop composite action with the beams; and

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c) The contact between the precas t panels at their longitudinal edge should ensure transfer of compression between the panels, necessary for composite action.

8.2.2 *Partial Depth Precast Slab*

Partial depth precast concrete deck panels are generally thin RCC/prestressed concrete panels that span between beams and also serve as forms/shuttering for the cast-in-place concrete deck. The typical geometrical parameters that govern the use of these panels as a composite unit for the floor system are as given below:

a) The minimum thickness of the precast panels shall be 75 mm;

b) As a composite floor system, the cast in-situ concrete and the partial-depth panels together create the total thickness of the slab, with the panel reinforcing steel serving as the positive moment reinforcement in the bending direction of the combined slab;

c) Dimensions of the precast panels shall be chosen from consideration of easy handling, ease of lifting by cranes and for catering to the construction loads, including load of wet cast in-situ concrete;

d) Partial-depth panels must be capable of developing sufficient composite action with the cast *in-situ* concrete to be an effective floor system;

e) To ensure full bond between the cast *in-situ* concrete and precast panels, it is recommended that the top surface of the precast panel is intentionally roughened while casting it and is cleaned by removing the laitance or other contaminants on the surface and other measures may be taken, before the placement of the cast *in-situ* concrete so as to ensure a good bond between precast and cast in place concrete; and

f) After the precast panels are in place, the top layer of the reinforcing steel shall be placed over it, and the cast-in-situ concrete shall be placed on top of the panels.

8.2.3 *General Design Principles*

The design of precast slabs is based on the following principles:

a) The precast slab together with any *in-situ* concrete (for partial depth slab) should be designed as continuous in both the longitudinal and the transverse direction;

b) The joints between slabs should be designed to transmit membrane forces as well as bending moment and shear forces;

c) The effective width of precast slab in the composite beam action shall be calculated as per **8.3.3**;

d) The design principles of composite beams involving either full depth or partial depth precast slabs are similar to standard composite decks using cast *in-situ* reinforced concrete;

e) Vertical shear check of the composite beam shall be done as per **8.3.6**; and

f) For serviceability limit states, guidelines given in **7.2.2** and **9** shall be followed.

8.2.4 *Joints between Steel Beam and Precast Concrete Slab*

a) Where precast slabs are supported on steel beams without bedding, the influence of the vertical tolerances of the bearing surfaces shall be considered; and

b) The shear transfer between steel flange and precast concrete through mechanical shear connector shall be designed as per **11** with the following precautions:

1) If shear connectors are welded to the steel beam projecting into the recesses within the slabs or joints between slabs, which are filled with concrete after erection, the detailing and the properties of the concrete should be such that it can be cast properly.

2) The minimum infill around the shear connectors should be at least 25 mm.

3) If shear connectors are arranged in groups, sufficient reinforcement should be provided near each group to prevent premature local failure in either the precast or the *in-situ* concrete.

4) Special provision for protection against corrosion shall be adopted, wherein the steel flange under

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precast slabs without bedding should have the same corrosion protection as the rest of the steelwork in addition to the top coating provided after erection.

5) Bedding with the purpose of protecting against corrosion may be designed to be non-load bearing.

8.2.5 *Joints between Precast members*

The critical sections of members close to joints should be designed to resist the most adverse combination of shear, axial force and bending caused by the ultimate vertical and horizontal forces. When the design of the precast members is based on the assumption that the joint between them is not capable of transmitting bending moment (*see* **8.3.4**), suitable precautions should be taken to ensure that if any crack developed is not excessive, and reduce the shear or axial force resistance of the member and should not be aesthetically or functionally objectionable.

Where space is left between two or more precast units, to be filled later with *in-situ* concrete or non-shrink grout, the gap should be large enough for easy placement and adequate compaction of the filling material to fill the gap completely.

8.2.6 *Structural Connection at Joints*

When designing and detailing the connections across joints between precast members, the overall stability of the structure, including its stability during construction, shall be considered. A typical joint connection is shown in Fig. 5.

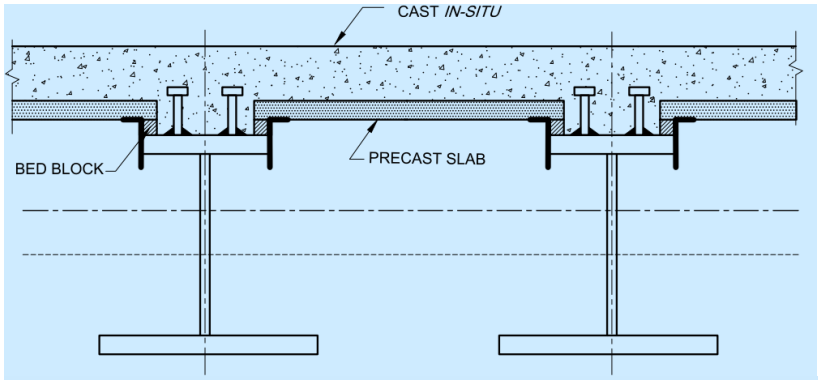

FIG. 5 TYPICAL JOINT CONNECTION FOR PARTIAL DEPTH PRECAST SLAB

8.3 Design Method

Connections should, where possible, be designed in accordance with the generally accepted methods applicable to reinforced concrete or structural steel.

8.3.1 *General Considerations in Design Details*

In addition to ultimate resistance requirements, the following should be considered:

a) *Protection* — Connections should be designed so that the standard of protection against weather and corrosion required for the rest of the structure is sufficiently maintained;

b) *Appearance* — Where connections are to be exposed, they should be designed to achieve and maintain the quality of appearance required for the rest of the structure; and

c) *Manufacture, assembly, and erection* — Methods of manufacture and erection should be considered during design. Care should be taken during erection and the following precautions are mandatory:

1) Where projecting bars or sections are required, they should be kept to a minimum and made as simple as possible. The lengths of such projections should be not more than necessary;

2) Fixing devices should be located in concrete sections of adequate strength;

3) The practicability of both casting and assembly should be considered; and

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4) Most connections require the introduction of suitable jointing material. Sufficient space should be allowed in the design for such material to ensure that the proper filling of the joint is possible.

8.3.2 *Reinforcement Continuity at Joint*

Where continuity of reinforcement is required through the connection, the joining method used should be such that the assumptions made in analyzing the structure and critical sections are realized. The standard methods applicable for achieving continuity of reinforcements are lapping, butt welding of bars and grout couplers.

8.3.3 *Effective Width of Concrete Slab*

a) For resistance calculation, the width of the slab, which is effective as the compression flange of the composite beam on each side of the steel beam, b_{eff}, shall be as per the equation given below:

$$b_{eff} = b_o + (b_{e1} + b_{e2}) = b_o + \frac{L_e}{4} \leq b_o + (b_1 + b_2)$$

where

b_o = centre to centre between the outer row of shear connectors;

b_{e1}, b_{e2} = are the value of the effective width of the concrete flange on each side of the web and taken as $L_e/8$ but not greater than the geometric width b_1 and b_2, respectively as shown in Fig. 6;

$L_e/8 \leq b_1$ and b_2 (*respectively*);

L_e = the effective equivalent span length of the corresponding composite beam;

centre-to-centre distance between the supports for simply supported beam; and

as shown in Fig. 7 in a continuously supported beam.

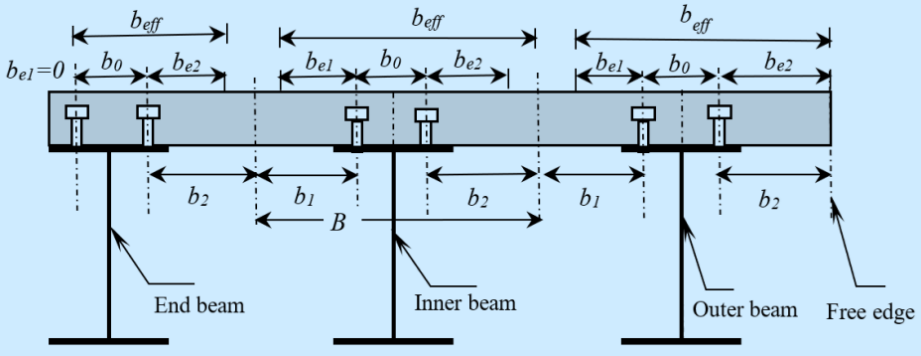

FIG. 6 EFFECTIVE WIDTHS FOR CONCRETE FLANGE OF COMPOSITE BEAMS

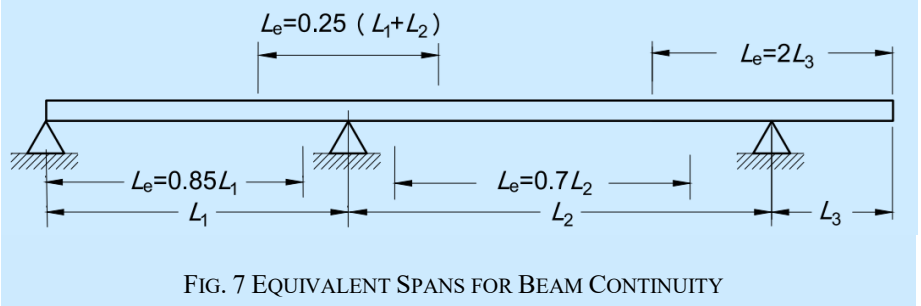

FIG. 7 EQUIVALENT SPANS FOR BEAM CONTINUITY

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Reinforcements placed parallel to the steel beam within the effective width of the concrete slab should be considered for hogging moment resistance calculation of the composite beam at the continuous support.

b) The effective width at an end beam may be determined as:

$$b_{eff} = b_0 + \beta_{e1}\, b_{e1} + \beta_{e2}\, b_{e2}$$

where

β_{e1} = $0.55 + 0.025\,(L_e/b_{e1}) \leq 1.0$

β_{e2} = $0.55 + 0.025\,(L_e/b_{e2}) \leq 1.0$

L_e = Equivalent effective end span; and

b_{e1} = 0 as shown in Fig. 6.

8.3.4 *Effective Cross Section for Strength Calculation*

In calculating the strength of the cross section of the composite beams, the following should be considered:

a) For Sagging Moment — Concrete in the effective width and structural steel beam to be included but not the steel reinforcements in the effective width of concrete; and

b) For Hogging Moment — Concrete to be neglected but longitudinal steel reinforcement along the beam length within the effective width of the concrete in tension and structural steel beam to be included.

8.3.5 *Design of Structure for Bending Moment*

8.3.5.1 The factored design flexural strength, *M*d, in a beam and the external action, *M,* shall satisfy

$$M \leq M_d$$

where

M_d = M_n / γ_{m0} = design flexural resistance calculated as given below;

γ_{m0} = partial safety factor against flexural failure (Table 7); and

M_n = nominal ultimate flexural resistance.

8.3.5.2 Considering local buckling, sections are to be analyzed as plastic, compact or semi-compact with the following additional consideration:

a) Load history and development in composite action are to be taken into consideration with appropriate values of the modular ratio, *m* at each stage, and stresses and deflections are to be the summation of values over successive stages;

b) Determination of bending resistance for a beam before composite action has set in or during the construction stage shall be done as per good practice [C-6B(3)];

c) The structural steel reinforcement and concrete fully interact with full shear transfer between the concrete and structural steel under composite action;

d) The effective width of concrete in composite action may be as mentioned in **8.3.3**;

e) For calculating the bending resistance of plastic, compact and semi-compact composite beams, the procedure presented in Annex B may be followed; and

f) The partial shear connection may be used in plastic and compact sections either for attaining economy without losing much in the moment capacity of the composite section or under conditions where the number of shear connectors required for full shear interactions cannot be provided. Requirements on the method of evaluating the bending resistance of partial shear connection are presented in **B-1.2**.

The bending moment and shear force distribution in continuous beams for secondary, as well as primary moment resistant frames/beams have to be determined by structural analysis.

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8.3.5.3 *Design of structure for effect of lateral buckling on moment resistance*

Lateral buckling may govern the design under the following conditions:

a) At the construction stage, in the top flange closer to mid-span in both simply supported and continuous beams; and

b) At the construction and composite stage, in the bottom flange closer to support in continuous beams.

At the construction stage, the effect of lateral buckling on the bottom flange in a continuous beam shall be taken care of by considering cantilever action up to the point of inflection from the support.

If required, suitable horizontal bracings or members may be provided at the bottom flange to reduce the effective length of the compression flange near support. For beams that are provided with such bracings or members giving effective lateral restraint to the compression flange at intervals along the span, the effective lateral restraint shall be capable of resisting a force equal to 2.5 percent of the maximum force in the compression flange taken as divided equally between the numbers of points at which the restraint in bracing members occur. In beams supporting composite slabs, wherein the sheeting is welded to the top flange before concreting is done, the lateral restraint of the deck sheet may be considered.

The effective laterally unsupported length of compression flange of steel sections in sagging and hogging moment regions may be taken as given in as per good practice [C-6B(3)]. In sagging moment segments of composite beams, the concrete slab may be assumed to provide lateral support, provided an adequate shear connection between the slab and the beam is provided. In the hogging moment regions of composite beams, the effective length considering the rotational restraint provided by the slab to the top tension flange may be taken into account as given in **B-2**.

8.3.5.4 *Simplified design of continuous beams in buildings without lateral bracings*

Continuous composite beams (or composite beams within a frame) with plastic, compact, or semi-compact sections may be designed without any additional lateral bracings to the compression flange when the following conditions are satisfied:

a) The concrete slab on top should be adequately attached with shear connectors to the structural steel section and to at least one another parallel member top flange to facilitate inverted U-frame action that rotationally restrains bottom flange lateral buckling at hogging moment sections;

b) Adjacent span lengths of continuous beams should not differ by more than 20 percent of the shorter span length. Cantilever span length should not exceed 15 percent of the adjacent span length;

c) The loading on each span should be uniformly distributed, with the permanent design loads exceeding 40 percent of the total design load;

d) At each support of steel member, the bottom flange should be laterally restrained with a web bearing stiffener at that location; and

e) The maximum depth of steel beam sections should not exceed the value given in Table 8.

Table 8 Maximum Depth of Steel Beams in Composite Construction (In mm)

[*Clause* 8.3.5.4 (e)]

Sl No.	Type of Member	Nominal Yield Strength Limit of Structural Steel (MPa)							
		250		300		350		> 400	
		Un-cased	En-cased	Un-cased	En-cased	Un-cased	En-cased	Un-cased	En-cased
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
i)	I Section	800	1 000	700	900	650	850	500	650
ii)	Parallel flange	600	800	550	750	400	600	270	430

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8.3.6 *Design of Structural Steel Web against Vertical Shear Force*

8.3.6.1 The factored design shear force, V_d, in a composite beam due to external action, V, shall satisfy:

$$V \leq V_d$$

where

Vd = design shear resistance calculated as given below.

$$V_d = V_n/\gamma_{m0}$$

γ_{m0}= partial safety factor as mentioned in Table 7.

V_n = nominal ultimate shear resistance.

The vertical shear force is assumed to be resisted by the web of the steel section only unless the value for contribution from the reinforced concrete part of the beam has been established. The nominal shear resistance, Vn may be governed by plastic shear resistance or strength of the web or governed by shear buckling as given below:

a) *Plastic Shear Resistance*

The nominal plastic shear resistance of composite beams under pure shear shall be calculated as indicated in as per good practice [C-6B(3)], disregarding the contribution of the concrete slab. The shear resistance for I-sections, channels both for major axis bending and minor axis bending as well as for rectangular and circular hollow sections of uniform thickness shall be as per good practice [C-6B(3)].

b) *Shear Buckling Resistance*

Resistance to shear buckling shall be verified as specified in as per good practice [C-6B(3)]. The nominal shear resistance, Vn, of webs with or without intermediate stiffeners as governed by buckling may be evaluated as detailed in as per good practice [C-6B(3)] either using the simple post-critical method or tension field method. The contribution of encasing concrete to shear resistance may be considered and that of shear reinforcement can be considered provided they consist of closed-loop stirrups. The total shear may be shared between the structural steel section and encasing concrete in the ratio of their contribution to plastic moment capacity.

8.3.6.2 *Reduction in bending resistance under high shear force*

If V is less than 0.6Vd, reduction in the plastic bending resistance of the section need not be considered. When V > 0.6Vd, the bending resistance is reduced as the contribution of the web to bending gets diminished. The reduced bending resistances of the following sections are given below:

a) *Plastic or compact section*

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq (1.2\, z_e f_y)/\gamma_{m0}$$

where

β = $(2V/V_d - 1)^2$

M_d = plastic design moment of the whole section disregarding high shear force effect considering web buckling effects

V = factored applied shear force.

V_d = design shear resistance as governed by web yielding or web buckling

M_{fd} = Plastic design resistance of the area of the cross section excluding the shear area, considering partial safety factor γ_{m0}

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b) *Semi-compact Section*

$$M_{dv} = Z_e f_y / \gamma_{mo}$$

where

Z_e = elastic section modulus of the whole section.

8.3.7 *Hybrid Sections*

Use of hybrid steel sections consisting of different grade steel elements is permitted. The plastic and compact section design strength is obtained by using the appropriate value of yield stress in the different elements of the hybrid section. The design strength of the semi-compact section is evaluated with necessary adjustment (reduction) in stresses of the flange element in the cross-section with higher yield stress by the reduction factor, R_h. The R_h may be determined using the procedure as presented in **B-3** of Annex B.

8.3.8 *Partially Encased Sections*

This section is applicable for partially encased composite plastic or compact beams, provided the web depth to thickness ratio, d/t_w, is less than or equal to 117ε.

The full shear connection between the structural steel section and concrete encasing the web shall be ensured. The steel reinforcement in the compression zone and concrete in the tension zone may be disregarded.

The partial shear connection may be used in the partially encased composite section, provided the requirements of **8.3.5.2** are satisfied.

9 DESIGN FOR SERVICEABILITY LIMIT STATES

9.1 General

9.1.1 Serviceability limit states are related to the criteria governing normal use. Serviceability limit state is the limit state beyond which the serviceability criteria specified below are no longer met:

a) Stress and deflection limit;

b) Vibration limit;

c) Durability consideration; and

d) Fire resistance

9.1.2 Linear elastic analysis is used for finding out design moments and stresses under various load combinations and load factors, as mentioned in Table 7, for serviceability limit states. Concrete is assumed as unreinforced and un-cracked for the analysis.

9.1.3 *Method of Construction*

The stress and strain at serviceability limit state depend on whether the steel beam is propped or un-propped during construction, as given below:

a) *Un-propped construction* — In un-propped construction, the steel beam has to carry the construction load, including shuttering, wet concrete and its own weight until the concrete hardens;

b) *Propped construction* — In propped construction, both the dead and imposed load are resisted by the composite section. When props are used, they should be kept in place until the *in-situ* concrete has attained a strength equal to approximately twice the stress to which the concrete may be subjected to upon removal of props; and

c) *Non-composite construction* — In non-composite construction, the concrete slab is designed according to [C-6B(2)] and the steel deck sheet act just as a permanent shuttering.

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The difference in the method (a) and (b) of construction does not, however, affect the ultimate limit load, wherein the total load, including the transient loads, shall be resisted by the composite section.

9.2 Negative Moments

Negative moments over intermediate supports may be adjusted as mentioned in **8.1.2.1**.

9.3 Stresses and Deflections

For calculating stresses at service load and deflection, the value of the modular ratio, *m* shall be taken as,

$m = \frac{E_s}{E_{cm}} \geq 7.5$ For short-term effect or loading.

$m = \frac{E_s}{K_c E_{cm}} \geq 15.0$ For permanent or long-term loads.

where

K_c = Creep factor, 0.5

E_s = Modulus of elasticity for steel = 2.0×10^5 N/mm^2;

E_{cm} = Modulus of elasticity of cast *in-situ* concrete as per Annex F;

f_{ck} = characteristic cube compressive strength of concrete in N/mm^2; and

E_{ci} = Modulus of elasticity of cast *in-situ* concrete at *i* days ($i < 28$ days).

The equivalent area of concrete slab at any stage shall be determined by dividing the effective width of the concrete slab by the modular ratio as given below.

$$m = \frac{E_s}{E_{ci}}$$

Final stresses and deflection are to be worked out separately at each stage of load history with relevant modular ratios and section modulus as discussed above and then added together.

9.3.1 *Limiting Stresses for Serviceability*

Limiting stresses for different stages of construction are as indicated below:

a) Concrete — The limiting compressive stress in concrete should not exceed one-third of the characteristic strength of concrete;

b) Reinforcement Steel — The limiting tensile stress in steel reinforcement should not exceed f_{yk}/γ_k; and

c) Structural Steel — The limiting stress in steel beam considering the different stages of construction should not exceed f_y/γ_{mo}.

Where bearing stress is combined with tensile or compressive stress, bending and shear stresses under the most unfavourable conditions of loading, the equivalent stress obtained from the following equation shall not exceed $0.9f_y$:

$$f_{ec} = \sqrt{f_{bc}^2 + f_p^2 + f_{bc}.f_p + 3\tau_b^2}$$

and

$$f_{et} = \sqrt{f_{bt}^2 + f_p^2 + f_{bt}.f_p + 3\tau_b^2}$$

where

f_{ec} and f_{et} = equivalent compressive and tensile stress in steel section;

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f_{bc} and f_{bt} = actual compressive and tensile stress in steel section;

f_p = actual bearing stress in steel section; and

τ_b = actual shear stress in steel section.

The value of bending stresses f_{bc} about each axis, to be used in the above formula, shall be individually lesser than the values of the maximum allowable stresses in bending about the corresponding axis.

9.3.2 *Limiting Deflection and Camber*

9.3.2.1 *Deflection limit*

The deflection under serviceability loads of a building or a building component should not impair the strength of the structure or components or cause damage to finishing. Deflections are to be checked for the most adverse but realistic combination of service loads and their arrangements by elastic analysis using a load factor given Table 6. The deflection of a member shall be calculated without considering the impact factor or dynamic effect of the loads on deflection. As per good practice [C-6B(3)] gives recommended limits of deflections for certain structural members and systems. Circumstances may arise where greater or lesser values would be more appropriate depending upon the nature of the material in element to be supported (vulnerable to cracking or not) and intended use of the structure, as required by the client.

9.3.2.2 *Provision of camber*

Where the deflection due to the combination of dead load and imposed load is likely to be excessive, consideration should be given to pre-camber the steel beams. The values of desired camber shall be specified in the design drawing. Generally, for spans greater than 25 m, camber approximately equal to the deflection due to dead loads plus half the imposed load may be used.

9.4 Vibration

Suitable provisions in the design shall be made for the dynamic effects of imposed loads, impact loads and vibration due to machinery operating loads. In severe cases, the possibility of resonance, fatigue or unacceptable vibrations shall be investigated. Unusually flexible structures (generally the height to the effective width of lateral load resistance system exceeding 5 : 1) shall be investigated for lateral vibration under dynamic wind loads as per(*see* Part C 'Structural Design/Section 1 Loads and Load effects' of this NBCS). Structures subjected to a large number of cycles of loading shall be designed against fatigue failure in accordance with as per good practice [C-6B(3)]. The floor vibration effect shall be considered using specialist literature.

9.5 Durability

9.5.1 Several factors that affect the durability of the buildings under conditions relevant to their intended life are listed below:

a) The environment;

b) The degree of exposure;

c) The shape of the member and the structural detail;

d) The protective measure; and

e) Ease of maintenance.

The durability of the structural steel component of a composite section shall be ensured by recommendations as given in as per good practice [C-6B(3)]. Specialist literature may be referred to for more detailed additional information in design for durability. For concrete, the durability shall be ensured by following the recommendations as given in as per good practice [C-6B(2)].

9.5.2 *Profiled Steel Sheeting for Composite Slabs*

The durability criteria for profiled steel sheets of composite deck slab shall be met by the following procedures:

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a) The exposed surfaces of the steel sheeting shall be adequately protected to resist particular atmospheric conditions; and

b) A zinc coating of a total mass 275 g/m^2 (including both sides) is sufficient for internal floors in a non-aggressive environment, but the specification may be varied depending on service conditions.

9.6 Fire Resistance

Fire resistance of a steel component of a composite member is a function of its mass, its geometry, and the actions to which it is subjected, its structural support conditions, fire protection measures adopted and the fire to which it is exposed. Design provisions to the resistance of fire for concrete shall be as per guidelines given in as per good practice [C-6B(2)]. For the design of structural steel components for fire resistance, as per good practice [C-6B(3)] shall be referred. Specialist literature may be referred to for more detailed information on the design for fire resistance of steel/composite structures. The aspect of fire resistance is given in **15**.

9.7 Control of Cracking in Concrete and Crack Width Calculation

Minimum reinforcements in terms of diameter and spacing required for crack control at the top of concrete as per good practice [C-6B(2)] are to be provided in composite beams at the zone of the negative moment to prevent cracks adversely affecting the appearance and durability of the structure. Crack width calculation as well as limiting crack width as given in as per good practice [C-6B(2)] may be followed, subject to the discretion of engineers. The crack width in concrete shall be restricted to values as indicated in as per good practice [C-6B(2)].

10 DESIGN FOR FATIGUE LIMIT

This section applies to the design of structures and structural elements subject to loading which could lead to fatigue failure. The following effects are not considered in the section:

a) Corrosion fatigue;

b) Low cycle (high stress) fatigue;

c) Thermal fatigue;

d) Stress corrosion cracking;

e) Effects of high temperature (> 150 °C); and

f) Effects of low temperature (< below transition temperature).

The fatigue design of various components of composite structures like members, welded joints, bolts, shear lugs etc. shall be carried out as per the specifications laid down in as per good practice [C-6B(3)]. Fatigue provisions in the design of shear connectors are discussed in **11.3**.

11 SHEAR CONNECTORS

The shear connectors shall fulfill the dual purpose of transferring shear force between concrete and structural steel as well as anchoring the two components relative to each other with minimum slip to ensure full or partial composite action as per the design requirement. The dimensional details of shear shall be as given in Fig. 8.

11.1 Longitudinal Shear in Beams and Slabs

Longitudinal shear load on shear connectors in a composite section, irrespective of boundary conditions of the members, is to be calculated for service and fatigue limit states on the basis of elastic theory. Appropriate sectional properties based on effective widths and modular ratios as per the load history and development of composite action shall be considered for the design of the section for resistance against longitudinal shear between steel and concrete.

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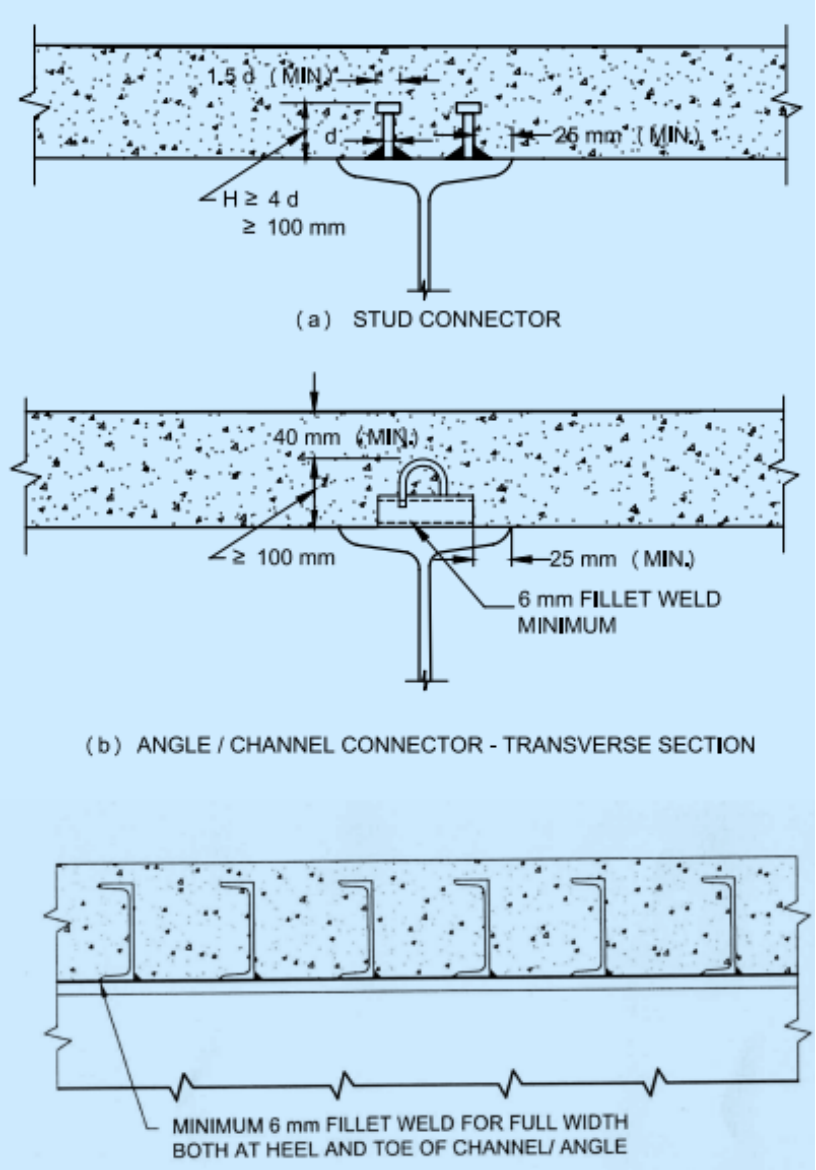

FIG. 8 DETAILS OF SHEAR CONNECTORS

NOTES

1 The diameter of the stud connector welded to the flange plate shall not exceed two and half times the flange plate thickness.

2 The height of the stud connectors shall not be less than four times their diameter nor 100 mm.

3 The diameter of the head of the stud shall not be less than one and a half times the diameter of the stud and the depth shall not be less than 0.4 d.

4 The size of the fillet weld joining other types of connectors to the flange plate shall not exceed half the thickness of the flange plate.

5 Channel and angle connectors shall have at least 6 mm fillet welds placed along the heel and toe of the channels/angles. The clear distance between the edge of the flange and the edge of the shear connectors shall not be less than 25 mm.

6 The overall height of a connector, including any hoop, which is an integral part of the connector, shall be at least 100 mm with a clear cover of 25 mm.

11.2 Design Strength of Shear Connectors

Shear connectors shall be checked for adequacy against failure in both ultimate limit states and fatigue limit states. The strength of shear connectors against failure under ultimate limit states and fatigue limit states shall be considered as per **11.2.1**, **11.2.2** and **11.3**, respectively.

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11.2.1 *Design Strength of Shear Connectors*

Design static strengths of flexible shear connectors, mainly stud connectors and channel connectors can be determined by the following equations:

a) *Stud connectors*

The design resistance, Q_d of stud shears connectors shall be as given below:

$$Q_d = \frac{0.8 f_{us}.\pi.d^2/4}{\gamma_{mv}} \leq \frac{0.29\ \alpha\ d^2\sqrt{0.8\ f_{ck}.E_{cm}}}{\gamma_{mv}}$$

where

$\alpha = 0.2\left\{\frac{h_s}{d}+1\right\}$ for $3 \leq \frac{h_s}{d} \leq 4$ and $\alpha = 1.0$ for $\frac{h_s}{d} > 4$;

Q_d = design strength of stud in, Newton (N);

γ_{mv} = partial safety factor for stud connector = 1.25;

d = diameter of the shank of the stud, in mm (16 mm $\leq d \leq$ 25 mm);

f_{us} = ultimate tensile strength of the stud material $\leq$ 500 N/mm^2;

f_{ck} = Characteristic compressive strength of concrete of density not less than 1750 kg/m^3;

h_s = nominal height of stud in mm; and

E_{cm} = Secant modulus of elasticity of concrete (*see* Annex F).

b) *Channel Connectors*

Assuming that the web of the channel is placed vertical and the shear applied is nominally perpendicular to the web, the design resistance of a channel connector shall be determined as given below:

$$Q_d = \frac{\left[20b(h)^{3/4}\right](0.8f_{ck})^{1/3}}{\gamma_{mv}}$$

where

Q_d = Design strength of channel in Newton (N);

h = height of the channel in mm;

b = length of the channel in mm;

f_{ck} = Characteristic compressive strength of concrete in MPa; and

γ_{mv} = factor for channel shear connector = 1.25.

While using the channel shear connectors, the following recommendations need to be followed:

a) The height h of the channel should not exceed 20 times the channel web thickness or 150 mm, whichever is less;

b) The width b of the channel should not exceed 300 mm;

c) The underside of the top flange of the channel should not be less than 30 mm clear above the bottom reinforcement; and

d) The size of the fillet weld connecting the channel to the flange plate should not exceed half the flange plate thickness.

11.2.2 The design strengths of some standard shear connectors are given in Table 9. The following points have to be considered while using Table 9:

a) $f$ck is the specified characteristic cube strength at 28 days;

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b) Connector strengths for concrete of intermediate grade may be obtained by linear interpolation;

c) For channels of lengths different from those quoted above, the capacities are proportional to the lengths greater than 150 mm;

d) For rolled angle and tee shear connectors, the values given for channel connectors are applicable, provided the height is at least equal to that of the channel;

e) For stud connectors of overall height greater than 100 mm, the design static strength should be taken as the values given in Table 9 for 100 mm high connectors;

f) The above provisions of stud connectors are not applicable to composite slab using a profiled deck. The strength of the shear connector in such cases is given in **11.2.3**;

g) The number of shear connectors given by the above Table 9 shall be distributed in the zone between the maximum and the zero moment sections. The number of connectors has to meet both ultimate strength consideration as well as fatigue consideration; and

h) In order to avoid undesirable slip, the maximum interface shear per unit length due to superimposed dead load and imposed load under service conditions at any point in the beam should satisfy **11.3.1**.

Table 9 Ultimate Static Design Strengths of Shear Connectors (*Q*n for Different Concrete Strengths)

(*Clause* 11.2.2 *and* 11.3.1)

Sl No.	Type of Connector	Material	Size		Ultimate Static Strength in kN per connector			
			Nominal diameter mm	Nominal height mm	For concrete strengths, f_{ck} MPa			
					25	30	40	50
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	Stud connectors	Material with a characteristic yield strength of 385 MPa, minimum elongation of 18 percent and a characteristic tensile strength of 495 MPa	25	100	112	125	149	156
			22	100	87	97	115	120
			20	100	72	80	90	100
			20	75	63	71	84	97
			19	100	65	72	86	90
			19	75	64	72	85	90
			16	75	46	51	61	64
			12	65	26	29	34	36
ii)	Channels 150 mm long (min)	As per [C-6B(4)] (E250 A/BR)	ISMC 125		244	259	285	307
			ISMC 100		206	219	241	260
			ISMC 75		166	176	194	209

11.2.3 *Design Resistance of Studs Used with Profiled Steel Sheeting*

11.2.3.1 *Sheeting with ribs parallel to the supporting beams*

The studs are located within a region of concrete that has the shape of a haunch (Fig. 9). When the sheeting is continuous across the beam, the width of the haunch b_0 is equal to the width of the trough, as given in Fig. 10. Where the sheeting is not continuous, b_0 is as given in Fig. 9. The depth of the haunch should be taken as d_p, the overall depth of the sheeting excluding embossments.

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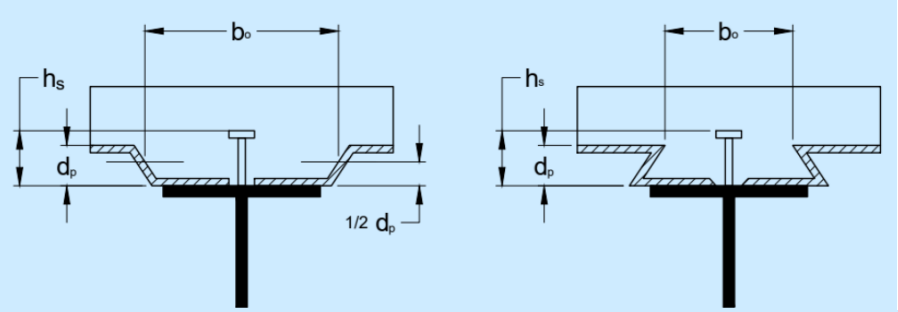

FIG. 9 BEAM WITH PROFILED DECKING SHEET RIBS PARALLEL TO THE BEAM

The design resistance of the shear connector should be taken as the resistance in a solid slab, multiplied by the reduction factor k_p given by the following expression:

$$k_p = 0.6\frac{b_0}{d_p}\left(\frac{h_s}{d_p} - 1\right) \leq 1.0$$

Where h_s is the overall height of the stud, but not greater than d_p+75 mm.

Where the sheeting is not continuous across the beam, as shown in Fig. 9, the side of the haunch and its reinforcement shoud be taken as the depth of the profile d_p excluding the embossments.

11.2.3.2 *Sheeting with ribs transverse to the supporting beams*

Provided that the conditions given in **11.2.2** are satisfied, the design shear resistance should be taken as the resistance in a solid slab, calculated as given by **11.2.1** (except that f_{us} should not be taken as greater than 450 N/mm^2) multiplied by the reduction factor k_t given by:

$$k_t = \frac{0.7}{\sqrt{n_r}}\frac{b_0}{d_p}\left(\frac{h_s}{d_p} - 1\right) \leq 1.0$$

where

n_r is the number of stud connectors in one rib at the beam intersection, not to exceed two in calculation of the reduction factor k_t and of the longitudinal shear resistance of the connection.

The factor k_t should not be taken greater than the appropriate value k_{tmax} given in Table 10. The values for k_t are applicable provided that: the studs are placed in ribs with a height d_p not greater than 85 mm and a width b_0 not less than d_p and for through deck welding, the diameter of the studs is not greater than 20 mm, or for holes provided in the sheeting, the diameter of the studs is not greater than 22 mm.

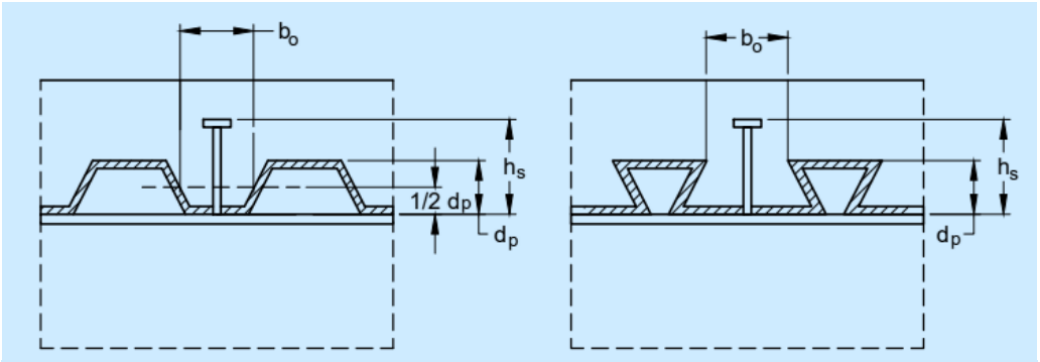

FIG.10 BEAM WITH PROFILED DECKING SHEET RIBS TRANSVERSE TO THE BEAM

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Table 10 Upper Limits $K_{t,\ max}$ for the Reduction Factor K_T

(*Clause* 11.2.3.2)

Sl No.	Number of Stud Connectors per Rib	Thickness t of Sheet (mm)	Studs 20 mm in Diameter and Welded Through Profiled Steel Sheeting	Profiled Sheeting with Holes and Studs 19 mm or 22 mm in Diameter
(1)	(2)	(3)	(4)	(5)
i)	n_r=1	≤1.0	0.85	0.75
		>1.0	1.0	0.75
ii)	n_r =2	≤1.0	0.70	0.60
		>1.0	0.8	0.60

11.3 Fatigue Strength of Shear Connectors

The fatigue shear stress range (fatigue Strength) of the shear connector shall be obtained from [C-6B(3)], corresponding to the design load life cycle, N_{SC}.

The strength shall be determined as given below:

$$\tau_f = \tau_{fn} \sqrt[5]{5 \times 10^6 / N_{SC}}$$

τ_{fn} = design normal and shear fatigue stress range respectively of the detail for 5 × 10^6 cycles as given in [C-6B(3)].

τ_{fn} = 67 N/mm^2 for stud connector as per good practice [C-6B(3)].

τ_{fn} = 59 N/mm^2 for channel connector as per good practice [C-6B(3)] [provided that the thickness of the top flange of steel beam is greater than or equal to 12 mm and the edge distance from the end of weld to the edge of the top flange is 10 mm].

The design fatigue strengths of some standard shear connectors are presented in Table 11.

Table 11 Fatigue Strengths *Q*r (in kN)

(*Clause* 11.3)

Sl No.	Type of Connectors (Headed Stud – Diameter)	Connector Material	N = Nos. of Cycles in Millions				
			0.1	0.5	2	10	100
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Headed studs φ 25	f_{ys} = 385 MPa f_{us} = 495 MPa Elongation = 18 percent	72	52	40	29	18
ii)	Headed studs φ 22		56	40	31	22	14
iii)	Headed studs φ 20		46	33	25	18	12
iv)	Headed studs φ 16		29	21	16	12	7
v)	Channel 150 long for a nominal weld of 8 mm	[C-6B(4)]	108	79	60	43	27

NOTE — For intermediate stress cycles the values may be interpolated from log scales (that is, the above equation). Other connectors, if used, should have their capacities established through tests.

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11.3.1 *Spacing of Shear Connectors*

The design shear action per unit length of the steel concrete interface, V_L, is given by:

$$V_L = \sum \left[\frac{V A_{ec} y}{I}\right]_{DL,IL}$$

where

V_L = Longitudinal shear per unit length;

V = The factored vertical shear forces due to dead load and imposed load (including impact if any) separately at each state of load history;

A_{ec} = The transformed compressive area of concrete above the plastic neutral axis of the composite section with the appropriate modular ratio depending on the nature of load (short term that is, imposed load, or long term, that is, dead load);

y = C.G. distance of transformed concrete area from the plastic neutral axis;

I = Moment of Inertia of the composite section using an appropriate modular ratio; and

DL, IL = Different load history conditions, that is, sustained load or composite action dead load, transient load, or composite action imposed load. These loads are to be considered with appropriate load factors at this stage.

Maximum spacing of shear connectors, S_{L1} is given as

$$S_{L1} = \frac{\sum Q_n}{V_L}$$

Q_n is the ultimate static strength of one shear connector (Table 9) and the summation is over the number of shear studs in one section.

The maximum longitudinal force at the interface due to bending moment shall also be calculated over the shear span, L, equal to the distance from zero moment to maximum moment section and is given by the following equations:

$$H_1 = A_{sl} f_y \times 10^{-3}/\gamma_{m0}$$

$$H_2 = 0.36 f_{ck} A_{ec} \times 10^{-3}$$

where

H_1, H_2 = Longitudinal interface shear force due to bending (kN);

A_{sl} = Area of tensile steel (mm^2) in the longitudinal direction;

A_{ec} = Effective area of concrete:

$b_{eff}.x_u$ (for neutral axis within the slab); and

$b_{eff}. d_s$ (for the neutral axis in the steel section)

Sufficient connectors should be provided to resist the longitudinal force H, the maximum compressive force action in the composite beam slab interface, which is the smaller of H_1 and H_2.

The maximum spacing of shear connectors, S_{L2}, is given as:

$$S_{L2} = \frac{\sum Q_n}{H}.L$$

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11.3.2 *Fatigue Strength of Shear Connectors*

$$V_r = \Sigma \left[\frac{V_R . A_{ec} . y}{I}\right]_{LL}$$

where

V_R = The shear range in which is the difference between the maximum and minimum vertical shear envelop due to imposed load and impact; and

H = is imposed load with impact.

V, Aec, y and I are as explained above

Spacing of shear connectors from fatigue consideration is given as:

$$S_R = \frac{\sum Q_r}{V_r}$$

Q_r is the design fatigue strength of one shear connector, which is to be taken from as per good practice [C-6B(3)] Table 10.

For full shear connection, the lowest spacing of S_{L1}, S_{L2}, and S_R is to be provided as the actual spacing of the shear connectors.

11.4 Partial Shear Connection

11.4.1 Partial shear connection may be used either for attaining economy without losing much in the moment capacity of the composite section or where the number of shear connectors required for full shear cannot be provided without compromising minimum spacing provisions.

Partial shear connections may be used in plastic and compact sections. The number of connectors n_p shall then be determined by a partial shear connection theory, taking into account the deformation capacity of the shear connector.

$$S_c \text{ degree of shear connection} = \frac{n_p}{n_f}$$

n_p = Number of shear connectors provided for partial shear connection;

n_f = Number of shear connectors required for full shear connection;

M_R = Required reduced bending resistance of the section;

M_d = The fully plastic moment of resistance of the composite section; and

M_{ps} = Plastic moment of resistance of steel section alone.

$$n_p = \frac{M_R - M_{ps}}{M_d - M_{ps}} n_f$$

11.4.2 *Limitation on the Use of Partial Shear Connection in Beams for Buildings*

Headed studs with an overall length after welding not less than 4 times its diameter and with shank diameter not less than 16 mm and neither greater than 25 mm may be considered as ductile with the following limits for the degree of shear connection, S_c:

a) For steel sections with equal flanges:

$$L_e \leq 25: S_c \geq 1 - 1.42\varepsilon^2 (0.75 - 0.03L_e); S_c \geq 0.4$$

$$L_e > 25: S_c = 1.0$$

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b) For steel sections having a bottom flange with an area of three times the area of top flange:

$$L_e \leq 20: S_c \geq 1 - 1.42\epsilon^2 (0.30 - 0.015L_e);\ S_c \geq 0.4$$

$$L_e > 20: S_c = 1.0$$

where

$\epsilon = \sqrt{250/f_y}$, L_e is the distance between points of zero bending moment in the sagging bending range in metres. For typical continuous beams, L_e may be assumed as shown in Fig. 6 and Fig. 7.

c) For steel sections having a bottom flange with an area exceeding the area of the top flange but less than three times that area, the limit for S_c may be determined from expressions in (a) and (b) above by linear interpolation.

11.5 Precautions against Separation of Steel Beam from Concrete

a) Where a concrete haunch is used, between the steel flange and the soffit of the slab, top of stud and top flange of channel shear connectors shall extend up to at least 40 mm above the transverse reinforcements in the haunches, provided the reinforcements are sufficient to transfer longitudinal shear; and

b) Where shear connectors are placed adjacent to the longitudinal edge of the slab, transverse reinforcement provided in accordance with **11.7** shall be fully anchored in the concrete between the edge of the slab and the adjacent row of connectors.

11.6 Detail of Haunches in Reinforced Concrete

11.6.1 Figure 11 indicates the dimension of haunches as applicable for slabs with haunches resting on steel beam. The edge of haunches shall be located outside a line drawn at 45 degrees from the outside edge of the base of the connector, as shown in Fig. 11.

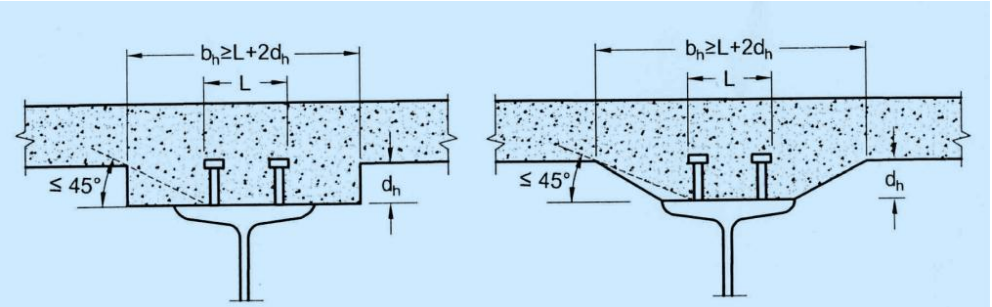

FIG. 11 DIMENSION OF HAUNCHES

11.6.2 *Clear Cover to Shear Connectors*

The clear depth of concrete cover over the top of the shear connectors shall not be less than 25 mm. The horizontal clear concrete cover to any shear connector shall not be less than 50 mm as shown in Fig.12.

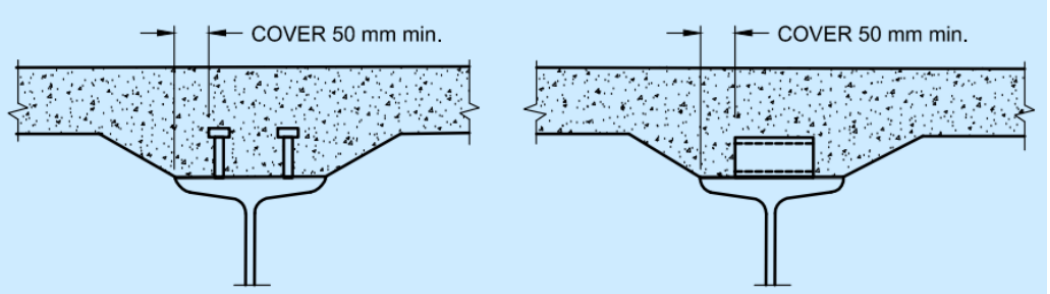

FIG. 12 CLEAR COVER TO SHEAR CONNECTORS

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11.6.3 *Spacing of Shear Connectors*

Where the compression flange of steel beam, in spite of being semi-compact is assumed to be compact or plastic based on the restraint provided by shear connectors, the centre-to-centre spacing of the shear connectors in the direction of the compression should satisfy the following:

a) Where the slab is in contact over the full length (for example, solid slab)

$$S_L < 21 \cdot t_f \cdot \varepsilon$$

b) Where the slab is not in contact over the full length (for example, slab with ribs transverse to the beam):

$$S_L < 14 \cdot t_f \cdot \varepsilon$$

where

$\varepsilon = \sqrt{250/f_y}$

t_f = thickness of the flange;

f_y = nominal yield strength of the flange in N/mm^2; and

S_L = maximum spacing of the shear connector.

In addition, the clear distance from the edge of the compression flange to the nearest line of shear connectors should not be greater than $9t_f\ \varepsilon$ or 50 mm, whichever is less.

In all cases, a shear connector shall be provided throughout the length of the beam and may be uniformly spaced between critical cross sections. The maximum longitudinal spacing of the shear connectors shall be limited to the lesser of 600 mm or 3 times the total slab thickness or four times the height of the connector (including any hoop which is an integral part of the connector), whichever is least.

Minimum spacing should be such as to allow proper concrete flow and compaction around the connectors. In stud connectors, the minimum spacing should not be less than 75 mm.

11.7 Transverse Shear Check (Requirement of Bottom Steel in Concrete Slab)

Planes, which are critical for longitudinal shear failure in the process of transfer of longitudinal shear from the beam to the slab are of four main types as shown in Fig. 13. The shear force Q in kN/m of beam is given by:

$$Q = \frac{N_r\, Q_u}{S_L}$$

where

N_r = Number of shear connection at a section;

Q_u = Design strength one shear connector; and

S_L = Longitudinal spacing of connector in meter.

The shear force transferred per metre length from steel beam to concrete slab above, V_L shall satisfy both the following conditions:

a) $V_L \leq 0.623\, L\sqrt{f_{ck}}$; or

b) $V_L \leq 0.232\, L\sqrt{f_{ck}} + 0.1.\, A_{st}.\, f_{yk}.\, n$

where

V_L = Longitudinal shear force per unit length calculated for the ultimate limit state;

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f_{ck} = Characteristic strength of concrete, in MPa;

f_{yk} = Yield stress of transverse reinforcement, in MPa;

L = Length (in mm) of possible shear planes; for two rows of shear studs as shown in Fig.15, $L = 2\ h_s + b_0$;

n = Number of times each lower transverse reinforcing bar is intersected by a shear surface (that is, the number of rows of shear the connector at one section of the beam). Generally, for T-beam n = 2 and for L-beam n = 1; and

A_{st} = Sectional areas (in cm^2) of transverse reinforcements per meter run of the beam. The amount of transverse steel (cm^2/m) in the bottom of the slab shall not be less than $\frac{2.5Q}{f_{yk}}$.

11.8 General Arrangements of Transverse Reinforcements

11.8.1 If the concrete by itself is insufficient to take the longitudinal shear, sufficient transverse reinforcements shall be provided to transfer longitudinal shear force from the beam to the effective width of the slab. The area of transverse reinforcement per unit length of the beam will be the sum total of all the reinforcement [A_t, A_b or A_h, as shown in Fig. 14(a), Fig. 14(b) and Fig. 14 (c)], which are intersected by the shear plane and are fully anchored on both the sides of the shear plane considered.

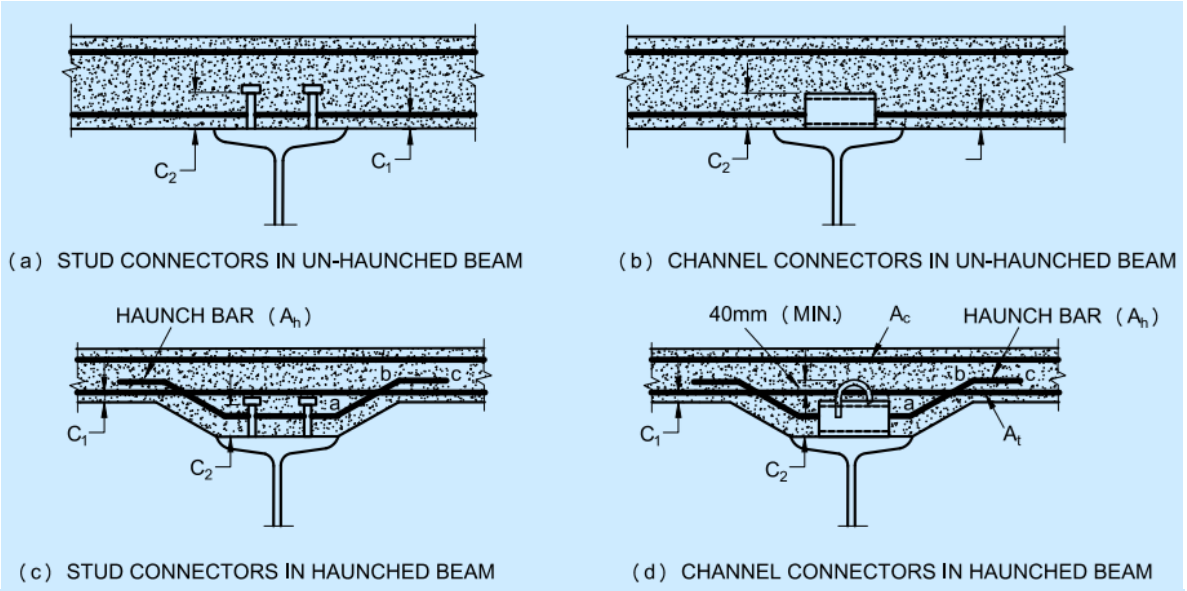

FIG. 13 ARRANGEMENT OF TRANSVERSE REINFORCEMENTS

11.8.2 *Total Transverse Reinforcements*

The total transverse reinforcements Ast· per unit length of the beam in case of shear plane 1 – 1, which crosses the whole thickness of the slab, will be the sum of ($A_t + A_b$) [Fig. 14(a)]. Area of reinforcements A_t and A_b include those provided for flexure. The total transverse reinforcements across plane 2-2 [Fig. 14(a)] is $A_{st} = 2\ A_b$ and that across plane 3-3 [Fig. 14(b)] is Ast = $2Ah$ as these planes do not cross the full thickness of the slab. In the case of shear plane 4 – 4 [Fig. 14(c)], the total transverse reinforcement is Ast = $2(A_h + A_b)$. The transverse reinforcements shall be placed at locations as shown in Fig. 14. The haunch bars shall be extended beyond the junction of bottom bars by a length equal to the anchorage length.

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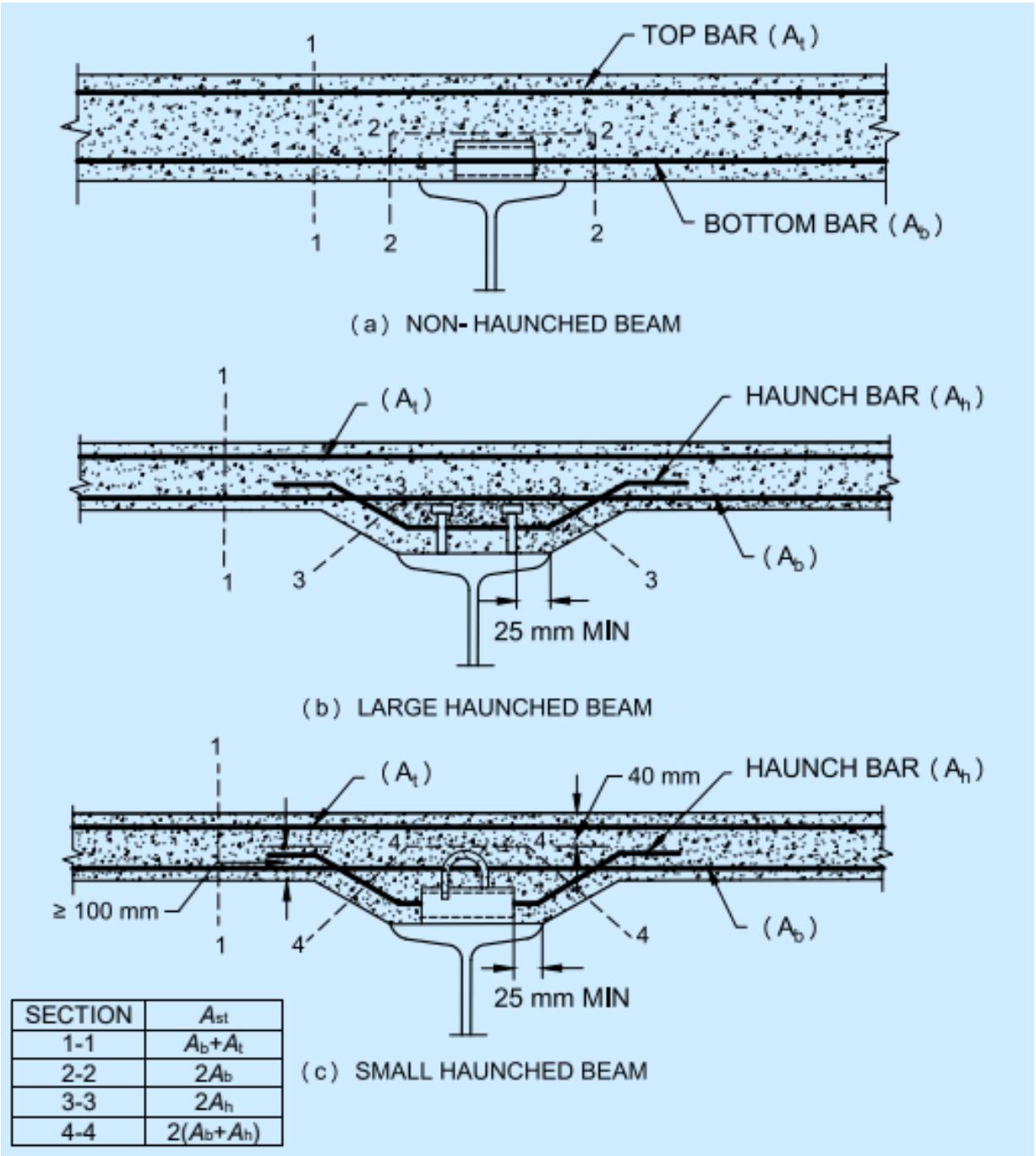

SECTION	A_{st}
1-1	A_b+A_t
2-2	$2A_b$
3-3	$2A_h$
4-4	$2(A_b+A_h)$

FIG. 14 TRANSVERSE REINFORCEMENTS ALONG SHEAR PLANES

12 COMPOSITE SLABS WITH PROFILED STEEL SHEETING

12.1 General

The provisions in this section deal with composite floor slabs spanning only in the direction of ribs of the decking. The provisions are applicable in buildings, wherein the slabs are subjected to predominantly static imposed loads. The application of the provision is limited to narrowly spaced web, which is defined by the ratio of the width of the sheet rib to the rib spacing $b_r/b_p \leq 0.6$ (*see* Fig. 15).

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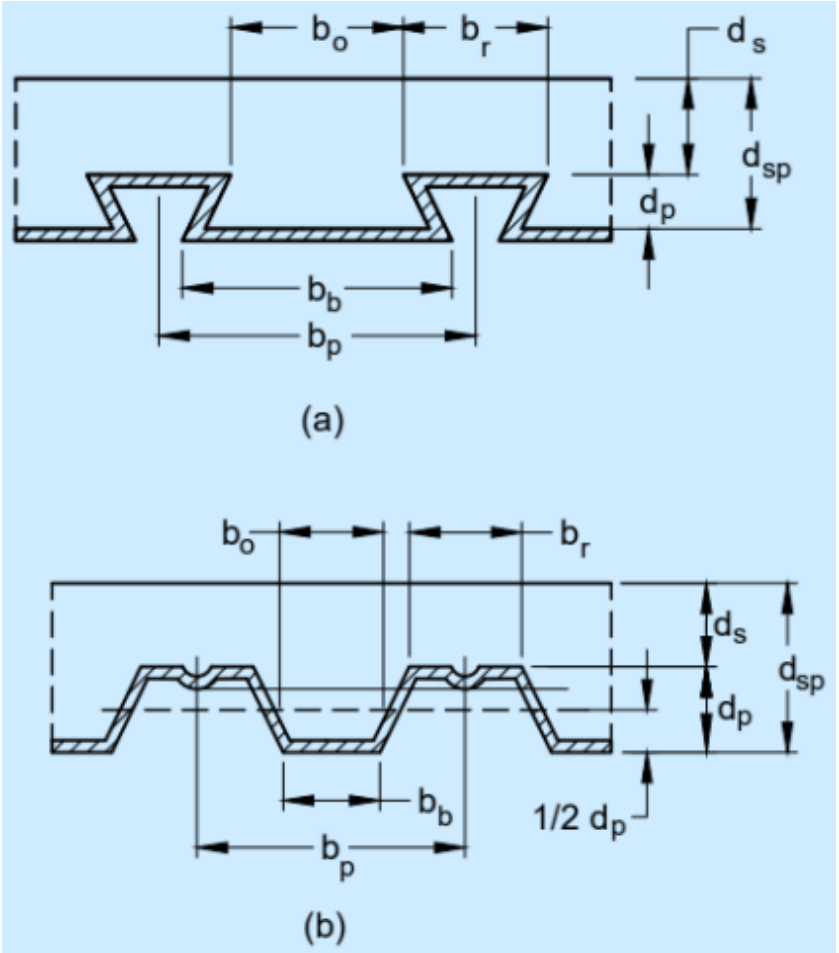

FIG. 15 PROFILE DIMENSION FOR COMPOSITE SLABS.

12.2 Prequalifications

12.2.1 *Prequalification of Slab dimension*

The prequalification of composite and non-composite slabs are presented below:

a) Where the slab acts composite with a steel beam or used as a diaphragm:

   1) the overall depth of the slab should not be less than 90 mm; and

   2) the thickness of concrete above the main top flat surface of the sheeting ribs should not be less than 50 mm.

b) Where the slab does not act composite with a steel beam or has no other stabilizing function:

   1) the overall depth of the slab should not be less than 80 mm; and

   2) the thickness of concrete above the main flat top surface of the sheeting ribs should not be less than 40 mm.

12.2.2 *Bearing Length*

The recommended minimum bearing lengths of steel decking on the support (L_{bs}) and the composite slab, including the cast in place concrete (L_{bs}) shall be as given in (*see* Fig. 16):

a) For composite slab bearing on steel or concrete: L_{bc}=75 mm, L_{bs}=50 mm; and

b) For composite slab bearing on other material: L_{bc}=100 mm, L_{bs}= 70mm.

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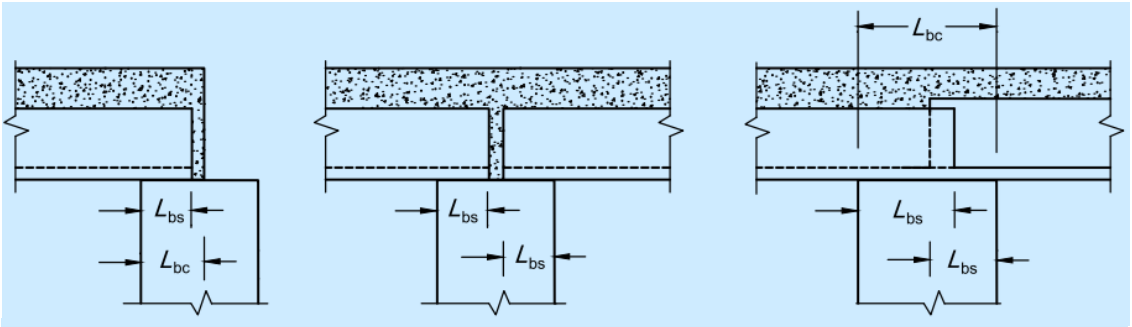

FIG. 16 MINIMUM BEARING LENGTHS

12.2.3 *Reinforcement for Shrinkage and Temperature Stresses*

The effect of temperature stress in composite slabs in buildings is normally neglected unless required in special conditions. The effect of shrinkage is to be considered. The total shrinkage strain for design may be taken as 0.000 3 in the absence of test data.

12.2.4 *Minimum Reinforcement*

The minimum reinforcement in either direction should be according to [C-6B(2)]. The spacing of the reinforcement bar should be in accordance with as per good practice [C-6B(2)].

12.2.5 *Size of Aggregates*

In addition to the above, the largest nominal aggregate size should be according to as per good practice [C-6B(2)].

12.3 Analysis for Internal Forces and Moments

12.3.1 *Analysis of Profile Steel Sheeting as Shuttering*

a) The elastic analysis shall be used where sheeting is considered. The design based on the elastic distribution of bending moment is conservative, as it does not take into account of redistribution of moments that can occur between support and mid span sections;

b) In the analysis of propped profile steel sheeting, the magnitude of bending moments and shear forces could be conservatively calculated by using the coefficients of bending moment and shear force as per good practice [C-6B(2)] ;

c) More accurate analysis and design of profile steel sheeting as a shuttering should be in accordance with as per good practice [C-6B(18)] ;

d) If the deflection at the center of the shuttering due to its own weight and the wet weight of concrete is less than (1/10) of the depth of the composite slab, the effect of ‘ponding’ due to shuttering deflection can be disregarded. Otherwise, that effect of additional load shall be evaluated by taking the additional uniform depth of concrete over the span as 0.7 times the central deflection. Ponding is explained in C-3’

e) The effect of ponding, especially in longer span, can be reduced by propping the shuttering during concreting until the concrete hardens. Plastic redistribution of moments in the propped shuttering is not allowed in evaluating stresses due to self-weight and wet concrete weight.

12.3.2 *Analysis of Composite Slab*

a) The application of linear methods of analysis is suitable for the serviceability limit states as well as for the ultimate limit states. Plastic methods shall be used only in the ultimate limit state;

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b) The following method of analysis may be used for ultimate limit states:

1) Linear analysis without redistribution (serviceability limit state), and with redistribution (Ultimate limit state). If the effects of cracking of concrete are neglected in the analysis for ultimate limit states, the hogging bending moments at internal supports may be optionally reduced by up to 30 percent and corresponding increases are made to the sagging bending moments in the adjacent spans;

2) Rigid-plastic global analysis may be used provided it is shown that sections where plastic rotations are required have sufficient rotation capacity. Plastic analysis without any direct check on rotation capacity may be used for the ultimate limit state if reinforcing steel satisfying the ductility requirement in **6.2.3** is used and the span length is not greater than 3.0 m. (Ultimate limit state); and

3) Elastic-plastic analysis accounting for the non-linear material behavior. (Ultimate limit states).

c) A continuous slab may be designed as a series of simply supported spans. Nominal reinforcement should be provided in accordance with as per good practice [C-6B(2)], over the intermediate supports to manage crack size; and

d) Concentrated loads may be assumed to be distributed over an effective width of the slab, assuming the load is distributed at an angle of 45° to the horizontal up to the top surface of the decking.

12.4 Design of Composite Slabs

Proprietary data backed by analysis and tests may be used in the design. Otherwise, the procedure given below may be used to evaluate the strength of composite slabs.

12.4.1 *Effective Span*

a) When a continuous composite slab is designed as a series of simply supported spans, for simplicity, the effective span can be taken as the lesser of the following:

1) Distance between centres of the supports; and

2) The clear span plus the effective depth of the slab.

b) Where the composite slab is designed as continuous, it is permitted to use an equivalent isostatic span for the determination of the resistance. The span length should be taken as:

1) 0.8 L for internal span (effective span L); and

2) 0.9 L for external span (effective span L).

12.4.2 *Design of Profile Steel Sheeting as Shuttering During Construction*

The stress analysis of the profiled steel sheeting should be evaluated using the design equations or tests. For the ultimate limit state, the resistance of the sheet to sagging and hogging bending, together with the effects of combined bending and web crushing, are normally critical:

a) For the serviceability limit state, the limiting value of deflection $\delta_{\max}$ of steel sheeting under its own weight plus the weight of wet concrete may be considered as $L/180$ (where L is the effective span between supports) as per good practice [C-6B(3)];

b) Profiled decking acting alone resists construction, dead, and imposed load until the concrete hardens and gains strength. ‘Ponding' effect, especially in long spans due to excessive deflection of decking, has to be accounted while evaluating design forces and design strength unless sufficient propping is provided during the construction stage until the concrete hardens; and

c) The longer spans will require propping to eliminate substantial deflection. Otherwise, a significant increase in concrete weight due to ponding has to be accounted.

12.4.3 *Design of Composite Slab for the Ultimate States*

The bending moment and shear force distribution in continuous beams for secondary, as well as primary moment

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resistant frames/beams, have to be determined by structural analysis. The design values of load effects shall not exceed the design values of resistance for the relevant ultimate limit states.

12.4.3.1 *Design flexural resistance*

The design flexural resistance calculation shall take the following provisions into consideration:

a) The design bending resistance, M_d of any cross section should be in accordance with Annex C, where the yield stress of the steel sheeting should be taken as f_{yp};

b) The effect of local buckling of compressed parts of the sheeting should be considered by using effective widths not exceeding twice the limiting values of Class-1 steel webs as discussed in as per good practice [C-6B(3)];

c) For the effective area A_p of the steel sheeting, the width of embossments and indentations in the sheet should be neglected unless tests infer that a larger area is effective;

d) The sagging bending resistance of a cross-section should be calculated from the equilibrium of stress distribution as shown in **C-4**;

e) In hogging bending regions, the contribution of steel sheeting shall be taken into account only where the sheet is continuous. The redistribution of moments that can occur between support and mid-span sections shall not be considered. If the contribution of the steel sheeting is neglected, the hogging bending resistance of a cross-section should be calculated as reinforced concrete for the stress distribution, as shown in **C-4.3**; and

f) In the case of partial shear connection, the sagging design bending resistance, M_d, shall be calculated as per **C-5**.

12.5 Shear Resistance of Composite Slab

12.5.1 *Punching Shear*

The punching shear resistance of a composite slab should be calculated as per good practice [C-6B(2)], where the critical perimeter is as shown in Fig. 17.

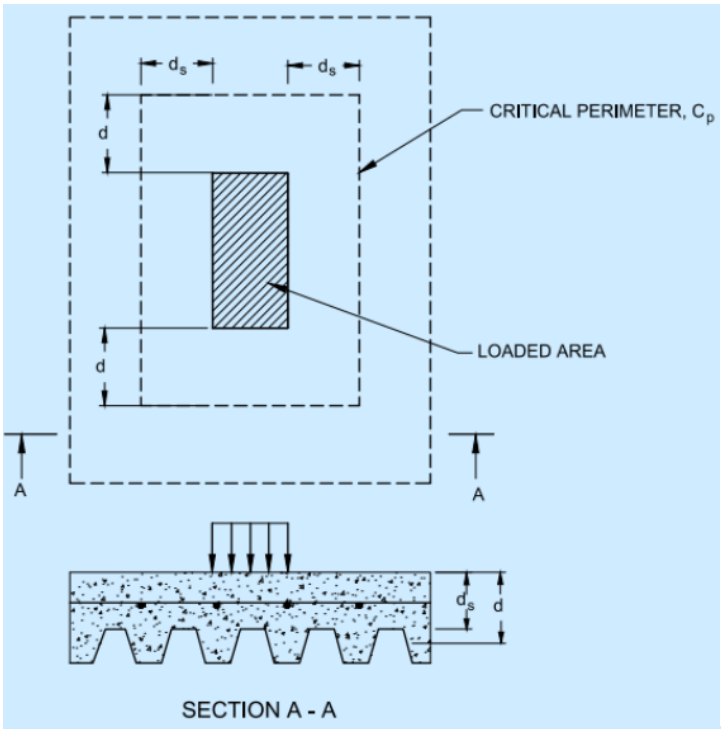

FIG. 17 PUNCHING PERIMETER IN A PROFILED COMPOSITE SLAB

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12.5.2 *Resistance to Shear*

The vertical shear resistance V_{dvd} of composite slab over a width equal to the distance between centres of ribs and the effective depth, which depends on the effective depth of the cross section to the centroid of the tensile reinforcement, should be in accordance with as per good practice [C-6B(2)].

The sheeting may be considered as the tensile reinforcement, provided that it is fully anchored beyond the section considered. For heavily loaded slabs, additional reinforcement may be required at the support when the profiled steel sheeting is discontinuous and has only limited anchorage.

In the case of partial shear connection, the shear capacity may be governed by the interface shear strength between steel decking and concrete, as presented in Annex C.

12.6 Serviceability Limit State

12.6.1 *Design against Cracking*

The crack width is calculated at the top surface in the hogging moment region using standard methods prescribed for reinforced concrete in as per good practice [C-6B(2)]. Crack width should not exceed 0.3 mm. The provision of 0.4 percent steel at the top of the slab will normally avoid cracking problems in propped construction. The provision of 0.2 percent of steel is normally sufficient for the same in un-propped construction.

If the environment is corrosive, it is advisable to design the slab as continuous at supports and take advantage of steel provided for negative bending moment resistance and for minimizing cracking during service loads.

12.6.2 *Deflection Limits*

As per good practice [C-6B(2)] gives a stringent deflection limitation of span/350 or 20 mm whichever is less, which may be unrealistic for un-propped construction. The span to depth ratio in the range of 25 to 35 for the composite condition is recommended for the simply supported slabs and the continuous slabs. The deflection of the composite slabs is influenced by the slip between sheeting and concrete. Testing is the best method to estimate the actual deflection for the conditions adopted.

12.6.3 *Fire Resistance*

The fire resistance is based on (a) thermal insulation criterion concerned with limiting the transmission of heat by conduction and (b) integrity criteria concerned with preventing the spread of flames and hot gases to nearby compartments. These are met by specifying the adequate thickness of insulation below the metal decking. When the metal decking is used as a permanent shuttering, the fire resistance of the slab shall be considered as per as per good practice [C-6B(2)].

13 COMPOSITE COLUMNS

13.1 General

13.1.1 This section applies to various forms of steel-concrete composite columns, including fully or partly encased steel columns and concrete in-filled rectangular or circular steel tubes, provided:

a) The columns or compression members consist of structural steel with grade conforming to the accepted standard [C-6B(4)] and normal weight concrete of strength M20 to M60.

b) This clause applies to isolated columns or compression members in framed structures where the other structural members are either composite or steel members.

c) The steel contribution ratio δ (Table 12) should fulfill the criteria:

$$0.2 \leq \delta \leq 0.9$$

d) Further:

1) The influence of local buckling of the structural steel section on the resistance of the composite section as a whole shall be considered in the design; and

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2) The effects of local buckling may be neglected for a steel section fully encased in accordance with **13.2**, and for other types of cross-section, the maximum width to thickness ratio given in as per good practice [C-6B(3)] shall not be exceeded.

13.1.2 *Types of Composite Columns*

Composite columns can be of two types:

a) Encased where concrete encases the steel section (*see* Fig. 18); and

b) In-filled where concrete fills rectangular or circular steel tube (*see* Fig. 19).

13.2 Details for Composite Action

In composite columns with fully encased steel sections, concrete cover to structural steel sections shall be at least 40 mm or one-sixth of the breadth *b* of the flange, over steel section. The concrete shall be adequately held by steel reinforcements and stirrups all around. The steel section shall be unpainted to ensure bond and friction between steel and concrete, but cleaned to ensure protection against corrosion and spalling of concrete. The cover to steel reinforcement should be in accordance with as per good practice [C-6B(2)].

Shear transfer between steel-concrete interfaces is ensured basically through bond for which calculated shear stress at interface shall be limited in accordance with Table 15, beyond which mechanical shear connectors are to be provided.

13.3 Members Under Axial Compression

Standard composite sections used as columns are as shown at Fig. 18 and Fig. 19.

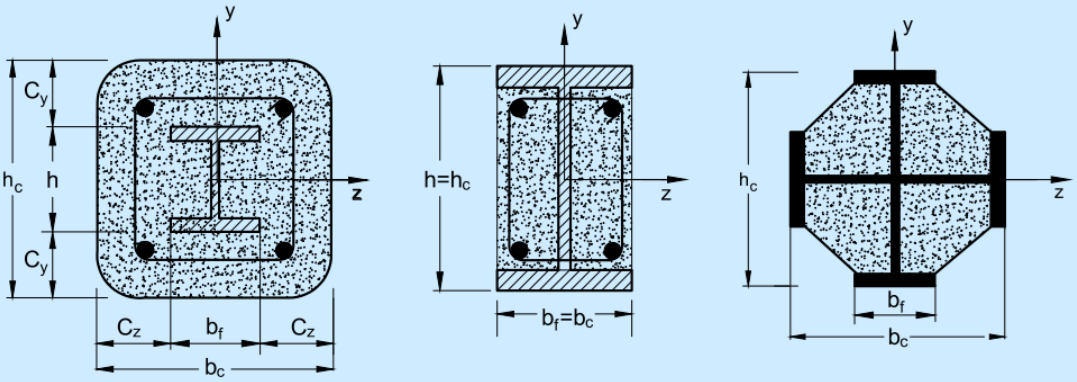

FIG. 18 FULLY AND PARTIALLY CONCRETE ENCASED COLUMNS

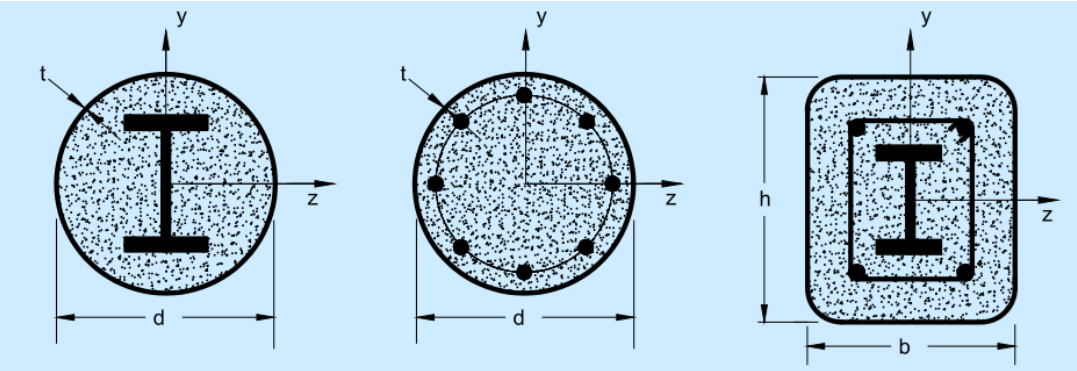

FIG. 19 CONCRETE IN-FILLED COLUMNS

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13.3.1 *General Design Philosophy*

To ensure structural stability of a compression member, second-order effects like residual stresses, geometrical imperfections, local instability, cracking of concrete, creep/shrinkage of concrete, yielding of structural steel and reinforcement etc. shall be considered. While designing the compression members, including design for the above effects, the following shall be taken into account:

a) The above effects shall be considered in any direction in which failure might occur if they affect the structural stability significantly;

b) Internal forces shall be determined by any of the analysis methods discussed in **6**;

c) Full composite action between steel and concrete shall be considered up to failure; and

d) Effects of creep and shrinkage shall be considered if they are likely to reduce the structural stability significantly. For simplification, creep and shrinkage effects may be ignored if the increase in the first-order bending moments due to creep deformations from permanent loads is not greater than 10 percent of first-order bending moments due to total design loads.

13.3.2 *Design Guidelines*

The simplified design provisions given in the following sections is applicable for composite members that are doubly symmetrical and uniform in cross-section throughout the length of the member. This method is not applicable to members consisting of two or more unconnected sections. Further, the composite member should conform to the following conditions.

a) The non-dimensional slenderness λ shall not be greater than 2.

b) For a fully encased steel section (Fig. 18), the maximum thickness of concrete cover that may be used in the strength calculation is limited to:

$$c_z\ (Max) = 0.4\ b_f$$

$$c_y\ (Max) = 0.3\ h$$

c) The longitudinal steel reinforcement that may be used in the calculation should not exceed 6 percent of the concrete area; and

d) The ratio of the cross-section's depth h_c to width b_c, (*see* Fig. 18), should be within limits ($0.2 \leq h_c/b_c \leq 5.0$)

13.4 Local Buckling of Steel Sections

13.4.1 To prevent premature local buckling of structural steel components in partly encased steel section and concrete filled steel sections (Fig. 18 and Fig. 19), the width to thickness ratio of individual elements of the steel sections in compression must satisfy the following limits:

$\frac{d}{t} \leq 88\varepsilon^2$ for concrete-filled circular tubular sections

$\frac{h}{t} \leq 50\varepsilon$ for concrete-filled rectangular tubular sections

$b_f/t_f \leq 43\varepsilon$ for partially encased I sections

where

$\varepsilon = \sqrt{\frac{250}{f_y}}$ and f_y is the yield strength of the steel section, in MPa.

13.4.2 For fully encased steel sections, the above local buckling check is not required. However, the concrete cover to the flange of a fully encased steel section should not be less than 40 mm, nor less than one-sixth of the breadth, b_f, of the flange.

13.4.3 Design of concrete-filled rectangular tubular sections where *h/t* ratios exceed the local buckling limits for semi-compact sections should be verified by tests.

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13.4.4 The cover to reinforcement should be in accordance with as per good practice [C-6B(2)].

13.4.5 The steel section shall not be painted to ensure friction between steel and concrete but cleaned at the abutting surface to ensure protection against corrosion and spalling of concrete.

13.5 Design Compressive Resistance of Short Composite Columns

13.5.1 A column is termed as a 'short column' when one of the following conditions is satisfied.

a) its non-dimensional slenderness ratio $\lambda \leq 0.2$; and

b) If the actual design compressive force on the composite column, P, is less than 0.1 P_{cr}.

where

$$\lambda = \sqrt{\frac{P_n}{P_{cr}}}$$

P_{cr} = the elastic buckling load of the column $= \frac{\pi^2 (EI)_e}{L^2}$;

$(EI)_e$ = effective elastic flexural stiffness of the composite column (13.5.2); and

L = the effective length of the column, which may be conservatively taken as the overall length L for an isolated non-sway composite column.

The design compressive strength P_n of short columns of plastic and compact sections may be evaluated using the design strength of the material with appropriate partial safety factors. The most common column sections are bi-symmetric. The design compressive strength of short bi-symmetric columns is evaluated using equations given in D-2.

NOTE — While providing spiral ties it must be designed as per of [C-6B(2)], and the ratio of the volume of helical reinforcement to the volume of the core is not less than 0.36 $(A_g/A_{co}-1)f_{ck}/f_{yk}$.

(Where, A_g = Gross area of section; A_{co} = Area of core of the confined core of column measured to the outer diameter of the helix).

13.5.2 *Effective Elastic Flexural Stiffness*:

a) *Short Term Loading* — The effective elastic flexural stiffness, $(EI)_e$, is obtained by adding up the flexural stiffness of the individual components of the cross-section:

$$(EI)_e = E_s I_s + 0.6 E_{cm} I_c + E_{st} I_{st}$$

where

E_s and E_{st} = modulus of elasticity of the structural steel section and the reinforcement, respectively; and

E_{cm} = secant modulus of the concrete.

b) *Long Term Loading* — In slender columns ($\lambda > 0.2$), the effect of long-term loading should be considered.

$$(EI)_e = E_s I_s + 0.6 E_{ceff} I_c + E_{st} I_{st}$$

$$E_{ceff} = E_{cm} \frac{1}{1 + \left(\frac{P_{G,E_d}}{P_{E_d}}\right)\varphi_t}$$

where

φ_t = creep coefficient according to [C-6B(2)]

P_{E_d} = Total design axial load

P_{G,E_d} = Part of the *axial load* P_{E_d} that is caused due to the permanent gravity loads

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If the eccentricity *'e'* of loading is more than twice the cross-section dimension *'D'* ($e > 2D$), the effect on the bending moment distribution caused by increased deflections due to creep and shrinkage of concrete will be very small and may be neglected. The effect of long-term loading need not be considered if the non-dimensional slenderness (λ) of the composite column is less than the limiting values given in Table 12.

Table 12 Limiting Values of λ or Long Term Loading

(*Clause* 13.1 *and* 13.5.2)

Sl No.	Cross-sections	Braced Non-sway Systems	Un-braced and/or Sway System
(1)	(2)	(3)	(4)
i)	Concrete encased cross-section	0.8	0.5
ii)	Concrete filled cross-section	$\frac{0.8}{1-\delta}$	$\frac{0.5}{1-\delta}$
NOTE — δ is the steel contribution ratio defined as $\delta = \frac{A_s.f_y}{P_d \gamma_{m0}}$. P_d is defined in D-2.			

When λ exceeds the limits prescribed above and $e/D < 2$, the effect of creep and shrinkage of concrete should be considered by adopting the modulus of elasticity of concrete E_{cs} instead of E_{cm} where E_{cs} is defined as follows:

$$E_{cs} = 0.75E_{cm}\left[1 - \frac{0.5P_{dd}}{P}\right]$$

where

P =the applied factored axial load; and

P_{dd} =the part of the factored load permanently acting on the column.

The effect of long-term loading may be ignored for concrete filled tubular sections with $\lambda \leq 2.0$ provided that $\delta > 0.6$ for braced (non-sway) columns and $\delta > 0.75$ for unbraced (sway) columns.

13.6 Long Members Subjected to Axial Compression

The non-sway composite columns need not be checked for buckling if any one of the following conditions is satisfied:

a) $P < 0.1P_{cr}$ where P is the axial force in the column, and P_{cr} is the elastic buckling load of the column, defined in **13.5.1**.

b) Non-dimensional slenderness $\lambda < 0.2$.

For columns not satisfying the above conditions, safety against buckling strength shall be checked about the corresponding axis. The following equation needs to be satisfied for buckling load:

$$P \leq \chi \cdot P_d$$

where

P = the applied factored axial load;

P_d = is the design plastic resistance to compression (**D-2**);

$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}}$ = is the stability reduction factor;

$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$;

λ = Non-dimensional slenderness ratio;

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α = A value that represents initial imperfections and residual stresses which is based on the buckling curve to be adopted, which is further dependent on the type of section and the axis of bending as given below (*see* Table 13):

Curve *a* = for concrete-filled tubular sections with reinforcement percentage less than 3 percent of gross cross section area and buckling about both axes.

Curve b = for fully or partially concrete-encased I-sections buckling about the strong axis (z-z axis) of the steel sections and for concrete-filled tubular sections with steel reinforcement percentage more than 3 percent of gross cross section area and buckling about both axes or concrete filled tubular section with the addition of I section at the core and buckling about both axes.

Curve c = for fully and partially concrete-encased I-sections buckling about the weak axis (y-y axis) of the steel sections.

Table 13 Imperfection Factor *α* for the Buckling Curves

(*Clause* 13.6)

Sl No.	Buckling Class	a_0	*a*	*b*	*c*	*d*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	α	0.13	0.21	0.34	0.49	0.76

13.7 Members Subjected to Combined Compression and Bending

When the bending moment in the section is zero (that is, $M = 0$), the design compressive strength is as given in **13.5**. Similarly, the plastic moment of resistance of a column at zero compression load is as given **D-3**. The equations for evaluating the design bending resistance of symmetric column sections are given in **D-4**.

13.7.1 *Bending Resistance under High Shear*

While determining the plastic resistance when the simultaneous shear force is high at the section as given below, the bending resistance shall be reduced as given below:

If the shear force V on the steel section exceeds 60 percent of the design shear resistance V_d of the steel (**8.3.6.1**), the influence of the transverse shear on the resistance in combined bending and compression shall be taken into account by a reduced design steel strength $(1 - \beta) f_y / \gamma_{m0}$ in the shear area A_v (β is determined as per **8.3.6.2**).

a) Unless a more accurate analysis is used, the design shear action, V_d, may be distributed into (V_s, acting on the structural steel) and (V_c, acting on the reinforced concrete section) as:

$$V_s = V \frac{M_{ds}}{M_d}$$

$$V_c = V - V_s$$

where

M_{ds} = design moment of resistance of steel section alone;

M = design moment of resistance of the entire composite section; and

V_d = may be assumed to be resisted by steel sections alone as a simplifying approximation.

13.7.2 *Second Order Effects on Bending Moment*

If neither of the two conditions (a) and (b) for short compression members in **13.5.1** is satisfied, then the second-order effects on first-order analysis moment shall be considered by modifying the maximum first-order bending

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moment (moment obtained linear elastic analysis), M_{max}, with an amplification factor k, which is defined as:

$$k = \frac{C_m}{1 - \frac{P}{P_{cr}}} \geq 1.0$$

where

P = the applied factored axial load;

P_{cr} = elastic critical load for the relevant axis and corresponding to a modified effective flexural stiffness given by $(EI)_e$ as given in **13.5.2** and the effective length of the column is taken as per [C-6B(3)];

C_m = equivalent moment factor given in Table 14 to account for non-uniform bending moment over the length of the member; and

ψ = the ratio of moments at the ends of the laterally unsupported members.

13.8 Members Subjected to Axial Force and Uniaxial Bending

13.8.1 While checking a section for combined axial force and bending moment, first, it should be ensured that the section is safe against an axial force acting alone, considering buckling along each principal axis. The resistance of the section shall then be checked for combined axial compression and uniaxial bending moment, as described below:

The design against combined bending and axial compression are adequate when the following condition is satisfied:

$$M'_d = \alpha_m \mu_d M_d$$

where

α_m = reduction coefficient based on steel yield strength;

= 0.9 for 250 ≥ f_y ≤ 400 MPa;

= 0.8 for f_y > 400 MPa;

M'_d = design bending moment, which is to be amplified to account for the second-order effects as discussed in **13.7.2**, when the column is not short; and

M_d = Design bending resistance of the section about the bending axis evaluated as in **D-3**.

The moment resistance reduction ratio μ_d for a long composite column under combined compression and uniaxial bending shall be evaluated as follows:

$$\mu_d = \frac{(\chi - \chi_d)}{(1 - \chi_c)\chi} \quad \text{when } \chi_d \geq \chi_c \text{ and}$$

$$\mu_d = 1 - \frac{(1-\chi)\chi_d}{(1-\chi_c)\chi} \quad \text{when } \chi_d < \chi_c$$

χ_c = Axial resistance ratio to the concrete P_c/P_d;

χ_d = Design axial resistance ratio to the concrete P/P_d;

χ = reduction factor due to column buckling as per good pratice [C-6B(3)];

P_c = axial compressive strength of concrete;

= $A_c \frac{\alpha_{cc.}}{\gamma_c} . \eta . \lambda . f_{ck}$; and

P_d = *see* **D-2**.

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13.8.2 *Members Subjected to Axial Force and Bi-axial Bending*

Members subjected to combined axial compression and biaxial bending shall satisfy the following interaction relationships equations:

$$\frac{M_z}{\mu_{dz} M_{dz}} + \frac{M_y}{\mu_{dy} M_{dy}} \leq 1$$

$$\frac{M_z}{\mu_{dz} M_{dz}} \leq \alpha_{mz}$$

$$\frac{M_y}{\mu_{dy} M_{dy}} \leq \alpha_{my}$$

where

μ_{dz} and μ_{dy} = moment resistance reduction factors in z and y directions, respectively. (*see* 13.8)

$\alpha_{mz} = \alpha_{my} = \alpha_m$ (*see* 13.8)

Table 14 Equivalent Uniform Moment Factor

(*Clause* 13.7.2)

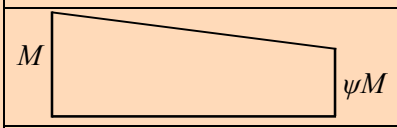
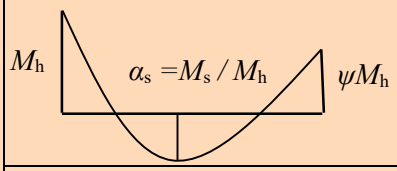
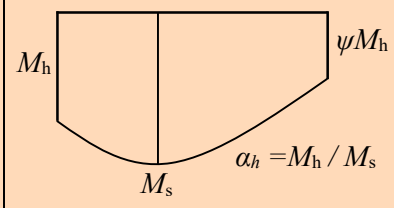
Bending Moment Diagram	Range		C_{my}, C_{mz}, C_{mLT}	
			Uniform Loading	Concentrated Load
(1)	(2)		(3)	(4)
	$-1 \leq \psi \leq 1$		$0.6 + 0.4\ \psi \geq 0.4$	
	$0 \leq \alpha_s \leq 1$	$-1 \leq \psi \leq 1$	$0.2 + 0.8\ \alpha_s \geq 0.4$	$0.2 + 0.8\ \alpha_s \geq 0.4$
	$-1 \leq \alpha_s \leq 0$	$0 \leq \psi \leq 1$	$0.1 - 0.8\ \alpha_s \geq 0.4$	$-0.8\ \alpha_s \geq 0.4$
		$-1 \leq \psi \leq 0$	$0.1(1-\psi) - 0.8\ \alpha_s \geq 0.4$	$0.2(-\psi) - 0.8\ \alpha_s \geq 0.4$
	$0 \leq \alpha_h \leq 1$	$-1 \leq \psi \leq 1$	$0.95 - 0.05\ \alpha_h$	$0.90 + 0.10\ \alpha_h$
	$-1 \leq \alpha_h \leq 0$	$0 \leq \psi \leq 1$	$0.95 + 0.05\ \alpha_h$	$0.90 + 0.10\ \alpha_h$
		$-1 \leq \psi \leq 0$	$0.95 + 0.05\ \alpha_h\ (1 + 2\ \psi)$	$0.90 + 0.1\alpha_h\ (1 + 2\ \psi)$
For members with sway buckling mode, the equivalent uniform moment factor $C_{my} = C_{mz} = 0.9$.				

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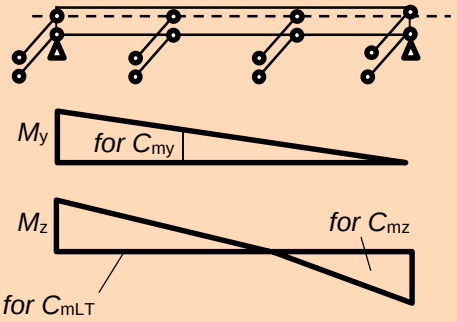
Table 14 (*Concluded*)

Bending Moment Diagram	**Range**	C_{my}, C_{mz}, C_{mLT}	
		Uniform Loading	Concentrated Load
(1)	(2)	(3)	(4)

C_{my}, C_{mz}, C_{mLT}shall be obtained according to the bending moment diagram between the relevant braced points

Moment factor	Bending axis	Points braced in direction
C_{my}	y-y	z-z
C_{mz}	z-z	y-y
C_{mLT}	z-z	z-z



13.9 Mechanical Shear Connection and Load Introduction

Proper sharing of loads between the steel section and concrete of composite columns should be ensured at points of load introduction due to load and moment reactions coming from members connected to the ends of the column and also for axial loads applied anywhere within the length of the column, considering the shear resistance at the interface between steel and concrete.

Where composite columns and compression members are subjected to significant transverse shear, for example, by local transverse loads and by end moments, the provision shall be made for the transfer of the corresponding local longitudinal shear stress at the interface between steel and concrete. For axially loaded columns and compression members, longitudinal shear outside the region of load introduction need not be considered.

13.9.1 *Load Introduction*

Shear connectors should be provided at regions of load introduction and at regions with change in cross section, if the design shear resistance at the concrete-steel interface, τ, (Table 15) is exceeded. The shear forces should be determined from the change of sectional forces of the steel or reinforced concrete section within the introduction length. If the loads are introduced into the concrete cross section only, the values resulting from an elastic analysis considering creep and shrinkage should be taken into account. Otherwise, the forces at the interface should be determined by elastic theory or plastic theory to determine the more severe case. In the absence of any accurate method, the introduction length should not exceed $2d$ or $L/3$, where d is the minimum transverse dimension of the column and L is the column length.

Table 15 Design Shear Strength (τ)

(*Clauses* 13.2, 13.9.1 *and* 13.9.3)

Sl No.	**Type of Cross-section**	**Design Shear Strength τ** (N/mm^2)
(1)	(2)	(3)
i)	Completely concrete-encased steel sections	0.30
ii)	Concrete filled circular hollow sections	0.55
iii)	Concrete filled rectangular hollow sections	0.40
iv)	Flanges of partially encased sections	0.20
v)	Webs of partially encased columns	0.00

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Due to the action of creep and shrinkage, no shear connection is required for composite columns or compression members if the load application is by endplate, where the full interface between steel and concrete is permanently under compression. Otherwise, the load application/introduction should be verified as given below:

a) If the cross-section is partially loaded, as shown in Fig. 20 (a), the loads may be distributed with a ratio of 1 : 2.5 over the thickness, t_e of the endplate. The concrete stresses should then be limited in the area of effective load introduction.

b) For concrete-filled circular hollow section or square hollow section, under partial loading as shown in Fig. 20(b), for example, by gusset plates or by stiffeners, the local design strength of concrete σ_c under the gusset or stiffener resulting from the sectional forces of the concrete section shall be determined as:

$$\sigma_c = \frac{\alpha_{cc} f_{ck}}{\gamma_c}\left[1 + \eta_0 \frac{t}{d}\frac{f_y}{\alpha_{cc} f_{ck}}\right]\sqrt{\frac{A_c}{A_1}} \leq \frac{A_c . \alpha_{cc} f_{ck} .}{A_1 . \gamma_c}; \leq \frac{f_y}{\gamma_{m0}}$$

where

$A_c/A_1 \leq 20$

t = is the wall thickness of the steel tube;

d = diameter of the tube or width of the square section;

A_c = is the cross sectional area of the concrete section of the column;

A_1 = is the loaded area under the gusset plate (*see* Fig. 20); and

η_0 = 4.9 for circular steel tubes and 3.5 for square sections.

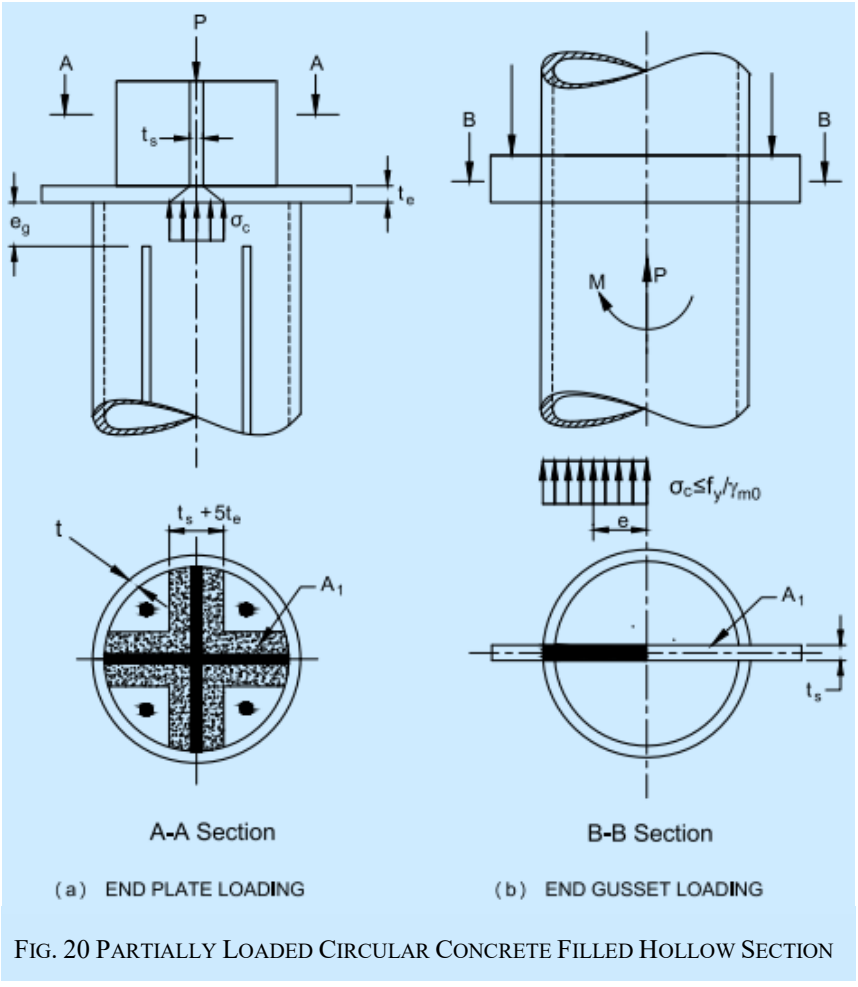

FIG. 20 PARTIALLY LOADED CIRCULAR CONCRETE FILLED HOLLOW SECTION

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c) For concrete-filled circular hollow sections, longitudinal reinforcements may be taken into account while determining the resistance of the composite column, even if the reinforcement is not directly connected to the endplate, provided that the gap [Fig. 20(a)] between the end of reinforcement and the surface of the endplate does not exceed 30mm.

13.9.2 *Shear connection*

When a mechanical connection is introduced in the form of stud connectors to the web of a fully or partially concrete-encased steel I-section, an account may be taken of the frictional forces that develop due to the restraint in lateral expansion of the concrete by the adjacent steel flanges. This resistance is assumed to be equal to $\mu.Q_u/2$ on each flange and each horizontal row of studs, as shown in Fig. 21 and may be added to the calculated resistance of the shear connectors. μ is the relevant coefficient of friction and be taken as 0.5. Q_u is the resistance of a single stud as per **11.2**.

The clear distance between the flanges should not exceed the values given in Fig. 21 to ensure the development of the frictional forces between concrete and steel flanges.

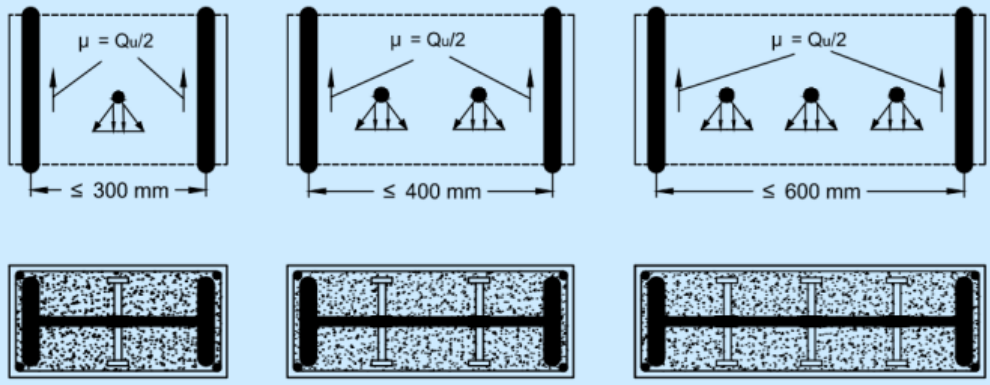

FIG. 21 ADDITIONAL FRICTIONAL FORCES IN COMPOSITE COLUMNS BY USE OF HEADED STUDS

13.9.3 *Longitudinal Shear Outside Area of Load Introduction*

a) Outside the area of load introduction, longitudinal shear at the interface between concrete and steel should be verified where it is caused by transverse loads and /or end moments. Shear connectors should be provided based on the distribution of the design value of longitudinal shear, where this exceeds the design shear resistance τ. In the absence of a more accurate method, elastic analysis, considering long-term effects and cracking of the concrete, may be used to determine the longitudinal shear at the interface. Provided that the surface of the steel section, in contact with the concrete, is unpainted and free from oil, grease, and loose scale or rust, the values given in Table 15 may be assumed for τ. The value of τ given in Table 15 for fully concrete-encased steel sections applies to sections with a minimum concrete cover of 40 mm. For greater concrete cover and adequate reinforcement, higher values of τ may be used. Unless verified by tests, for completely encased sections, the increased value β_c. τ may be used, with β_c given by:

$$\beta_c = 1 + 0.02\, c_z \left[1 - \frac{c_{z,min}}{c_z}\right] \leq 2.5$$

where

c_z = is the wall thickness of the steel tube; and

$c_{z,min}$ = diameter of the tube or width of the square section.

b) Unless otherwise verified, for partially encased I-sections with transverse shear due to bending about the weak axis caused by lateral loading or end moments, shear connectors should always be provided. If the resistance of the structural steel section alone against transverse shear is not sufficient to take care of the total transverse shear on the composite section, then the required transverse reinforcement for the shear force, V_c according to **12.5.2**, should be welded to the web of the steel section or should pass through the web of the steel section.

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13.10 Shear Check

The factored shear force in the compression members should be less than the design shear strength of the member, which is the sum total of the shear resistance given by the concrete section along with steel reinforcements as per good practice [C-6B(2)] and the shear resistance given by the steel section as per **8.3.6**. The shear force shall be distributed between the steel section and the concrete section in accordance with **13.9**.

14 CONSTRUCTION AND ERECTION

14.1 Fabrication and Inspection Procedure for Structural Steel

Fabrication and erection of steel sections and components, as and where situated in a composite structure would include fabrication procedures, both shop and site fabrications, along with fabrication tolerances, inspection, testing, handling, transportation, site storage, erection along with erection tolerances, etc.

Fabrication and erection specifications of all steel components of a composite structure shall refer to stipulations laid down in good practice [C-6B(3)].

15 FIRE RESISTANCE AND FIRE DESIGN

15.1 In closed structures, an accidental fire may lead to a rise in temperature under which failure of the material may take place unless a proper design and construction against fire is carried out. Open structures such as bridges are not generally vulnerable to failure under fire since the temperature does not go up to the level which may cause material damage. Also, in an open structure, the fire can be extinguished easily and quickly. Fire tests on open structures such as elevated parking lots and bridges have shown that the structure does not undergo any material damage due to reasons indicated above. However, all structures, including both closed and open ones, shall be protected from all possible accidental fires caused by different kinds of hazards.

Fire-resistant designs for open and closed structures at specialized locations, such as those in proximity to oil installations or pipelines carrying inflammable materials, etc as in the case of industrial buildings and structures, shall be done based on recommendations given in specialized literature. Also, adequate provisions may be made for firefighting equipment to access all parts of the structure as per Part F ‘Fire and Life Safety’ of this NBCS. After the occurrence of fire in a structure, it should be mandatory for the concerned authorities to have the structure inspected by competent experts in order to ascertain the condition of the structure before it can be declared safe for re-use.

In addition to the above, locations in any structural system that may be prone to accidental occurrence of fire, shall be adequately provided with basic fire protection methods as per specialist literature. These will include both active as well as passive fire protection.

15.2 Response to Fire

Steel is a good conductor of heat that experiences an almost uniform temperature increase over its entire volume. Concrete is a poor conductor of heat has a large temperature gradient within its volume. Due to this, the entire steel member gains the heat faster, whereas the surfaces of concrete exposed directly to heat experience high temperature, with the temperature dropping off drastically in the interior of the concrete mass.

Both steel and concrete exhibit a drastic reduction in strength with an increase in temperature above 400 °C. Due to this, the resistance of steel members tends to decrease drastically above 400 °C, whereas concrete at the surfaces exposed to high temperature tends to spall off due to high temperature gradient. The steel structure, if it does not fail under fire, regains its strength after the fire, the damage at the surface of the concrete is permanent due to chemical changes. The steel reinforcement in concrete is protected from loss of strength due to an increase in temperature by providing adequate concrete cover as per good practice [C-6B(2)]. Composite floor system deck sheeting is to be provided with fire protection.

However, literature shows that the minimum necessary fire resistance can be achieved by using the composite floor systems and the composite columns, without applying fire protection. Interaction of the steel beam and the concrete slab in the composite flooring leads to a higher critical temperature and higher fire resistance.

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The requirements for fire resistance shall apply to steel elements of steel-concrete composite structure designed to exhibit a required fire resistant level as per the relevant specifications.

15.2.1 *Design Fire*

The design of a structure against fire load is dependent on the required fire resistant level (FRL), which is dependent on the function of the structure itself and the period of structural adequacy (PSA). These shall be calculated based on the stipulation laid down in as per good practice [C-6B(3)].

The response of steel elements of a composite structure against fire as laid down in as per good practice [C-6B(3)] shall be binding on all steel elements of the composite structures which are susceptible to rise in temperature during its design life. For all general buildings, an adequate fire protection methodology shall be adopted.

15.2.2 *Fire Protection Methodology*

Apart from the direct design of steel components against fire as required in as per good practice [C-6B(3)] (protected as well as unprotected section), other protective measures, (both active and passive), also may be adopted as recommended in literature as fire-resistant procedures.

For active fire resistance, provisions of fire locating and fighting measures like the smoke detectors, fire extinguishers inside a building along with accessory fire water supply, sprinkler system, etc, shall be made available at vantage points. The planning of the structure shall be made in such a way that it is accessible from all sides to fire extinguishing vehicles.

For passive fire resistance, protective paints and materials like intumescent paints, vermiculite boards etc, may be used on exposed steel surfaces, and their provision shall be made as per the required fire-resistant level and as per their properties and specification provided by the manufacturers. The design of protected sections shall be done as per the stipulations laid down in the relevant section of as per good practice [C-6B(3)]. Detailed design against the fire may be based on provisions of other composite codes and specialized literature.

15.2.3 *Fire Resistance of Composite Slabs*

The fire resistance is assumed based on the following two criteria:

a) Thermal insulation criterion, concerned with limiting the transmission of heat by conduction; and

b) Integrity criterion is concerned with preventing the flames and hot gases from nearby compartments. It is met by specifying an adequate thickness of insulation.

16 TESTING METHODS

For testing methods, Annex E may be referred.

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ANNEX A

(*Clause* 4)

SYMBOLS

A-1 The letter symbols and notations given below apply to the provision of this section:

a	Span length of the slab transverse to the steel beams
A_s	Area of structural steel cross section
A_{sl}	Area of structural steel cross section in tension
A_c	Gross area of concrete
A_{co}	Equivalent area of the cracked composite beam section
A_{ec}	Area of concrete effective in compression
A_p	Effective cross-sectional area of profile steel sheeting
A_{st}	Area of steel reinforcements
A_f	Area of each flange of steel section
A_e	Effective cross sectional area
A_{sv}	Area of shear resistance
B	Centre to centre distance between beams and is equal to the transverse span of inner beam
b_c	Width of concrete encasement
b_p	Distance between the centres of adjacent ribs or one wave length of a profile deck
b_e	Effective width of the flange between pair of bolts
b_{eff}	Effective width of concrete flange
b_f	Width of the structural steel flange
D	Overall depth of girder/diameter of the steel cross section.
d	Depth of the web, Nominal diameter of bolts/rivets/studs.
d_2	Twice the clear distance from the compression flange of angles, plates, or tongue plates to the neutral axis
d_c	Vertical distance between the centroid of concrete slab and centroid of steel beam
d_s	Overall depth of the concrete slab
d_o	Nominal diameter of the pipe column or the dimensions of the column in the direction of depth of the base plate
d_p	Depth of profiled sheet deck
d_{sp}	Overall depth of the profiled sheet deck and concrete slab
e	Distance between the C.G. axis of the profiled steel sheeting and the extreme tension fiber of the composite slab
e_p	Distance between the plastic neutral axis and the extreme tension fiber of the composite slab
$E_{c,l}$	Modulus of elasticity of lightweight concrete
E_s	Modulus of elasticity of structural steel
E_{cm}	Secant Modulus of elasticity of concrete
E_{st}	Modulus of elasticity of steel reinforcements
F_f	Axial capacity of a single flange
F_{sr}	Axial capacity of reinforcing steel

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F_{cc} Compressive force in the concrete above the neutral axis

F_{sc} Compressive force in the steel section above the neutral axis

F_{st} Tensile force in the steel section below the neutral axis

F_{w} Design capacity of the web in bearing

f Actual normal stress range for the detail category

f_{c} Actual axial compressive stress in concrete at service load

$f_{ctm, l}$ Tensile strength of lightweight concrete

f_{ck} Characteristic compressive cube strength of concrete at 28 days

$f_{ctk(0.05)}$ Characteristic axial tensile strength of concrete based on 5 percent fractile

f_{ctm} Mean axial tensile strength of concrete

f_{f} Fatigue stress range corresponding to 5×10^{6} cycles of loading

f_{fd} Design fatigue normal stress range

f_{feq} Equivalent constant amplitude stress

$f_{f\,max}$ Highest normal stress range

f_{fn} Normal fatigue stress range

f_{o} Proof stress

f_{yk} Characteristic yield strength of reinforcement

f_{yf}, f_{yw} Yield strength of flange and web, respectively

f_{u} Characteristic ultimate tensile stress

f_{up} Characteristic ultimate tensile stress of the profiled deck

f_{y} Characteristic yield stress of structural steel

f_{yp} Characteristic yield stress of profiled deck

h Depth of the section/total depth of steel beam

H Distance between top of concrete and bottom of bottom flange of steel beam

h_{s} Distance between centroids of top and bottom flanges /nominal height of stud

h_{y} Distance between shear centre of the two flanges of the cross section

I Moment of inertia of the member about an axis perpendicular to the plane of the frame

I_{c} Moment of inertia of concrete (assumed uncracked) about the axis of bending for column

I_{co} Moment of inertia of the composite section

$I_{fc,y}$ Moment of Inertia of the compression flange about the minor axis of the steel beam

$I_{ft,y}$ Moment of Inertia of the tension flange about the minor axis of the steel beam

I_{s} Moment of inertia of the steel section about the axis of bending for column

I_{st} Moment of inertia of reinforcement about the axis of bending for column

I_{z} Moment of inertia about the major axis

I_{ys}, I_{zs} Second moment of area of steel section about the minor and major axis respectively

I_{ybf} Moment of inertia of the bottom flange about the minor axis of the steel section

I_{zco} Second moment of the equivalent area of the cracked composite beam section for major axis bending

k_{1} Flexural stiffness of the steel web per unit length along the beam

k_{2} Flexural stiffness of the cracked concrete or composite slab transverse to the spanning direction of steel beam

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KL	Effective length of the member
KL/r	Appropriate, effective slenderness ratio of the section
KL/r_y	Effective slenderness ratio of the section about the minor axis
KL/r_z	Effective slenderness ratio of the section about the major axis
L	Actual span of composite beams
L_{bs}	Minimum bearing lengths of steel decking on the support
L_{bc}	Composite slab including the cast in place concrete
L_c	Effective span of the cantilever for overhang
L_o	Length between points of zero moments (inflection) in the span
L_s	Shear span
L_x	Distance of the maximum moment cross section under consideration to the support
M	Bending moment
M_v	Reduced bending moment due to effect of shear force
M_{cr}	Elastic critical moment corresponding to lateral torsional buckling
M_d	Design bending strength/design bending resistance under only bending moment
M'_d	Design bending resistance under combined bending and compression
M_{dp}	Design bending resistance of profiled steel sheeting
M_e	Elastic moment capacity of the section
M_f	Design plastic resistance of the flange alone for steel section
M_p	Plastic moment capacity of the section
M_y	Factored applied moments about the minor axis of the cross section
M_z	Factored applied moments about the major axis of the cross section
m	Modular ratio
m_{dl}	Modular ratio (long term)
m_{ll}	Modular ratio (short term)
N_{SC}	Number of stress cycles
P	Design axial force
P_{cr}	Elastic critical buckling load
P_p	Plastic resistance of encased steel column section or concrete filled rectangular or square column section
R_h	Flange stress reduction factor for hybrid section
r	Appropriate radius of gyration
r_y	Radius of gyration about the minor axis
r_z	Radius of gyration about the major axis
S	Spacing
S_l	Spacing of shear connectors for longitudinal shear due to flexural force
S_r	Spacing of shear connectors due to bending moment
t	Thickness of element/angle, time in minutes
t_f	Thickness of flange of steel section
t_p	Thickness of profiled deck sheet

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t_w	Thickness of web of steel section,
V, V_v, V_L	Factored applied shear force
V_d	Design shear resistance
V_{dvd}	Vertical shear resistance
V_p	Plastic shear resistance under pure shear
V_{vd}	Design vertical shear resistance
W	Total equivalent load
X	Distance from the centre line of edge beam to the edge of slab
x_e	Depth of elastic neutral axis of the composite section from the centroid of steel section
x_u	Depth of neutral axis at limit state of flexure from top of the concrete
Z_e	Elastic section modulus
Z_p	Plastic section modulus
Z_{pc}, Z_{pcn}	Plastic section modulus of concrete about its own centroid and about the neutral axis of the composite section, respectively
Z_{pr}, Z_{prn}	Plastic section modulus of reinforcement about its own centroid and about the neutral axis of the composite section, respectively
Z_{ps}, Z_{psn}	Plastic section modulus of structural steel section about its own centroid and about the neutral axis of the composite section, respectively
y_g	Distance between the point of application of the load and shear centre of the cross section
α	Imperfection factor
α_c	Strength coefficient of concrete
δ	Steel contribution ratio
χ	Stress reduction factor due to buckling under compression
χ_m	Stress reduction factor, χ at f_{ym}
χ_m	Stress reduction factor to account for lateral torsional buckling of a beam
γ	Unit weight of steel
γ_c	Partial safety factor for material (concrete)
γ_f	Partial safety factor for load
γ_{m0}	Partial safety factor against yield stress and buckling (structural steel)
γ_{m1}	Partial safety factor against ultimate stress (structural steel)
γ_{mf}	Partial safety factor for the strength of bolt
γ_{fft}	Partial safety factor for fatigue load
γ_{mft}	Partial safety factor for fatigue strength
γ_{mv}	Partial safety factor against shear failure
γ_{mw}	Partial safety factor for the strength of the weld
γ_k	Partial safety factor for material (reinforcements)
ε	Yield stress ratio, (250/*fy*) 1/2
λ, λ_r	Non dimensional slenderness ratio = $\sqrt{f_y(KL/r)^2/\pi^2E} = \sqrt{f_y/f_{cr}} = \sqrt{P_y/P_{cr}}$
λ_e	Equivalent slenderness ratio
μ_s	Poisson's ratio of structural steel

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μ_c	Correction factor
η_1, η_2	Strength coefficients
η_{10}, η_{20}	Strength coefficients
τ	Actual shear stress range for the detail category
τ_f	Fatigue shear stress range
τ_{fd}	Design fatigue shear stress range
$\tau_{f,max}$	Highest shear stress range
τ_{fn}	Fatigue shear stress range at N_{SC} cycle for the detail category

ANNEX B

(*Clauses* 8.1.1, 8.1.2, 8.3.5.2, 8.3.5.3 *and* 8.3.7)

BENDING RESISTANCE OF COMPOSITE BEAMS

The stress distribution and the methods of evaluation of the sagging and hogging bending resistances under full and partial shear interaction of composite beams are discussed in this Annex.

B-1 SAGGING MOMENT RESISTANCE OF COMPOSITE SECTION WITH PLASTIC OR COMPACT STRUCTURAL STEEL BEAMS

B-1.1 Sagging Bending Resistance with Full Shear Interaction

The equations for calculating the bending resistance of composite beams are presented in this section. It is assumed that sufficient shear connectors are provided between the structural steel and concrete to develop full shear with no-slip during the service load.

Initially, evaluating the resistance is based on parabolic stress block as per good practice [C-6B(2)], over the entire depth of concrete in compression is presented. This is theoretically more accurate than the rectangular stress block (maximum of 2-3 percent difference), but computationally more tedious, particularly when it comes to sections subjected to combined bending and compression.

Subsequently, equations based on rectangular stress block for concrete in compression is presented. This is assumed so that the calculations are simplified. This method is easy to use, particularly when the neutral axis is in or closer to compression flange of the steel section.

In both the above methods, the stress in the reinforcing steel in the concrete slab under compression is disregarded.

B-1.1.1 *Based on Parabolic Stress Block Satisfying Stress Strain Relationship*

Calculation of bending resistance is based on parabolic stress block, which is compatible with strain diagram. The design bending resistance depends on the location of the plastic neutral axis. Table 16 gives the various cases of design bending moments of a composite section depending upon the location of the plastic neutral axis as shown in Fig. 22, Fig. 23 and Fig. 24. For hybrid sections, appropriate yield strength in flanges and web shall be considered for calculation of the plastic moments.

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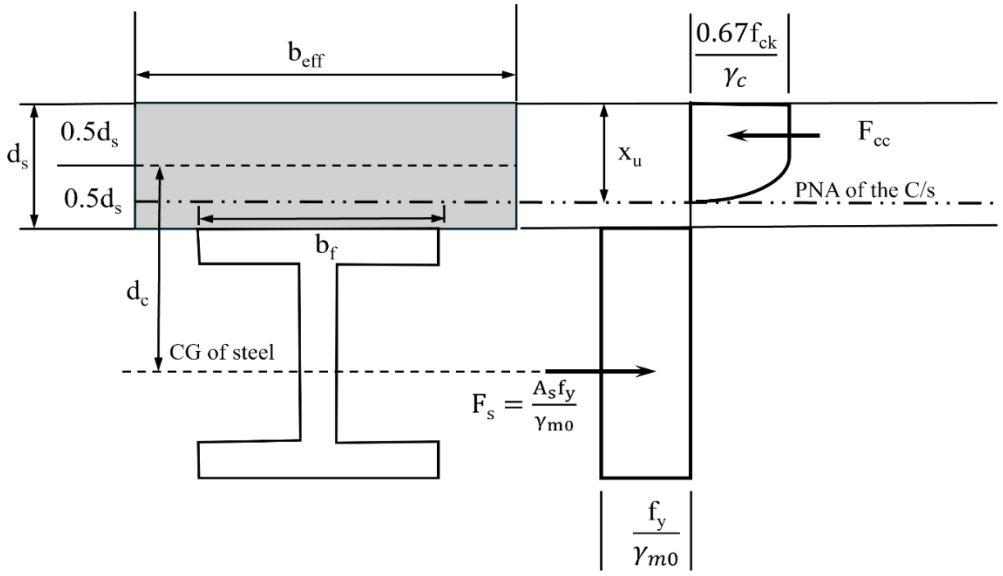

FIG. 22 STRESS DISTRIBUTION IN A COMPOSITE BEAM WITH PLASTIC NEUTRAL AXIS WITHIN THE CONCRETE SLAB AT ULTIMATE MOMENT

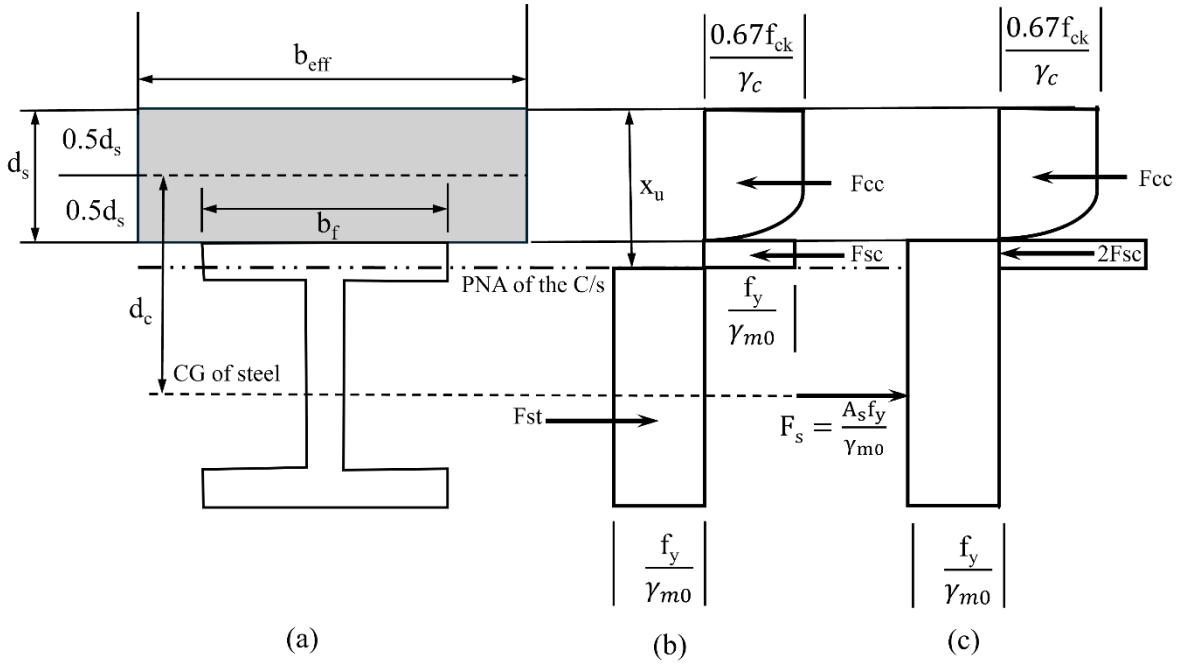

FIG. 23 STRESS DISTRIBUTION IN A COMPOSITE BEAM WITH PLASTIC NEUTRAL AXIS WITHIN THE FLANGE OF STEEL BEAM AT ULTIMATE MOMENT

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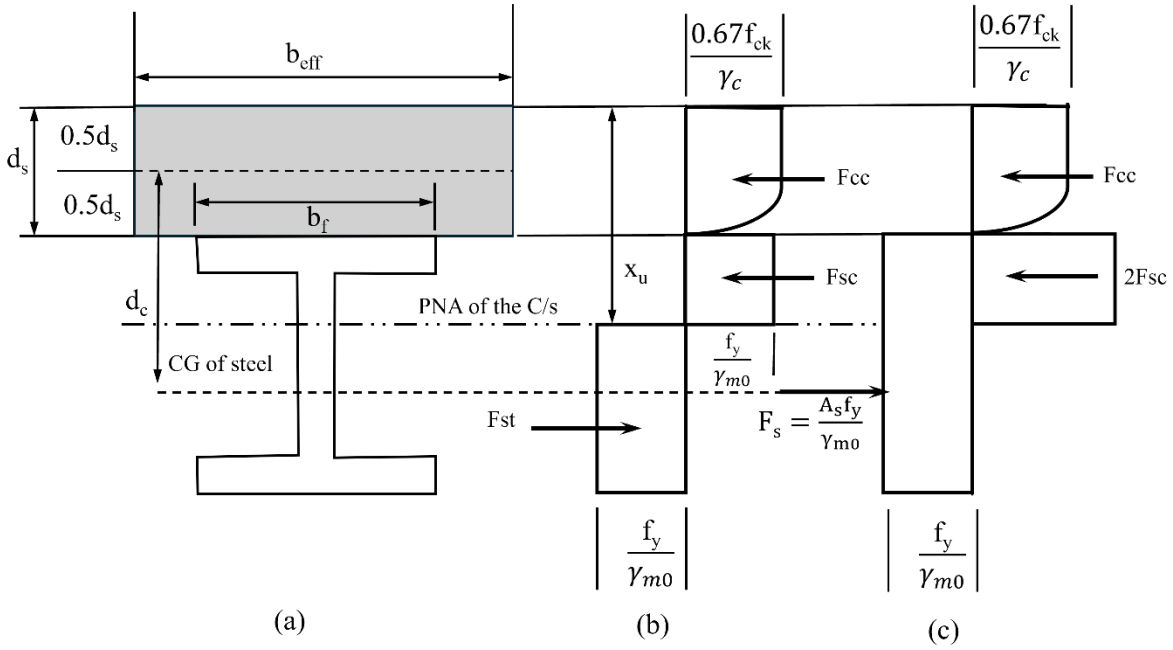

FIG. 24 STRESS DISTRIBUTION IN COMPOSITE BEAM WITH PLASTIC NEUTRAL AXIS WITHIN THE WEB OF STEEL BEAM AT ULTIMATE MOMENT

Table 16 Sagging Moment Resistance of Composite Section with full Shear Interaction (Parabolic Stress Block)

(*Clause* B-1.1.1)

Case	Position of Plastic Neutral Axis	Location of PNA x_u	Design Moment Capacity M_d
(1)	(2)	(3)	(4)
1	Within slab (Fig. 22) $b_{eff}\, d_s > a.A_s$	$x_u = aA_s/b_{eff}$	$M_d = A_s\,(f_y/\gamma_{m0})\,(d_c + 0.5d_s - 0.42\,x_u)$
2	In steel flange (Fig. 23) $b_{eff}.d_s < a\,A_s < (b_{eff}\,d_s + 2a\,A_f)$	$x_u = d_s + \frac{(aA_s - b_{eff}d_s)}{2b_f a}$	$M_d = (f_y/\gamma_{m0})\,[\{A_s\,(d_c + 0.08d_s)\} - \{b_f\,(x_u - d_s)\,(x_u + 0.16\,d_s)\}]$
3	In web (Fig. 24) $a\,A_s > (b_{eff}\,d_s + 2a\,A_f)$	$x_u = d_s + t_f + \{a(A_s - 2A_f) - b_{eff}\,d_s\}\,/(2at_w)$	$M_d = (f_y/\gamma_{m0})\,[\,\{A_s\,(d_c + 0.08d_s)\} - \{2A_f\,(0.5t_f + 0.58d_s)\} - \{2t_w\,(x_u - d_s - t_f)\,(0.5\,x_u + 0.08\,d_s + 0.5\,t_f\,\}]$

where $a = \frac{f_y}{0.36\,f_{ck}}$ and d_s is taken as per **8.1.2.1**

B-1.1.2 *Based on Rectangular Stress Block*

The design bending resistance depends on the location of the plastic neutral axis, as shown in Fig. 25, Fig. 26 and Fig. 27. Table 17 gives the equations for evaluating the bending resistance of a composite section, depending upon the location of the plastic neutral axis. For hybrid sections appropriate yield strength in flanges and web shall be considered for calculation of the plastic moments.

Fig. 26 (b) and 27 (b), show the actual stress distribution and Fig. 26 (c) and 27 (c), show the equivalent stress distribution to simplify the equations.

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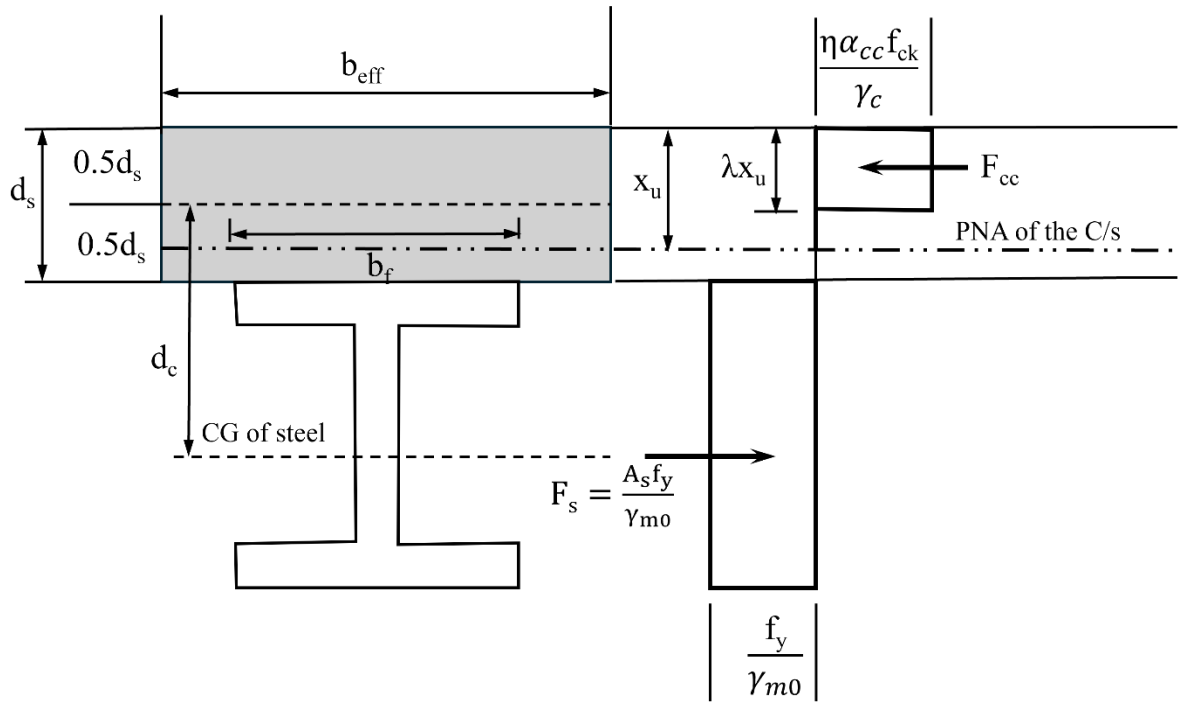

FIG. 25 STRESS DISTRIBUTION IN A COMPOSITE BEAM WITH PLASTIC NEUTRAL AXIS WITHIN CONCRETE SLAB AT ULTIMATE MOMENT

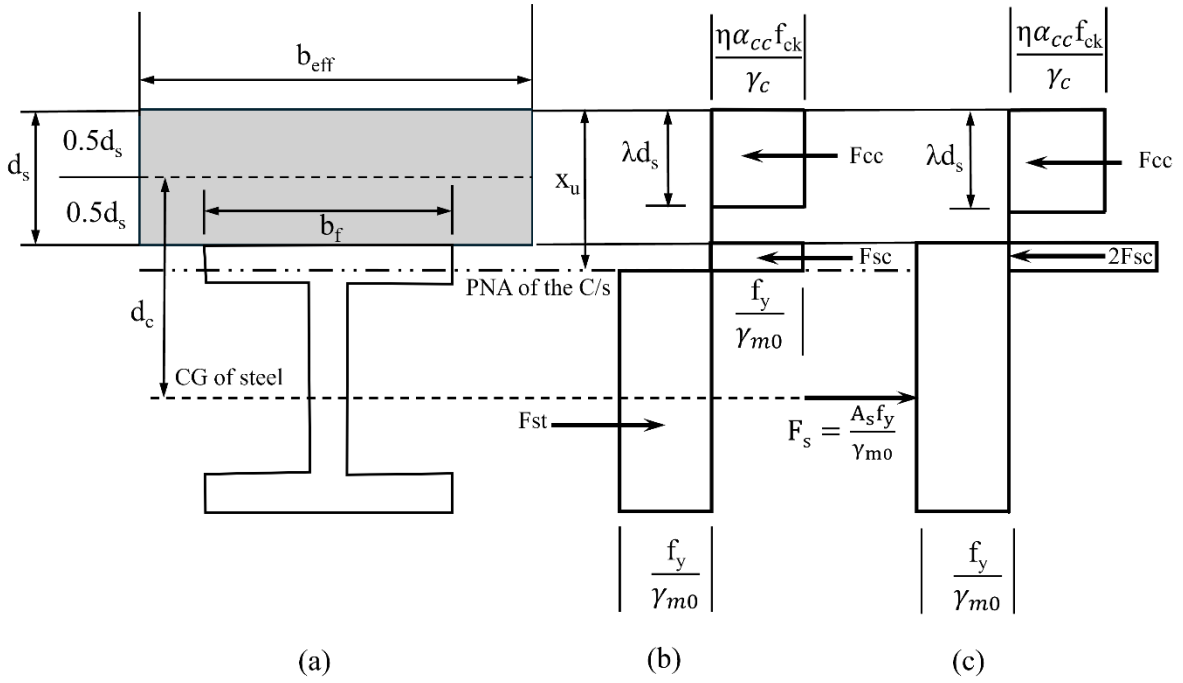

FIG. 26 STRESS DISTRIBUTION IN A COMPOSITE BEAM WITH PLASTIC NEUTRAL AXIS WITHIN THE FLANGE OF THE STEEL BEAM AT ULTIMATE MOMENT

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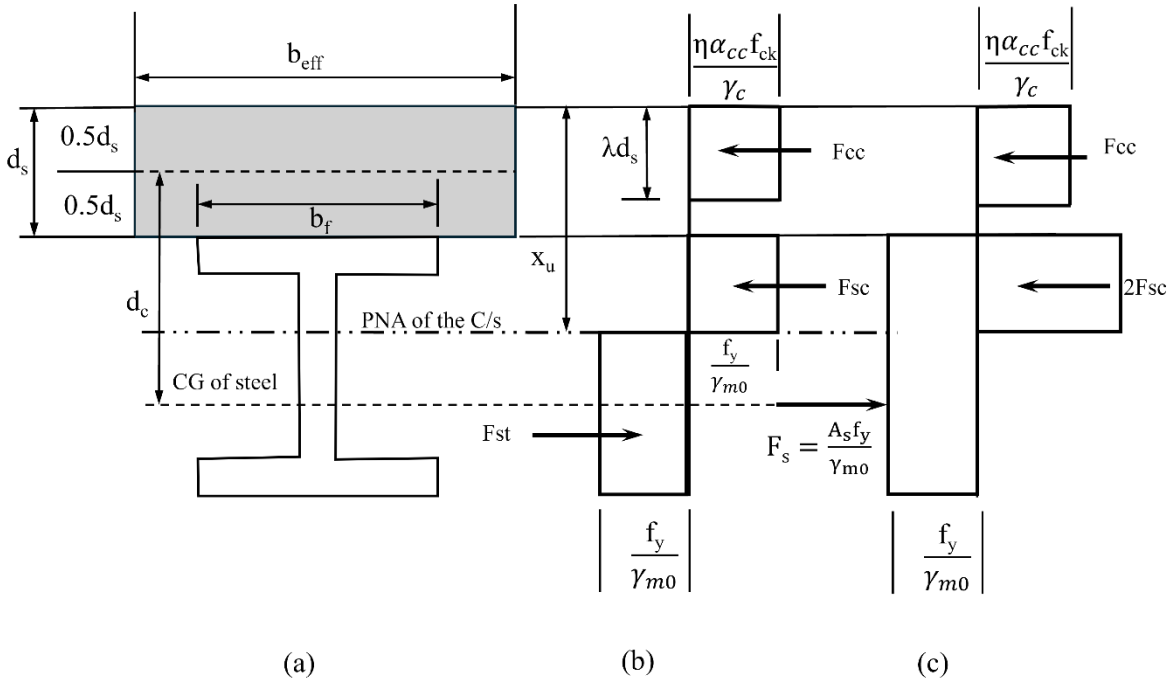

FIG. 27 STRESS DISTRIBUTION IN A COMPOSITE BEAM WITH PLASTIC NEUTRAL AXIS WITHIN THE WEB OF THE STEEL BEAM AT ULTIMATE MOMENT

Table 17 Sagging Moment Resistance of Composite Section with Full Shear Interaction (Rectangular Stress Block)

(*Clause* B-1.1.2)

Case	Position of Plastic Neutral Axis	Location of PNA x_u	Design Moment Capacity, M_d
(1)	(2)	(3)	(4)
1	Within slab (Fig. 25) $b_{eff}\, d_s > a\, A_s$	$x_u = aA_s/b_{eff}$	$M_d = A_s f_y / \gamma_{m0}\,(d_c + 0.5d_s - \lambda\, x_u/2)$
2	Plastic neutral axis in steel flange (Fig. 26) $b_{eff}\, d_s < a\, A_s < (b_{eff}\, d_s + 2a\, A_f)$	$x_u = d_s + \frac{(aA_s - b_{eff}d_s)}{2b_f a}$	$M_d = f_y / \gamma_{m0}\,[A_s\,\{d_c + 0.5d_s\,(1 - \lambda)\} - b_f\,(x_u - d_s)\,\{x_u + (1 - \lambda)\,d_s\}]$
3	Plastic neutral axis in web (Fig. 27) $b_{eff}\, d_s + 2a\, A_f < a\, A_s$	$x_u = d_s + t_f + \frac{a(A_s - 2A_f) - b_{eff}d_s}{2at_w}$	$M_d = f_y / \gamma_{m0}\,[A_s\,\{d_c + 0.5\,d_s\,(1 - \lambda)\} - 2A_f\,\{0.5t_f + (1 - \lambda/2)\,d_s\} - t_w\,(x_u - d_s - t_f)\,\{x_u + (1 - \lambda)\,d_s + t_f\}]$
where $a = \frac{f_y / \gamma_{m0}}{\alpha_{cc}/\gamma_c \cdot \eta \cdot \lambda \cdot (f_{ck})}$ and d_s is taken as per **8.1.2.1**.			

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Table 17 (*Concluded*)

Case	Position of Plastic Neutral Axis	Location of PNA x_u	Design Moment Capacity, M_d
(1)	(2)	(3)	(4)

NOTE — Notations used in the determination of plastic bending resistance

A_f = area of the top flange of steel beam of a composite section.

A_s = cross sectional area of structural steel beam of a composite section.

b_{eff} = effective width of concrete slab.

b_f = width of top flange of steel section.

d_s = overall depth of concrete slab.

d_c = vertical distance between centroids of concrete slabs and steel beam in a composite section.

t_f = average thickness of the top flange of the steel section.

t_w = thickness of the web of the steel section.

x_u = depth of neutral axis at ultimate limit state of flexure from top of concrete.

M_d = design bending resistance.

α_{cc} = 0.67.

γ_c = material safety factor for concrete .

= 1.50 (for basic and seismic combinations).

= 1.0 (for accidental combinations).

γ_{m0} = material safety factor for structural steel = 1.10

γ_k = material safety factor for reinforcing steel = 1.15

η = 1.0 [for $f_{ck} \leq 60$ MPa]

= 1.0 - (f_{ck} - 60) / 250 [for $60 < f_{ck} \leq 110$ MPa]

λ = 0.8 [for $f_{ck} \leq 60$ MPa]

= 0.8 - (f_{ck} - 60) / 500 [for $60 < f_{ck} \leq 110$ MPa]

B-1.2 Bending Moment with Partial Shear Interaction

B-1.2.1 Partial shear connection is applicable only in the case of plastic resistance of sections with plastic and compact classifications. Provisions for partial shear connection is applicable either for attaining economy without losing much in moment capacity of the composite section or in conditions where the number of shear connectors required for full shear interactions cannot be provided due to lack of space or increased spacing of shear connectors due to provision of lesser numbers of shear connectors than that needed for full composite action.
Degree of shear connection S_c is given as

$$S_c = \frac{n_p}{n_f} = \frac{F_{cp}}{F_{cf}} = \frac{M - M_{ds}}{M_d - M_{ds}}$$

where

n_p = Number of shear connectors provided for partial shear connection;

n_f = Number of shear connectors required for full shear connection;

F_{cp} = Capacity of shear connectors in partial shear connection with n_p numbers of connectors;

F_{cf} = Capacity of shear connectors in full shear connection with n_f numbers of connectors;

M = Required bending resistance of the section;

M_d = Design bending resistance of the composite section; and

M_{ds} = Design bending resistance of steel section alone.

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B-1.2.2 To obtain required bending resistance M, the number of shear connectors required (assuming all connectors have equal capacity) is given as:

$$n_{\mathrm{p}} = n_{\mathrm{f}}\{(M - M_{\mathrm{ds}})/(M_{\mathrm{d}} - M_{\mathrm{ds}})\}$$

B-1.3 Sagging Bending Resistance of Composite Section with Non-Compact Structural Steel Beams

Since the compression flange of semi-compact steel sections buckle locally under compression on reaching yield stress f_y, the design resistance of the composite section consisting of semi-compact sections is evaluated using equations of compact sections as per **B-1**, wherein the effective width of elements of section like the compression flange is restricted to limiting value of that of the compact section.

B-2 MOMENT OF RESISTANCE OF COMPOSITE SECTION (NEGATIVE MOMENT-CONTINUOUS BEAMS)

Figure 28 shows stress distribution across a composite beam section subjected to hogging bending moment. Since the steel bottom flange is in compression section classification shall be done as per [C-6B(3)]. For classification of the web, the distance, $\overline{y}$, of the plastic neutral axis above the centroid of the area of the steel section shall first be found.

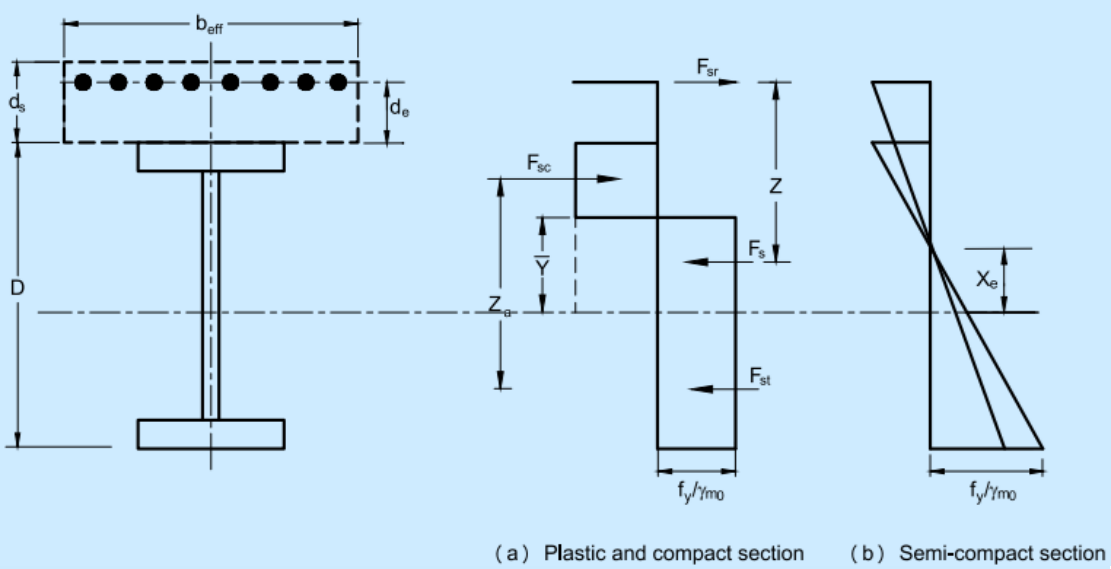

FIG. 28 STRESS DIAGRAM FOR HOGGING MOMENT REGION AT ULTIMATE MOMENT

B-2.1 Hogging Bending Resistance for Plastic and Compact Structural Steel Sections

The design tensile force in reinforcement is given as:

$$F_{\mathrm{sr}} = f_{\mathrm{yk}}.A_{\mathrm{st}}/\gamma_{\mathrm{k}}$$

where

γ_{k} = partial safety factor for reinforcing steel = 1.15;

f_{yk} = Characteristic yield strength of the reinforcing steel; and

A_{st} = the effective area of longitudinal reinforcement within the effective width b_{eff} of the beam

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The plastic moment of resistance for plastic and compact structural steel section is given as:

$$M_{ds} = \frac{Z_p . f_y}{\gamma_{m0}}$$

where

Z_p is the plastic section modulus of the structural steel section and γ_{m0} is the material safety factor to be taken as 1.10. In the absence of any tensile reinforcements, the bending resistance of the section would be that of the structural steel section as given by M_{ds} above. To allow for reinforcements, it is assumed that the stress in a depth $\overline{y}$ changes from tension to compression for Plastic and compact section. The corresponding depth for a non-compact section is x_e. For plastic and compact sections, stress distribution $\overline{y}$ may be determined from

$$\overline{y}\, t_w \frac{2 f_y}{\gamma_{m0}} = F_s$$

The locations of the neutral axis and the bending resistance for the plastic and compact section are given in Table 18.

Table 18 Hogging Bending Resistance of Plastic and Compact Composite Section

(*Clause* B-2.1)

Case	Position of Plastic Neutral Axis	Value of $\overline{y}$	Moment Capacity M_{dh}
(1)	(2)	(3)	(4)
1	In Web	$\overline{y} \le \frac{D}{2} - t_f$	$M_{dh} = M_{ds} + F_{sr}\, Z$
2	In Flange	$\frac{D}{2} \ge \overline{y} > \frac{D}{2} - t_f$	$M_{dh} = F_b \frac{D}{2} + F_{sr}\, d_s - \frac{(F_b - F_{sr})^2}{F_f} \frac{t_f}{4}$

where

$Z = \frac{D}{2} + d_s - \frac{\overline{y}}{2}$

F_f = Axial capacity of a single flange = $\frac{f_y . A_f}{\gamma_{m0}}$

F_b = Axial capacity of steel section = $\frac{f_y . A_s}{\gamma_{m0}}$

F_{sr} = Axial capacity of reinforcing steel = $\frac{f_{yk} . A_{st}}{\gamma_k}$

A_s = area of structural steel cross section

A_f = area of top flange of structural steel cross section

A_{st} = area of reinforcing steel within the effective width of concrete flange

d_s – overall depth of the concrete slab

d_e= Effective depth of the slab

t_f = Average thickness of the top flange of the steel section

t_w = Thickness of the web of the steel section

NOTE — The web shall be classified as being in compression throughout.

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B-2.2 Hogging Bending Resistance for Non-Compact Section

Where elastic analysis is used, creep is considered in the choice of modular ratio m ($= E_s/E_{cm}$). Here, at the section considered, the loading causes hogging bending moment $M_{e(s)}$ in the steel member alone and $[M_{e(c)}]$ in the composite member.

The location of the elastic neutral axis of the composite semi-compact section [*see* Fig. 28(b)] is given as:

$$x_e(A_s + A_{st}) = A_{st}\left(\frac{D}{2} + d_s\right)$$

and the second moment of area of the composite section is:

$$I_{co} = I_s + A_s.x_e{}^2 + A_{st}\left(\frac{D}{2} + d_s - x_e\right)^2$$

Where, I_s is the second moment of area of the steel section alone.

The yield moment is mostly governed by the total stress in the steel bottom flange. The locations of the neutral axis and the moment of resistance for non-compact section is given in Table 19.

The bending moment $[M_{e(s)}]$ causes no stress in the slab reinforcements. In propped construction, the tensile stress in the reinforcement may govern the design. It is given in the following equation:

$$\sigma_{sr} = \frac{\left(\frac{f_y}{\gamma_{m0}} - f_s\right)\left(\frac{D}{2} + d_s - x_e\right)}{\left(\frac{D}{2} + x_e\right)} \leq \frac{f_{yk}}{\gamma_k}$$

Table 19 Hogging Bending Resistance of Non-compact Composite Section
(*Clause* B-2.2)

Location of Neutral Axis	Moment Capacity of Steel Section Alone $[f_s]\,\bar{y}$	Moment Capacity $M_{d(c)}$
(1)	(2)	(3)
$x_e(A_s + A_{st}) = A_{st}\left(\frac{D}{2} + d_s\right)$	$f_s = M_{e(s)}.\frac{D/2}{I_s}$	$M_{d(c)} = M_{e(s)} + \frac{\left(\frac{f_y}{\gamma_{m0}} - f_s\right) I_{co}}{\left(\frac{D}{2} + x_e\right)}$
NOTE — The web shall be classified as being in compression throughout.		

where

A_{st} = cross sectional area of reinforcements within the effective width of the concrete flange;

A_s = cross sectional area of steel beam of a composite section;

d_s = overall depth of the slab;

d_e = effective depth of the slab;

$M_{e(s)}$ = hogging moment in the steel section alone;

f_s = compressive stress in steel flange due to moment $M_{e(s)}$;

I_s = moment of inertia of the steel beam alone;

I_{co} = moment of inertia of the composite section;

t_f = average thickness of the top flange of the steel section; and

t_w = thickness of the web of steel section.

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B-3 FLANGE STRESS REDUCTION FACTOR (R_h) FOR HYBRID SECTIONS

Flange stress reduction factor is applicable for hybrid sections using higher-grade steel flanges where the section is non-compact, or in other words, where the section reaches the plastic moment capacity. In such cases, design limiting stress for both compression and tension shall be modified by the reduction factor R_h and shall be taken as:

$$f_n = R_h \, . \, f_{yf} / \gamma_{m0}$$

where

R_h = $(D.f_{yw}) /(d.f_{yf}) \leq 1.0$;

f_{yf}, f_{yw} = the yield strength of the flange and web respectively; and

D, d = the distance of the extreme compression/tension flange and the corresponding extreme fiber of the web from the neutral axis of the composite section respectively.

For homogeneous grade steel sections, R_h shall be 1.0. The reduction factor shall not apply to compact or plastic sections because the effect of lower strength material in the web is accounted for in calculating the plastic moment as specified in **B-1.1**.

B-4 MOMENT OF RESISTANCE AGAINST LATERAL-TORSIONAL BUCKLING (CONSTRUCTION STAGE)

At the construction stage, before the concrete hardens, the structural steel beam strength may be dictated by the lateral buckling strength. This may be evaluated as per good practice [C-6B(3)].

Effect of lateral-torsional buckling on flexural strength need not be considered if $\lambda_{LT} \leq 0.4$.

B-4.1 Sagging Lateral Buckling Resistance of Structural Steel Beams

The design buckling resistance moment of a laterally unrestrained beam under un-propped condition during the construction stage shall be taken as:

$$M_d = \beta_b Z_p \left(\frac{\chi_{LT} f_y}{\gamma_{m0}}\right)$$

where

$$\chi_{LT} = \frac{1}{[\varphi_{LT}+(\varphi_{LT}^2-\lambda_{LT}^2)^{\frac{1}{2}}]} \leq 1.0 \text{ and}$$

$$\varphi_{LT} = 0.5\left[1 + \alpha_{LT}(\lambda - 0.2) + \lambda_{LT}^2\right] \text{ and}$$

α_{LT} = As per Table 13

The non-dimensional slenderness ratio, λ_{LT}, is given by:

$$\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}} \leq \sqrt{\frac{1.2\, Z_e f_y}{M_{cr}}}$$

$$\lambda_{LT} = \sqrt{\frac{f_y}{f_{crb}}}$$

where

β_b = 1.0 for Plastic and Compact sections;
= Z_e / Z_p for semi-compact sections; and

M_{cr} = the elastic critical moment corresponding to lateral torsional buckling.

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In the case of simply supported prismatic members with symmetric cross-section, the elastic critical moment, M_{cr}, can be determined as:

$$M_{\mathrm{cr}} = \sqrt{\left\{\left(\frac{\pi^2 E I_{\mathrm{y}}}{(L_{\mathrm{LT}})^2}\right)\left[G I_t + \frac{\pi^2 E I_w}{(L_{\mathrm{LT}})^2}\right]\right\}} = \beta_{\mathrm{b}} Z_{\mathrm{p}} f_{\mathrm{crb}}$$

The maximum bending compressive stress corresponding to lateral buckling, $f_{cr.b}$ of non-slender rolled I-sections may be approximately calculated using the following equation:

$$f_{\mathrm{crb}} = \frac{1.1\pi^2 E}{(L_{\mathrm{LT}}/r_{\mathrm{y}})^2}\left[1 + \frac{1}{20}\left(\frac{L_{\mathrm{LT}}/r_{\mathrm{y}}}{h_{\mathrm{f}}/t_{\mathrm{f}}}\right)^2\right]^{0.5}$$

The following simplified conservative equation may be used in the case of prismatic members made of standard rolled I-sections and welded doubly symmetric I-sections for calculating the elastic critical lateral buckling moment.

$$M_{cr} = \frac{\pi^2 E I_y h_f}{2L_{LT}^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5} = \beta_{\mathrm{b}} Z_{\mathrm{p}} f_{\mathrm{cr,b}}$$

where

$f_{cr,b}$ = lateral torsional buckling stress;

I_w = warping constant;

I_t = torsional constant = $\sum b_i t_i^3 / 3$ for open sections;

I_y = moment of inertia about the weak axis;

r_y = radius of gyration of the section about the weak axis];

L_{LT} = effective length for lateral torsional buckling;

h_f = Center to center distance between flanges; and

t_f = thickness of the flange.

M_{cr} for different beam sections, considering loading, support condition, and non-symmetric cross-section, shall be more accurately calculated using the method given in as per good practice [C-6B(3)].

B-4.2 *Hogging Lateral Buckling Resistance of Composite Beams*

The effective unsupported length, L_{LT}, in case of bare steel section may be taken as recommended in as per good practice [C-6B(3)]. When a slab connects in the hogging moment region the top flange of two or more beams running parallel, then the effect of U-Frame action in restraining the lateral buckling of bottom flange under compression (Fig. 29) may be considered as given below:

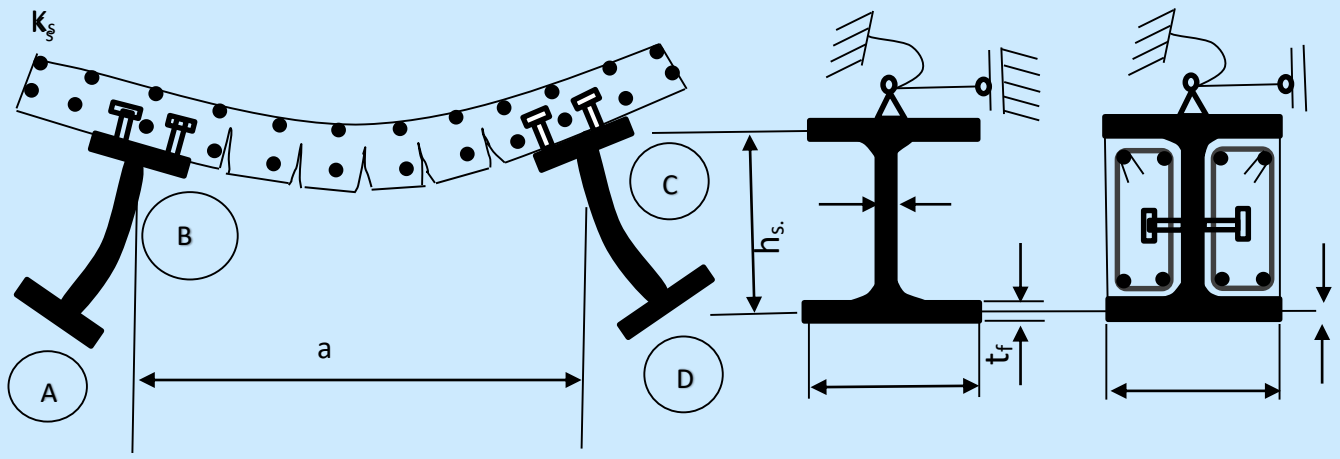

FIG. 29 INVERTED U-FRAME ACTION TO RESTRAIN LATERAL BUCKLING OF BOTTOM FLANGE

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B-4.2.1 The rotational spring restraint stiffness of the concrete slab, k_s, per unit length of steel beam is given by

$$k_s = k_1 k_2/(k_1 + k_2)$$

where

k_1 = Flexural stiffness of the cracked concrete or composite slab transverse to the spanning direction of steel beam $= \alpha (EI)_s/a$

α = 2 for an edge beam,

= 3 for inner beam and

= 4 for inner beam with four or more such beams running parallel and supporting the continuous slab;

a = span length of the slab transverse to the steel beams

$(EI)_s$ = cracked flexural stiffness of the concrete or composite slab per unit width parallel to the steel beam, taken as the lower of the sagging bending slab stiffness of the slab in between the steel beams and hogging bending stiffness of the slab at the supporting steel section;

k_2 = flexural stiffness of the steel web per unit length along the beam,

$= E_s t_w^3/[4(1- \nu_s^2)h_s)]$ in un-encased steel section

$= E_s t_w b_c^2/[16 h_s (1+ 4 m t_w/b_c)]$ in encased steel section as per **8.3.8**

E_s = modulus of elasticity of structural steel;

E_s = Poisson's ratio of structural steel;

t_w = thickness of the steel section web;

b_c = width of concrete encasement (Fig. 27);

h_s = distance between centroids of top and bottom flanges

m = long term modular ratio of encasing concrete as per good practice [C-6B(2)].

B-4.2.2 The torsional rigidity of the steel section may be taken as $G_s I_{st}$, where Gs is the shear modulus of steel, I_{st}, is the St. Venant's torsion constant of the steel section. The torsional rigidity stiffness contribution of encasing concrete may be taken as being equal to $G_c I_{ct}$ /10, where G_c, may be taken as equal to 0.3 Es/m and I_{ct} is the St. Venant's torsion constant of encasing concrete of rectangular shape and m is the modular ratio for long term effects.

B-4.2.3 The elastic hogging critical buckling moment of the composite beam at an internal support of a continuous beam is given by:

$$M_{cr} = (k_c C_4/L) [(G_s I_{st} + k_s L^2/\pi^2) E_s I_{ybf}]^2$$

where

I_{ybf} = moment of inertia of the bottom flange about the minor axis of the steel section;

I_{zco} = second moment of the equivalent area, A_{co}, of the cracked composite beam section for major axis bending;

K_c = composite beam section property = $(h_s I_{zc}/I_{ys})/[(h_s^2/4 + (I_{ys}+ I_{zs})/A_s)/e + h_s]$;

e = $A_{co} I_{zs}/[A_s Z_c (A_{co} - A_s)]$;

h_s = distance between the centers of the flanges of the steel section;

I_{ys}, I_{zs} = second moment of area of steel section about the minor and major axis, respectively;

Z_c = distance between the centroid of steel section and the mid depth of concrete slab; and

C_4 = constant depending upon the bending stress distribution over the span length as given in Table 20 and Table 21.

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Table 20 Linear BMD over the Span

(*Clause* B-4.2.3)

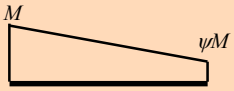
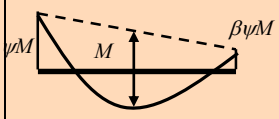
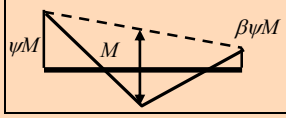
BMD	C_4								
	Ψ = - 1	- 0.75	- 0.5	- 0.25	0.0	0.25	0.5	0.75	1.0
	18.1	16.3	14.6	12.8	11.1	9.5	8.2	7.1	6.2

Table 21 Non-Linear BMD over the Span

(*Clause* B-4.2.3 *and* D-4.1)

BMD	β	C_4								
		Ψ = 0.5	0.75	1.0	1.25	1.5	1.75	2.0	2.25	2.5
	0.00	41.5	30.2	24.5	21.1	19.0	17.5	16.5	15.7	15.2
	0.50	33.9	22.7	17.3	14.1	13.0	12.0	11.4	10.9	10.6
	0.75	28.2	18.0	13.7	11.7	10.6	10.0	9.5	9.1	8.9
	1.00	21.9	13.9	11.0	9.6	8.8	8.3	8.0	7.8	7.6
	0.00	28.4	21.8	18.6	16.7	15.6	14.8	14.2	13.8	13.5
	1.00	12.7	9.8	8.6	8.0	7.7	7.4	7.2	7.1	7.0

B-5 MOMENT OF RESISTANCE FOR FILLER BEAM DECKS

Filler Beam decks most likely to be designed for structures with wide column-free zones having higher imposed loads. The stress distribution diagram for a standard filler beam decks is as shown in Fig. 30.

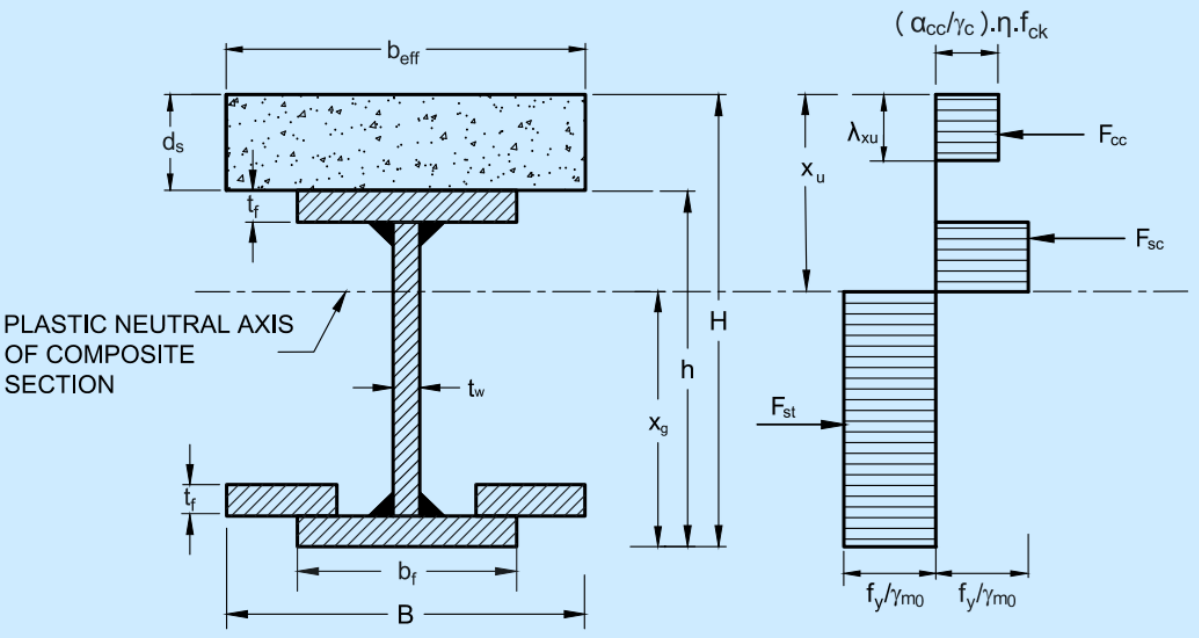


FIG. 30 STRESS DIAGRAM FOR FILLER BEAM

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For equilibrium, $F_{st} = F_{sc} + F_{cc}$

where

F_{st} = Tensile force in the steel section below the neutral axis;

F_{sc} = Compressive force the steel section above the neutral axis; and

F_{cc} = Compressive force in the concrete above the neutral axis.

The depth of the neutral axis is given as,

$$x_u = H - x_g$$

where

$$x_g = \frac{\frac{\alpha_{cc}}{\gamma_c}\,\eta\, f_{ck}\,[BH - b_f t_f - t_w(h - t_f)] + t_w h \frac{f_y}{\gamma_{m0}}}{\frac{\alpha_{cc}}{\gamma_c}\eta f_{ck}(B - t_w) + 2t_w \frac{f_y}{\gamma_{m0}}}$$

Bending resistance,

$$M_p = F_{sc} \,.\, X_{sc} + F_{cc} \,.\, X_{cc} + F_{st} \,.\, X_{st}$$

where

X_{sc}, X_{cc} *and* X_{st} are respectively the distance between the neutral axis of the composite beam and the individual centre of gravities of the corresponding forces].

B = center-to-center distance between two filler beams;

b_{eff} = effective width of concrete for one filler beam;

b_f = width of the top flange of the steel section;

t_w = thickness of the web of steel section;

H = distance between top of concrete and bottom of bottom flange of steel beam;

h = total depth of steel girder;

x_g = distance of neutral axis from bottom of bottom flange of steel beam;

A_{st} = cross sectional area of reinforcements within the effective width of the concrete flange;

A_s = cross sectional area of steel beam of a composite section;

d_s = overall depth of the slab;

d_e = effective depth of the slab; and

M_p = ultimate bending moment.

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ANNEX C

(*Clauses* 12.4.3.1 *and* 12.5.2)

COMPOSITE SLABS

C-1 GENERAL

Composite slabs consist of metal decking spanning between the supports and acting integrally with the concrete after the concrete hardens (Fig. 31). The steel decking performs several roles as given below:

a) It supports loads during construction and acts as a working platform;

b) It develops adequate composite action with concrete to resist the imposed loading;

c) It transfers in-plane loading by diaphragm action to vertical bracing or shear walls;

d) It may stabilize the compression flanges of the beams against lateral buckling until the concrete hardens;

e) It reduces the volume of concrete in the tension zone of the slab; and

f) It distributes shrinkage strains, thus preventing serious cracking of concrete.

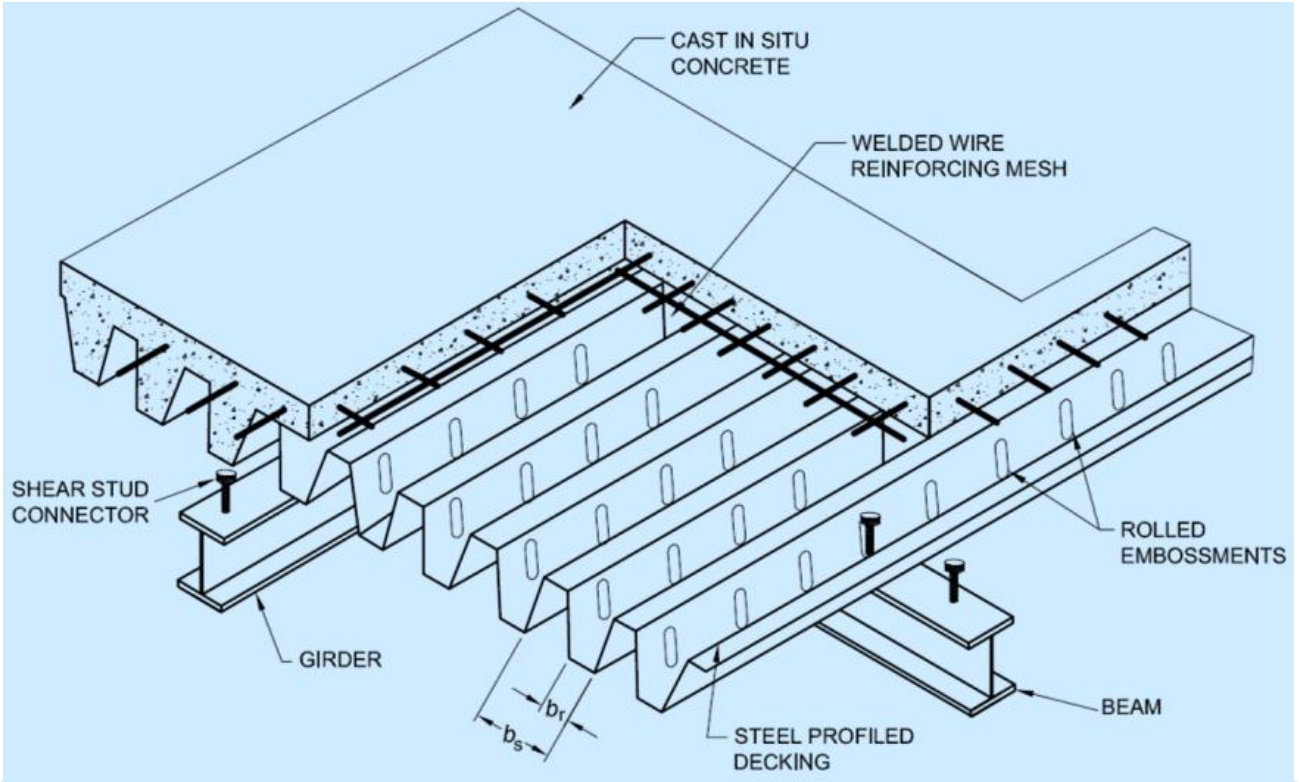

FIG. 31 COMPOSITE SLAB

C-2 STRUCTURAL ELEMENTS

C-2.1 Composite floors with profiled decking consist of the following structural elements along with in-situ concrete and steel beams:

a) Profiled decking;

b) Shear connectors; and

c) Reinforcement for shrinkage and temperature stresses.

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Connections between the structural steel beam and decking elements are generally designed as ‘simple’, that is, not moment-resisting. Stud shear connectors are welded through the sheeting onto the top flange of the beam to facilitate the shear transfer between concrete and steel decking, serving as end anchorage, as well as the shear transfer between the slab and steel beam for mobilising composite action along the beam span.

C-2.2 Profile Decking

The steel deck is normally rolled into the desired profile from 0.9 mm to 1.5 mm galvanized coil sheets. It is profiled such that the profile heights are usually in the range of 38 mm to 75 mm, and the pitch of corrugations is between 150 mm and 350 mm. Generally, composite slab spans of the order of 2.5 m to 3.5 m between the beams are chosen, and the beams are designed to span between 6 m to 12 m. There are two well-known generic types of profiles as given below (Fig. 32):

a) Dovetail profile with flange indentations; and

b) Trapezoidal profile with web indentations.

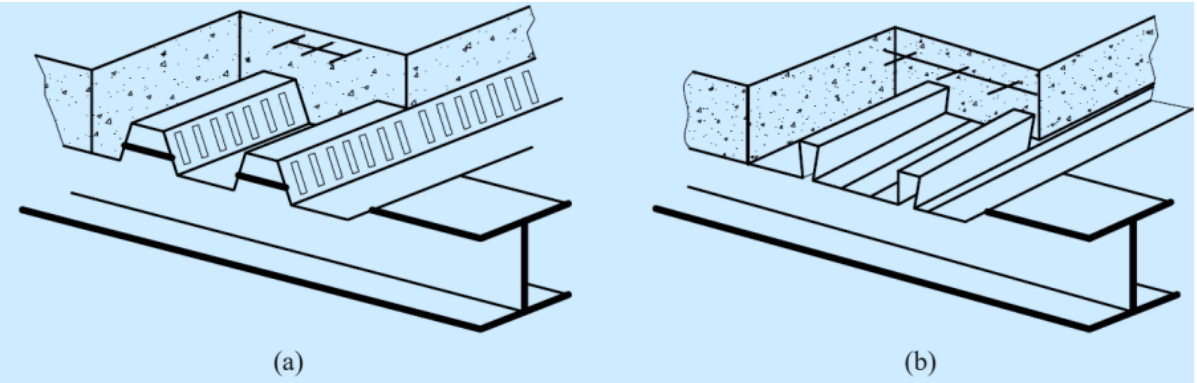

FIG. 32 COMPOSITE SLAB DECKING PROFILE

C-2.3 Shear Connectors

To enable composite action to be assumed between the profiled steel sheet and the concrete, the longitudinal shear force be transferred by the sheet by the above following form of connection (Fig. 33):

a) Mechanical interlock through the provision of indentations or embossments rolled into the profile;

b) Frictional interlock of re-entrant profile;

c) Through-deck welded stud connectors or any other local connection between the steel and the concrete; and

d) Deformation of the ends of the ribs at the ends of the sheeting.

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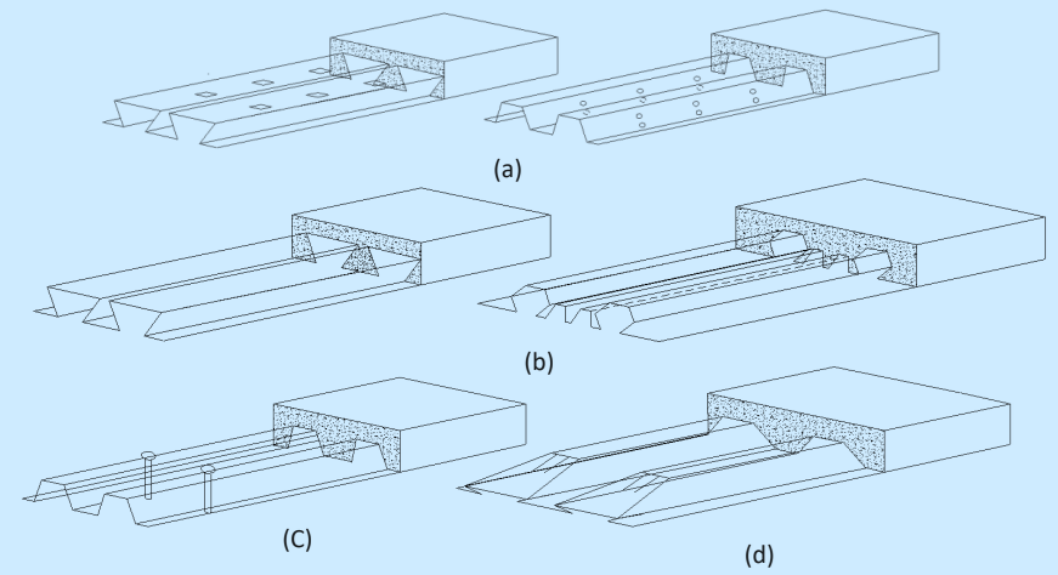

FIG. 33 TYPICAL FORMS OF SHEAR CONNECTORS IN COMPOSITE SLAB

C-2.4 Reinforcement for Shrinkage and Temperature Stresses

In buildings, the temperature difference in the slabs is negligible; thus, there is no need to provide reinforcement to account for temperature stresses. The effect of shrinkage is to be considered, and the total shrinkage strain for design may be taken as 0.003 in the absence of test data.

C-3 PONDING OF PROFILED DECKING

Fig. 34 shows ponding of the profiled deck. In unpropped construction, the profiled decking acting as a formwork to support wet concrete and construction imposed load. Due to this, the deck sheeting may deflect and cause an increase in the thickness of concrete at or closer to mid-span. This is referred to as 'Ponding.' Ponding increases the volume of concrete consumed as well as dead load. These effects should be accounted for, particularly in long spans.

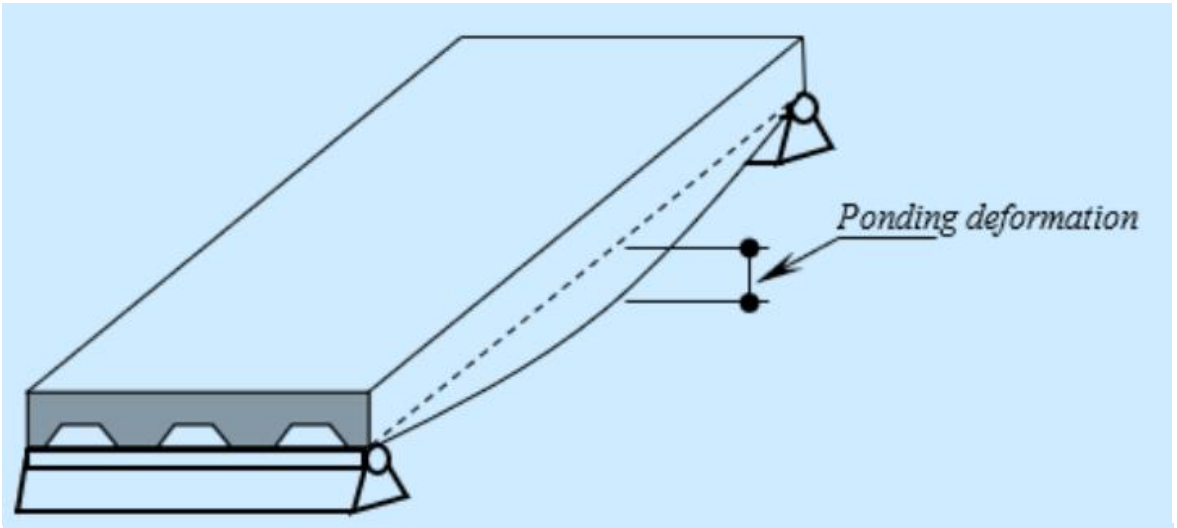

FIG. 34 PONDING IN PROFILE DECKING DUE TO WEIGHT OF CONCRETE SLAB

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C-4 BENDING RESISTANCE OF COMPOSITE SLAB WITH FULL SHEAR CONNECTION

NOTE — In C-4, instead of the usual F_{cc}, the concrete compressive force is denoted as F_{ccf} to represent the concrete compressive force with a full shear connection.

C-4.1 Sagging Bending Resistance [Neutral Axis is above the Steel Decking (Fig. 35)]

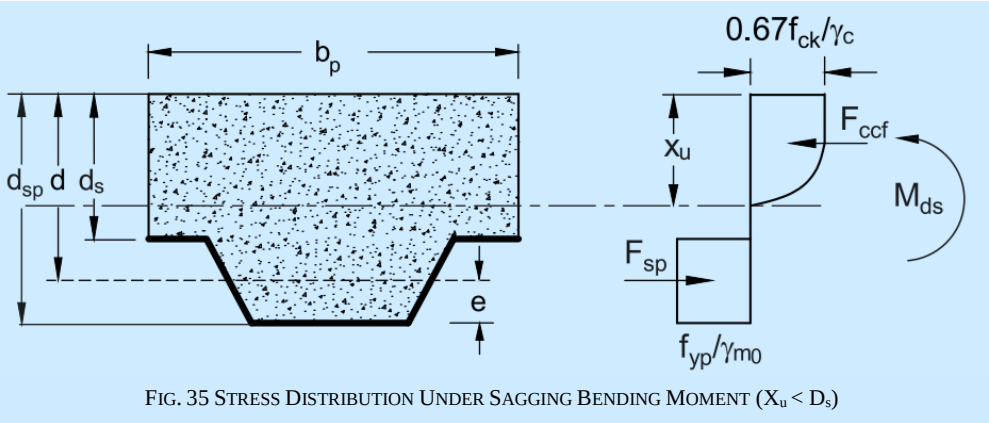

FIG. 35 STRESS DISTRIBUTION UNDER SAGGING BENDING MOMENT ($X_u < D_s$)

The design stress distribution is shown in Fig. 35. This is valid when $x_u \leq d_s$, that is, when the neutral axis lies above steel decking. Centroid of concrete force lies at $0.42x_u$ from top concrete surface. The equations for the design resistance are given in Table 22.

C-4.2 Sagging Bending Resistance [Neutral Axis is within the Steel Decking (Fig. 36)]

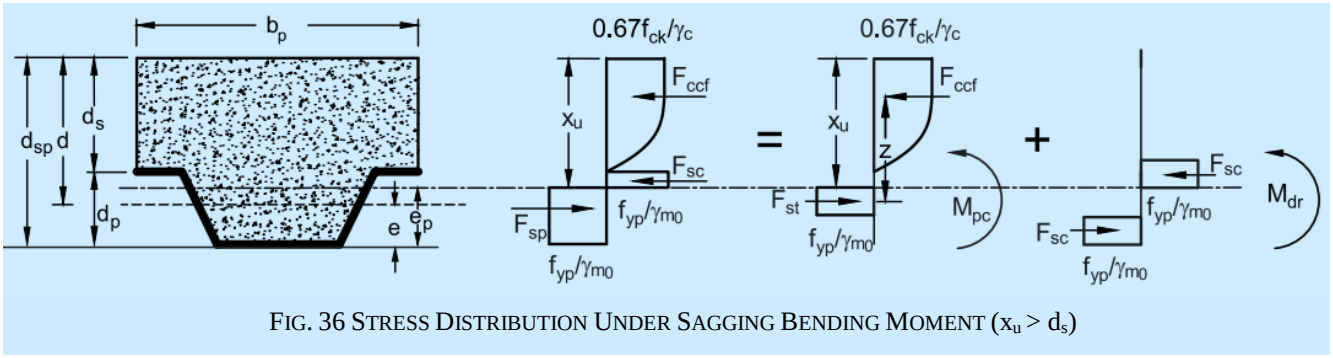

FIG. 36 STRESS DISTRIBUTION UNDER SAGGING BENDING MOMENT ($x_u > d_s$)

The design stress distribution is shown in Fig. 36. This is valid when $x_u > d_s$, that is. when the plastic neutral axis lies in the steel decking. The depth of the parabolic compressive stress block in concrete is between the top of the concrete slab to the top fibre of the deck sheet as shown in Fig. 36. Also, e_p is the distance between the plastic neutral axis of the composite slab to the extreme fibre of the deck sheet in tension and e is the distance between the centroidal axis of the profiled steel sheeting to the extreme fibre of the deck sheet in tension. The centroid of concrete force F_{ccf} lies at $0.42d_s$ from top of the concrete surface. The equations for the design resistance are given in Table 22. The value M_{dp} used in Table 22 is plastic moment capacity of the deck sheet which may be provided by the manufacturer or calculated independently.

C-4.3 Hogging Bending Resistance of Composite Deck

The contribution of steel decking is neglected. The design stress distribution is shown in Fig. 37. This is valid when x_u is in the range of $(d_s < x_u < d_{cp})$, that is., when the neutral axis lies in the steel decking. The centroid of concrete force lies at $0.42x_u$ from the bottom concrete surface b_0 the concrete width at the neutral axis is to be evaluated by trial and error. The equations for the design resistance are given in Table 22.

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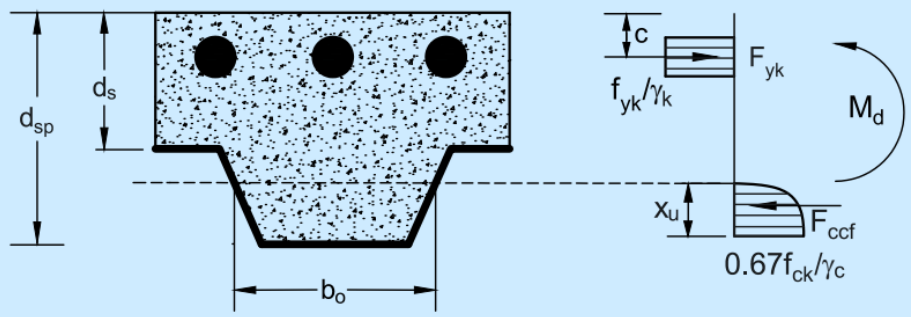

FIG. 37 STRESS DISTRIBUTION UNDER HOGGING BENDING MOMENT ($d_s < x_u < d_{sp}$)

Table 22 Design Moment Capacity, M_{ds}, of Composite Slabs with Full Shear Connection

(*Clauses* C-4.1, C-4.2 *and* C-4.3)

Case	Position of Plastic Neutral Axis	Value of x_u	Design Moment Capacity, M_{ds}
(1)	(2)	(3)	(4)
1	Sagging moment, NA above sheeting material (Fig. 35)	$x_u = \frac{A_p f_{yp}/\gamma_{mo}}{0.36 f_{ck} b_p} \le d_s$	$M_{ds} = \frac{A_p f_{yp}}{\gamma_{mo}}(d - 0.42x_u)$
2	Sagging moment NA within the steel decking (Fig. 36)	$x_u > d_s$ A simplified stress block is shown in Fig. 36. Using which M_{ds} is calculated without the need for x_u	$M_{ds} = F_{ccf} z + M_{dr}$ $M_{dr} = 1.25\, M_{dp}\left[1 - \left(\frac{F_{ccf}}{A_p f_{yp}/\gamma_{mo}}\right)\right] < M_{dp}$ $z = d_{cp} - 0.42 d_s - e_p + \frac{F_{ccf}}{F_{sp}}(e_p - e)$ $F_{ccf} = 0.36 f_{ck} b_p d_s$ $F_{sp} = \frac{A_p f_{yp}}{\gamma_{mo}}$
3	Hogging moment (Fig. 37)	$x_u = \frac{\frac{A_{st} f_{yk}}{\gamma_k}}{0.36 f_{ck} b_o}$	$M_d = \frac{A_{st} f_{yk}}{\gamma_k} \times (d_{sp} - c - 0.42x_u)$

Notations:

A_{st} – area of the reinforcing bar within one wave length b_s of decking sheet

d – distance between the C.G axis of the profiled steel sheeting and the extreme fibre of the composite slab in compression $(d_{sp} - e)$

e – distance between the C.G axis of the profiled steel sheeting and the extreme fibre of the composite slab in tension

e_p – distance between the plastic neutral axis and the extreme tension fibre of the composite slab.

b_p – distance between the centres of adjacent ribs or one wave length of a profile deck

A_p – effective cross-sectional area of profile decking sheeting

M_{dr} – design bending resistance of profile decking sheeting

M_{ps} _ Plastic moment of resistance of the profile decking sheeting

f_{ck} – characteristic compressive strength of concrete

f_{yp} – yield strength of profile decking sheeting

d_{sp} – total depth of the composite slab $(d_{sp} = d_s + d_p)$

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C-5 BENDING RESISTANCE OF COMPOSITE SLAB WITH PARTIAL SHEAR CONNECTION

NOTE — In C-5, instead of the usual F_{cc}, the concrete compressive force is denoted as F_{ccp} to represent the concrete compressive force with a partial shear connection.

C-5.1 The stress distribution in slabs with partial shear connection is similar to Fig. 37. However, the compressive force in the concrete above the neutral axis ($F_{ccp} < F_{ccf}$) is controlled by the interface shear resistance due to the partial shear connection. The interface shear resistance is obtained from tests. The details of tests to obtain the shear strength between the interface of decking and concrete is given in C-5.2.

C-5.2 Shear Resistance of Composite Slab

The longitudinal shear behaviour may be considered as ductile if the failure load exceeds the load causing a recorded end slip of 0.1 mm by more than 10 percent. If the maximum load is reached at a mid-span deflection exceeding $L/50$, the failure load shall be taken as the load at the mid-span deflection of $L/50$. Otherwise, the behaviour is classified as brittle.

The shear resistance due to the chemical bond shall not be considered. This clause is applicable to the composite slabs with the mechanical or frictional interlock. Literature gives two main methods for determining the shear transfer for partial shear connection from test results. They are (i) the m-k method and (ii) the partial shear connection method. The m-k method is valid for both ductile and brittle failure cases but disregards the effect of overhang beyond the support, whereas the partial shear connection method is valid only for ductile behaviour, but accounts for the effect of the support reaction and the overhang beyond the support.

C-5.2.1 *m-k Method for Composite Slabs without End Anchorage*

The m-k method is based on establishing the gradient and intercept of an assumed linear relationship between two parameters of the slab, obtained from two groups of composite slab tests. The evaluation of m-k values is illustrated in Fig. 38.

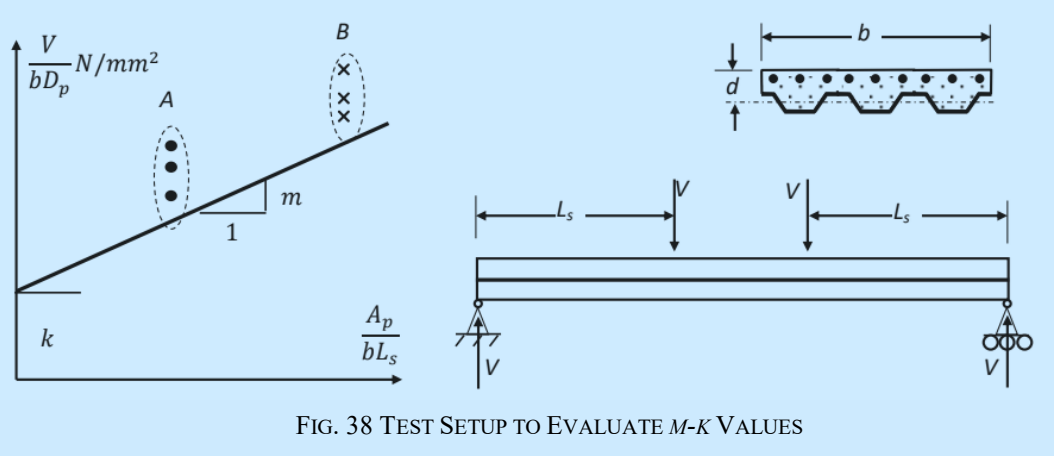

FIG. 38 TEST SETUP TO EVALUATE *M-K* VALUES

For cases when the longitudinal shear behaviour may be considered ductile, V is taken as the value of the support reaction at the failure load. However, if the behavior is brittle, the value should be reduced using a factor 0.8. By plotting the results from the composite slab tests in terms of the vertical shear parameter [$V/(b\ D_p)$] against the shear bond parameter [$A_p/(b\ L_s)$] for two groups of data corresponding to the long specimens (Group-A) and the short specimens (Group-B). The relationship between vertical shear and shear bond capacity is approximated by constructing a straight line through the two groups of data (Fig. 38). The effect of any overhang in the test specimen is neglected in the m-k method, unlike in the partial shear connection approach.

From all the values of V, 5 percent fractile of the characteristic shear strength linear regression line should be calculated to define the characteristic m and k values (Fig. 38). The minimum value of each group is further reduced by 10 percent for design consideration if two groups of three tests are used and the deviation of any individual test result in a group does not differ by more than 10 percent from the mean.

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If the m-k method is used, it should be shown that the maximum design vertical shear V for a width of slab 'b' does not exceed the design shear resistance V_{ld} specified in Table 23.

C-5.2.2 *Partial Interaction Method*

The degree of shear connection η is defined as,

$$\eta = \frac{F_{ccp}}{F_{ccf}}$$

where

F_{ccp} = compressive force in the concrete as governed by partial shear transfer, as obtained from a standard tests; and

F_{ccf} = compressive force in the concrete for full shear connection as given in **C-4**.

The variation in bending resistance with the degree of shear connection is shown in Fig. 39. In case $\eta = 0$ composite action between the steel sheet and the concrete does not exist, and it is assumed that the bending resistance is provided by the bending resistance, M_{dp}, of profiled steel sheet alone. For the case $\eta = 1$, in the full shear connection, the full tensile resistance of the sheet is developed, or the full compressive resistance of concrete above the ribs of the sheet is attained. For intermediate cases such that $0 < \eta < 1$, the partial shear connection exists. Typical behaviour of open trough profile steel sheeting with partial shear connection is illustrated in Fig. 39.

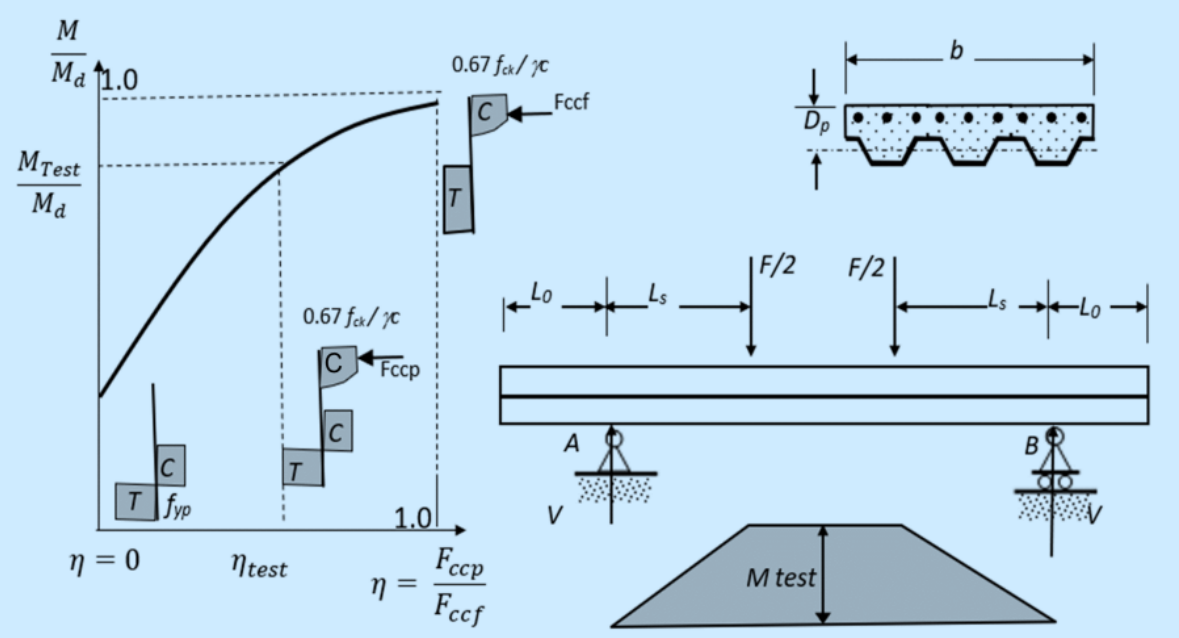

FIG. 39 TEST TO DETERMINATION OF THE DEGREE OF THE PARTIAL SHEAR CONNECTION

For a given bending resistance, the degree of shear connection provided in the test η_{test} can be assessed from the points on the partial shear interaction curve in Fig. 39. By neglecting the effect of the support reaction, the longitudinal shear strength τ_u can be obtained from

$$\tau_u = \frac{\eta_{test} F_{ccf}}{b (L_s + L_o)}$$

where

F_{ccf} = Compressive normal force in the concrete flange with full shear connection;

b = Width of the slab;

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L_s = Shear span; and

L_o = Length of the overhang (Fig. 39).

If the additional longitudinal shear, the additional resistance caused by the friction due to the support reaction is taken into account, the equation becomes

$$\tau_u = \frac{\eta_{test} F_{ccf} - \mu V}{b (L_s + L_o)}$$

where

μ = frictional coefficient taken as 0.5; and

V = support reaction under the test load.

The characteristic shear strength $\tau_{u.fk}$ should be calculated from the test values as the minimum value of stress below which not more than a specified percentage (usually 5 percent) of the corresponding stresses of samples tested are expected to occur. This is divided by the partial safety factor governed by the ultimate state to obtain the design value, $\tau_{u.d}$.

The determination of the design moment resistance of the composite slab with the partial shear connection is similar to the case (Fig. 39). The design bending resistance of the partial shear connection slab is evaluated using equations given in Table 23.

Table 23 Design Moment Capacity, M_{dp}, of Composite Slabs with Partial Shear Connection

(*Clauses* C-5.2.1, C-5.2.2 *and* D-2)

Case	Position of Plastic Neutral Axis	Methods/ Value of x_u	Design Moment Capacity, M_{dp}
(1)	(2)	(3)	(4)
1	Sagging moment (Fig. 38)	*m-k* method	$M_{dp} = V_{lp} L_s$ $V_{lp} = \frac{bd_p}{\gamma_{m1}}\left(\frac{mA_p}{bL_s} + k\right)$
2	Sagging moment, NA above profile decking sheet (Fig. 39)	Partial shear connection method $x_u = \frac{F_{ccp}}{0.36 f_{ck} b_p} \leq d_s$	$M_{dp} = \frac{A_p f_{yp}}{\gamma_{mo}}(d - 0.42x_u)$ $F_{ccp} = \tau_{ud} b_p L_x \leq F_{ccf}$
3	Sagging moment, NA within the profile decking sheet (Fig. 39)	Partial shear connection method A simplified stress block is shown in Fig. 39. Using which M_{dp} is calculated with $x_u = \frac{F_{ccp}}{0.36 f_{ck} b_p}$	$M_{dp} = F_{ccp} z + M_{pr}$ $F_{ccp} = \tau_{ud} b_p L_x \leq F_{ccf}$ $z = d_{cp} - 0.42x_u - e_p + \frac{F_{ccp}}{A_p f_{yp}/\gamma_{mo}}(e_p - e)$ $M_{dr} = 1.25\, M_{dp}\left[1 - \left(\frac{F_{ccp}}{A_p f_{yp}/\gamma_{mo}}\right)\right] < M_{dp}$
Where L_x is the distance of the cross section being considered to the nearest support.			

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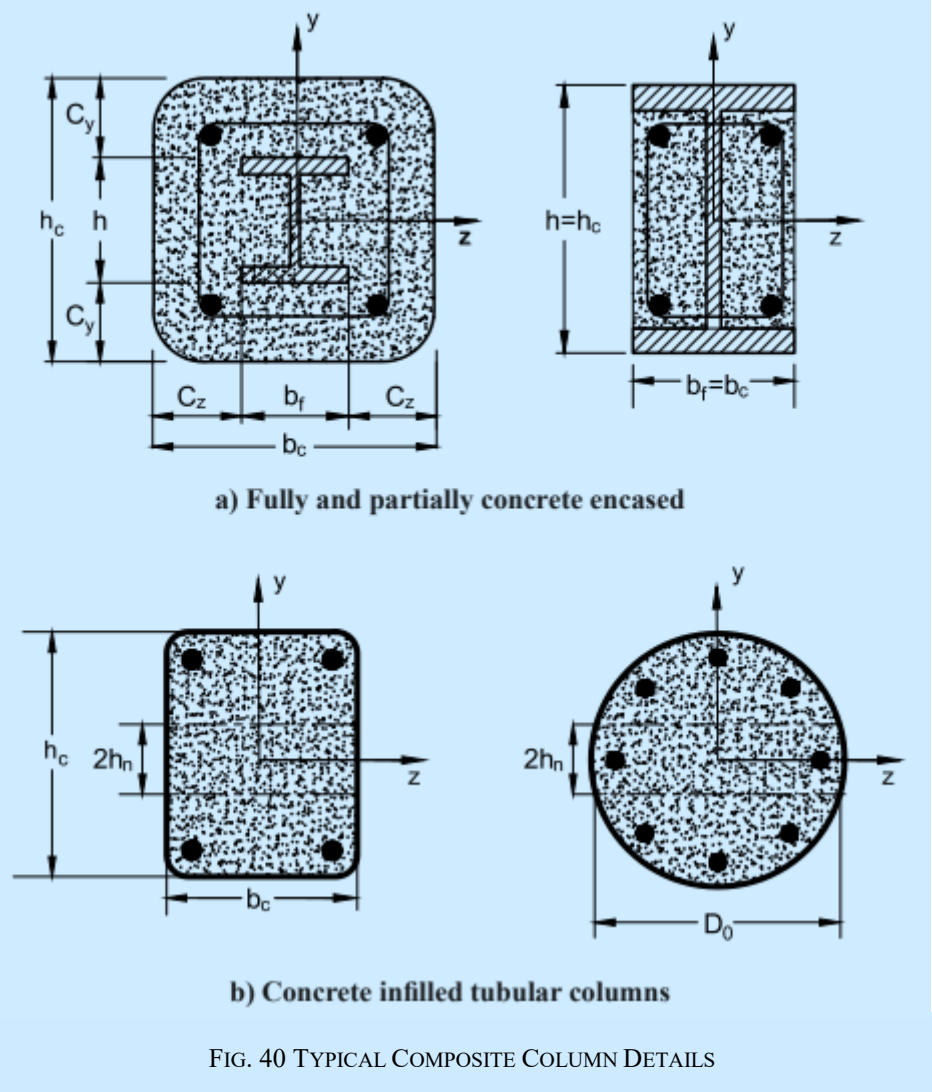
ANNEX D

(*Clause* 13.8.1)

PLASTIC DESIGN RESISTANCE OF SYMMETRIC COMPOSITE COLUMNS

D-1 GENERAL

This section deals with the calculation of the design resistance of the doubly symmetric composite columns. The resistance equations are derived based on the rectangular stress block in concrete. The general dimensional view of the fully or partially concrete-encased single I-section is as given in Fig. 40 (a). The general dimensional view of the concrete infilled tubular composite column section is given in Fig. 40 (b).



a) Fully and partially concrete encased

b) Concrete infilled tubular columns

FIG. 40 TYPICAL COMPOSITE COLUMN DETAILS

The following are the notations used in this section, in addition to the dimensions shown in Fig. 40.

A_c = cross sectional area of concrete;

A_s = cross sectional area of structural steel;

A_{st} = cross sectional area of total steel reinforcement;

A_{stc} = cross sectional area of steel reinforcement in the compression zone;

f_{ck} = characteristic compressive cube strength of concrete;

f_y = minimum guaranteed yield strength of structural steel section;

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f_{yk} = minimum Guaranteed yield strength of steel reinforcement;

M_d = design bending resistance under only bending moment;

M_n = ultimate bending resistance under only bending moment;

P_d = design compressive resistance under compression only;

P_n = ultimate compressive resistance under compression only;

M'_d = design bending resistance under combined bending and compression;

M'_n = ultimate bending resistance under combined bending and compression;

P'_d = design compressive resistance under combined bending and compression;

P'_n = ultimate compressive resistance under combined bending and compression;

r = inner radius of the rounded corners in RHS;

γ_c = partial safety factor for the compressive strength of concrete = 1.5;

γ_{m0} = partial safety factor for the yield strength of structural steel = 1.1; and

γ_k = partial safety factor for the yield strength of steel reinforcement = 1.15.

D-2 DESIGN COMPRESSIVE RESISTANCE OF SHORT COMPOSITE COLUMN (Point A)

The stresses on different components of the composite columns at the ultimate stage under only axial force are as shown in Fig. 41. The ultimate design compressive resistance, P_n, of the plastic, compact and semi-compact sections are given by:

$$P_n = A_s f_y + A_{st} f_{yk} + \alpha_c A_c (\alpha_{cc} f_{ck})$$

where

α_c = 0.85, for partially or fully encased column; and

α_c = 1.0, for infilled columns.

α_{cc} = 0.67

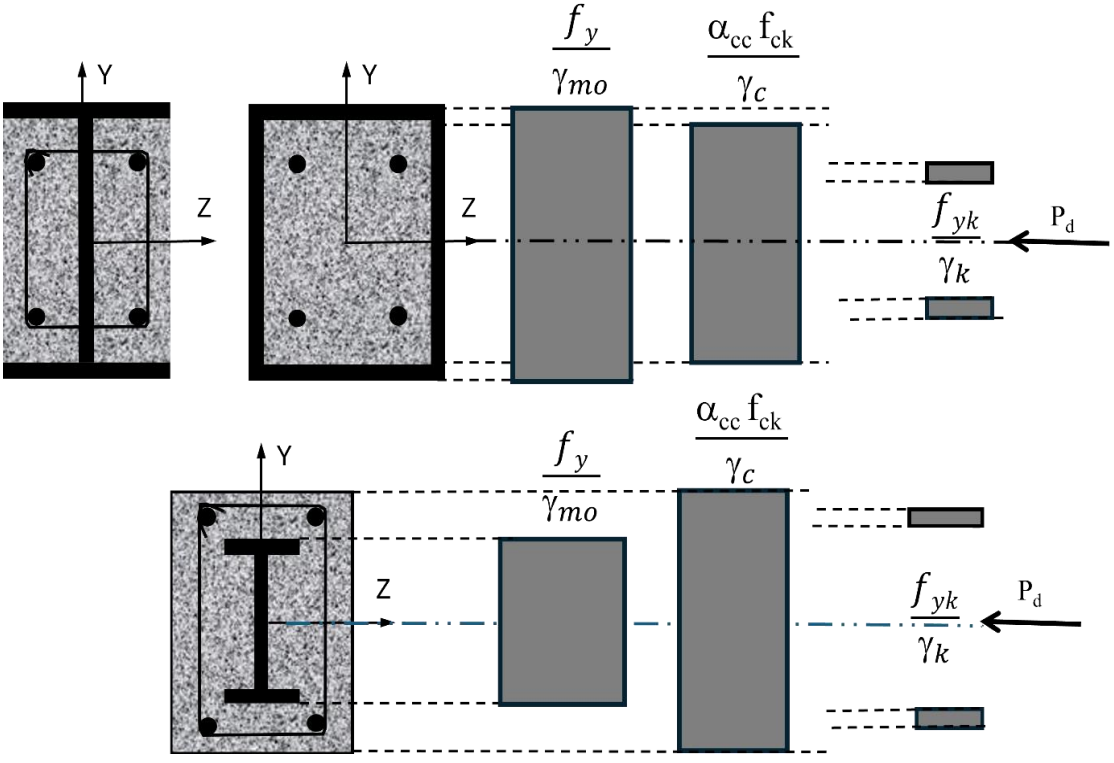

FIG. 41 STRESS DISTRIBUTION AT ULTIMATE STAGE UNDER ONLY AXIAL FORCE

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The axial design resistances of different types of the composite column are given in Table 24. For concrete filled circular tubular sections in Table 24, the increase in concrete strength due to confinement may be taken, provided $\lambda \leq 0.5$ calculated using **13.5.1** and e/d <0.1 where e is the eccentricity of the loading, given by the ratio of the design moment to design axial load and d is the external diameter of the circular column. If anyone of the conditions is not satisfied then $\eta_1=0$ and $\eta_2=1$.

To account for minimum eccentricity and grade of reinforcing steel, the design axial capacity can be estimated using the following expression,

$$P_d = A_s \frac{f_y}{\gamma_{m0}} + \alpha_k A_{st} \frac{f_{yk}}{\gamma_k} + 0.6 \alpha_c A_c \frac{f_{ck}}{\gamma_c}$$

α_k values can be taken as

Grade of Reinforcing Steel	Value of α_k
Fe250	0.90
Fe415	0.82
Fe500	0.77

Table 24 Design Compressive Resistance of Short Composite Compression Members

(*Clause* D-2)

Type of Section	Design Compressive Resistance
(1)	(2)
Partially or fully encased composite section	$P_d = A_s \frac{f_y}{\gamma_{m0}} + A_{st} \frac{f_{yk}}{\gamma_k} + 0.57 A_c \frac{f_{ck}}{\gamma_c}$
Concrete filled rectangular tubular section	$P_d = A_s \frac{f_y}{\gamma_{m0}} + A_{st} \frac{f_{yk}}{\gamma_k} + 0.67 A_c \frac{f_{ck}}{\gamma_c}$
Concrete filled circular tubular section	$P_d = A_s \eta_2 \frac{f_y}{\gamma_{m0}} + A_{st} \frac{f_{yk}}{\gamma_k} + A_c \frac{(\alpha_{cc} f_{ck})}{\gamma_c} \left[1 + \eta_1 \frac{t}{d} \frac{f_y}{(\alpha_{cc} f_{ck})}\right]$ $\eta_1 = \eta_{10} \left[1 - \frac{10e}{d}\right]$ $\eta_2 = \eta_{20} + (1 - \eta_{20}) \frac{10e}{d}$ $\eta_{10} = 4.9 - 18.5 \lambda + 17 \lambda^2 \geq 0.0$ $\eta_{20} = 0.25(3 + 2\lambda) \leq 1.0$

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D-3 DESIGN BENDING RESISTANCE OF COMPOSITE COLUMN

The stresses on the different components of the composite columns at the ultimate stage under only the bending moment about the z-z axis are as shown in Fig. 42.

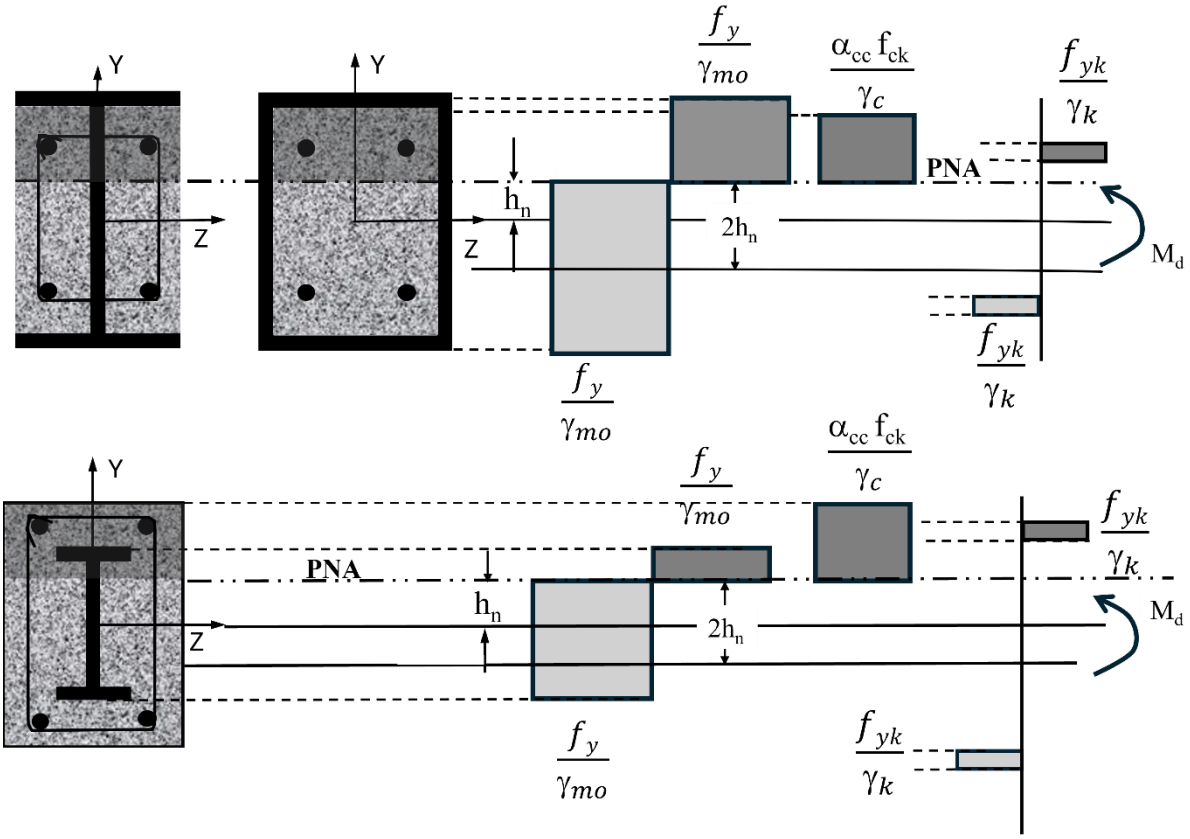

FIG. 42 STRESS DISTRIBUTION AT ULTIMATE STAGE UNDER ONLY BENDING MOMENT

The design bending resistance is given by:

$$M_d = (Z_{ps} - Z_{psn})\, f_y / \gamma_{m0} + (Z_{pr} - Z_{prn})\, f_{yk} / \gamma_k + 0.5\, \alpha_c\, (Z_{pc} - Z_{pcn})\, (\alpha_{cc} f_{ck}) / \gamma_c$$

where

Z_{ps}, Z_{pr}, and Z_{pc} = Plastic section moduli of the steel section, reinforcement and concrete about their own centroidal axis, respectively (the concrete in tension region is assumed to be uncracked for this calculation).

Z_{psn},, Z_{prn} and Z_{pcn} = Plastic section moduli of the steel section, reinforcement and concrete, all within the depth h_n on both sides of the neutral axis of the gross section, about the neutral axis of gross section, respectively. (Table 25 gives equations for evaluating the value of h_n)

α_c = the strength coefficient of concrete,

= 0.85 for fully and partially encased composite sections,

= 1.0 for concrete filled hollow sections and

= 0.89 for fully encased or concrete filled hollow sections with spiral ties

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Table 25 Location of Neutral Axis in Composite Column at Ultimate Moment

(*Clause* D-3)

Section	Case	Depth, h_n
(1)	(2)	(3)
$f_{cd} = \frac{(\alpha_{cc} f_{ck})}{\gamma_c}$, A_{sn} = Area of structural steel within $2h_n$ distance A_{stn} = Area of reinforcing steel within $2h_n$ distance		
a) *Major axis Bending*		
Concrete encased sections	$h_n \le [h/2 - t_f]$	$\frac{A_c a_c f_{cd} - A_{stn}\left(\frac{2f_{yk}}{\gamma_k} - a_c f_{cd}\right)}{2b_c a_c f_{cd} + 2 t_w\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right)}$
	$[h/2 - t_f] \le h_n \le h/2$	$\frac{a_c A_c f_{cd} - A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right) + (b - t_w)(h - 2t_f)\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right)}{2 b_c a_c f_{cd} + 2 b\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right)}$
	$h/2 \le h_n \le h_c/2$	$\frac{A_c a_c f_{cd} - A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right) - A_{sn}\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right)}{2b_c a_c f_{cd}}$

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Table 25 (*Continued*)

Section	Case	Depth, h_n
(1)	(2)	(3)
Concrete filled sections	*RHS*	$\frac{A_c a_c f_{cd} - A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right)}{2\,b_c\,a_c f_{cd} + 4\,t\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right)}$
	CHS	In RHS equation substitute D for b_c as an approximation
b) *Minor axis Bending*		
Concrete encased sections	$h_n \leq t_w/2$	$\frac{A_c a_c f_{cd} - A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right)}{2\,h_c a_c f_{cd} + 2\,h\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right)}$
	$t_w/2 < h_n < b/2$	$\frac{A_c a_c f_{cd} - A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right) - t_w(h-2t_f)\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right)}{2\,h_c a_c f_{cd} + 4t_f\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right)}$

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Table 25 (*Concluded*)

Section	Case	Depth, h_n
(1)	(2)	(3)
	$b_f/2 \leq h_n \leq b_c/2$	$\frac{A_c a_c f_{cd} - A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right) - A_{sn}\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right)}{2\, h_c a_c f_{cd}}$
Concrete filled sections	RHS	$\frac{A_c a_c f_{cd} - A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right)}{2\, h_c a_c f_{cd} + 4t\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right)}$
	CHS	In RHS equation substitute D for h_c as an approximation

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D-4 DESIGN RESISTANCE OF COMPOSITE SHORT COLUMN UNDER COMBINED BENDING AND COMPRESSION

The stresses on the different components of the composite columns at the ultimate stage under only the bending moment about the z-z axis are as shown in Fig. 43.

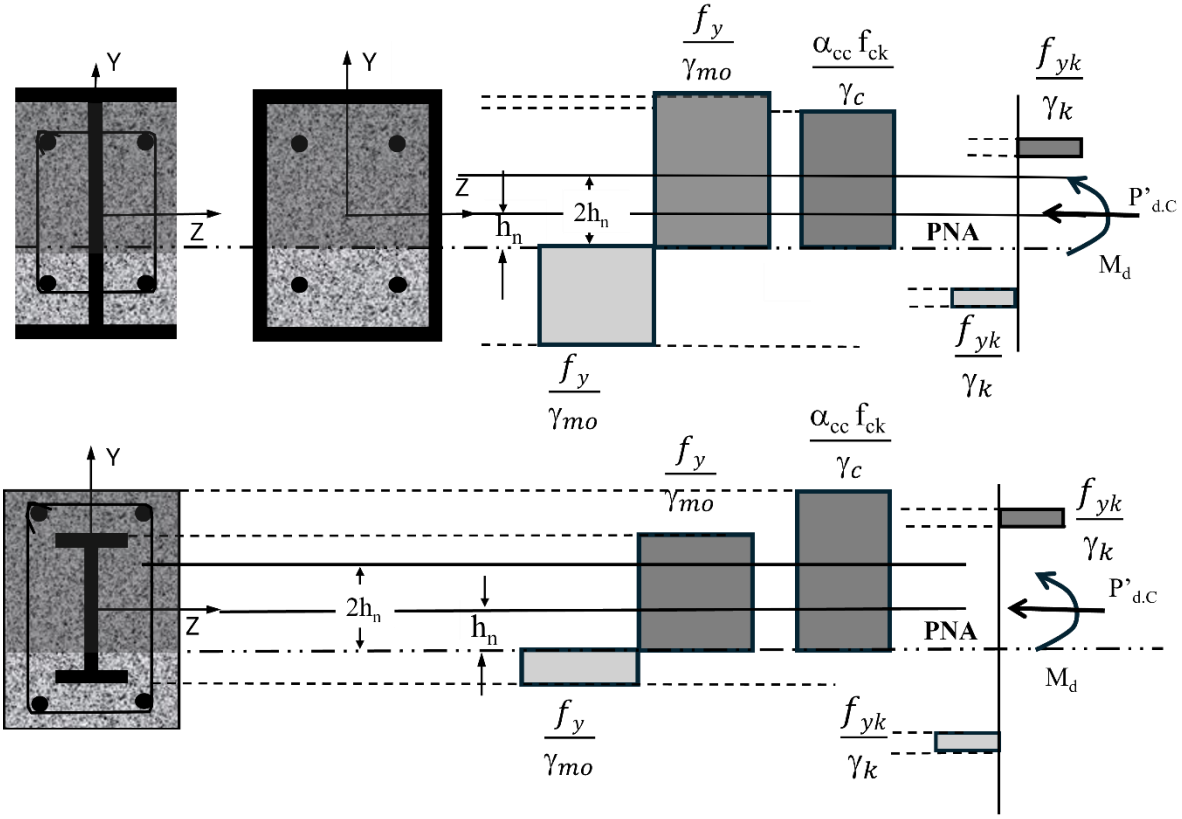

FIG. 43 SHORT COLUMN COMPRESSION AND BENDING INTERACTION

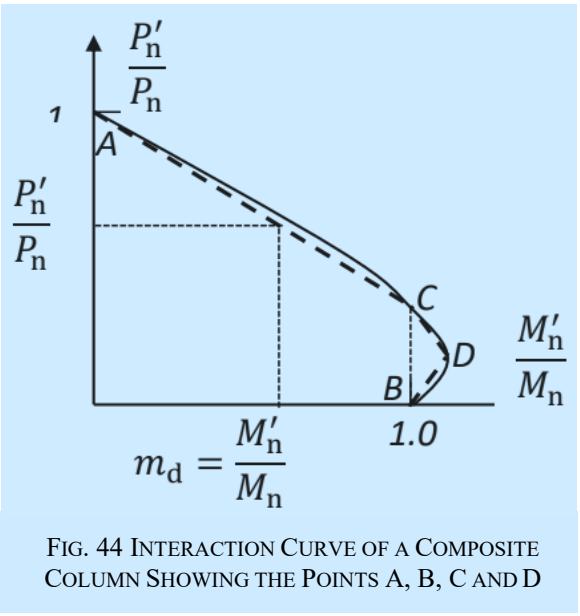

FIG. 44 INTERACTION CURVE OF A COMPOSITE COLUMN SHOWING THE POINTS A, B, C AND D

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If the concrete section in compression is below the neutral axis of the gross section, h_n, is assumed. The integration of the stresses over the cross section gives the ultimate nominal compressive resistance, P'_n, for the cross section. The integration of the moment of the stresses over the cross section about the neutral axis of the gross section gives the corresponding ultimate bending resistance, M'_n. For various values of h_n, the corresponding simultaneous compressive and bending resistance of the cross section can be calculated to obtain an interaction curve as shown in Fig. 44.

It is seen that the resistance under only compression (Table 23) is a point, (A), on the curve. The resistance under only bending is point, (B), on the curve. Further, from the case of pure bending, as the axial load is increased from zero value, the bending resistance also increases until point D. Similarly, the point C corresponds to the case, where in spite of the axial compression, the ultimate plastic bending resistance, M_n, is reached.

It is tedious and time-consuming to develop the full interaction curve for every section designed. In order to simplify, the values corresponding to the points A, B, C, and D are computed and in between a straight-line variation is assumed in the design. The simplified procedure and the equations are discussed in the following sections.

The values of P_d and M_d corresponding to plastic section under pure compression (point A, Fig. 41) and pure bending (Point B, Fig. 42) are obtained as in **D-2** and **D-3**.

D-4.1 The Design Resistance at Point C (Fig. 45)

The design bending resistance corresponding to point C is the same as pure bending resistance.

$$M'_{d,C} = M_d$$

The design compressive resistance corresponding to point C on the interaction diagram is obtained as given below. The typical position of the neutral axis for points *B* is shown in Fig. 43 and at the point C in the beam-column interaction curve in Fig. 44. The location of the neutral axis from the CG of section, h_n, can be determined from

Table 21. The integration of the stresses shown in Fig. 46 is equal to $P'_{pd,C}$ at point C. The axial force corresponding to point C in the interaction curve is given by the equation in Table 26.

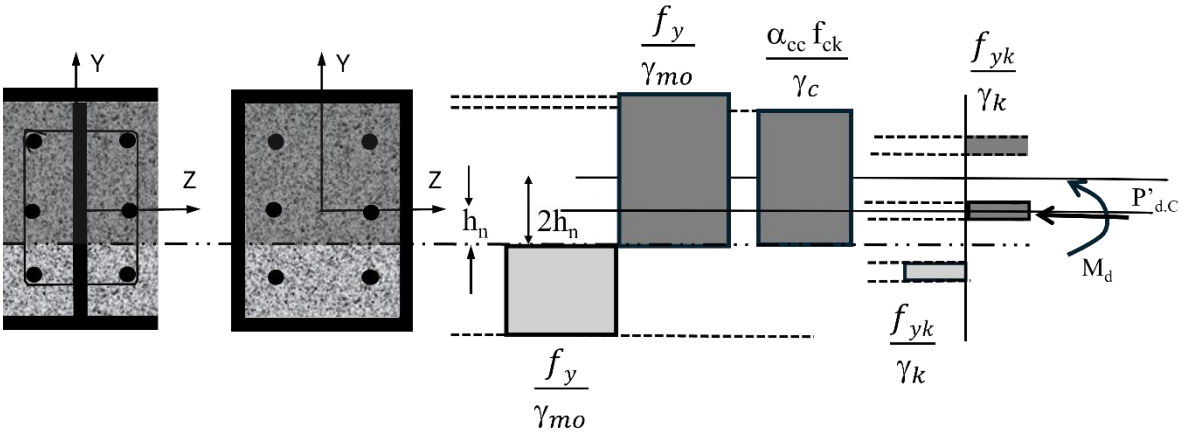

FIG. 45 STRESS DISTRIBUTIONS FOR THE POINT C OF THE INTERACTION CURVE FOR CONCRETE FILLED RECTANGULAR TUBULAR SECTIONS

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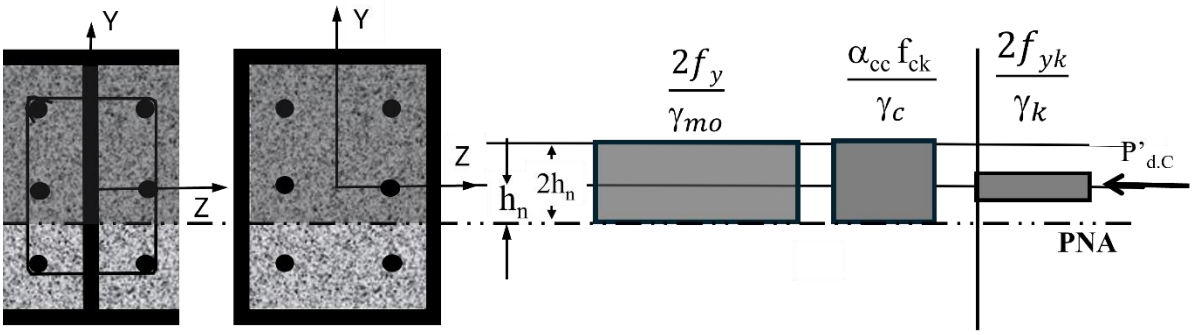

FIG. 46 STRESS CAUSING RESISTING COMPRESSION AT POINT C

Table 26 Design Compressive Resistance at Point C

(*Clause* D-4.1)

Section	Case	$P'_{d,C}$
(1)	(2)	(3)
a) *Major Axis Bending*		
Concrete encased column	$h_n \le [\, h/2 - t_f \,]$	$P'_{d,C} = 2\, h_n b_c a_c f_{cd} + 2h_n t_w \left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right) + A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right)$
	$[h/2 - t_f\,] \le h_n \le h/2$	$P'_{d,C} = 2\, h_n b_c a_c f_{cd} + (A_s - (h - 2h_n)\, b_f)\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right) + A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right)$
	$h/2 \le h_n \le h_c/2$	$P'_{d,C} = 2\, h_n b_c a_c f_{cd} + (A_{sn})\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right) + A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right)$
	RHS	$P'_{d,C} = 2\, h_n b_c f_{cd} + 4h_n t\left(2\frac{f_y}{\gamma_m} - a_c f_{cd}\right) + A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right)$
Concrete infilled sections	CHS	In RHS equation substitute D for b_c
b) *Minor Axis Bending*		
Concrete encased column	$h_n \le t_w/2$	$P'_{d,C} = 2\, h_n h_c a_c f_{cd} + 2h_n h\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right) + A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right)$
	$t_w/2 < h_n < b/2$	$P'_{d,C} = 2\, h_n h_c a_c f_{cd} + \left(4h_n t_f + t_w\,(h - t_f)\right)\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right) + A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right)$
	$b/2 \le h_n \le b_c/2$	$P'_{d,C} = 2\, h_n h_c a_c f_{cd} + A_{sn}\left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right) + A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right)$
Concrete infilled sections	RHS	$P'_{d,C} = 2\, h_n h_c\, a_c f_{cd} + 4h_n t_f \left(2\frac{f_y}{\gamma_{m0}} - a_c f_{cd}\right) + A_{stn}\left(2\frac{f_{yk}}{\gamma_k} - a_c f_{cd}\right)$

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D-4.2 The Design Resistance at Point D (Fig. 44)

The stress diagram on the section corresponding to point D on the interaction curve is shown in Fig. 47.

$$P'_{d,D} = 0.5\, P'_{d,C}$$

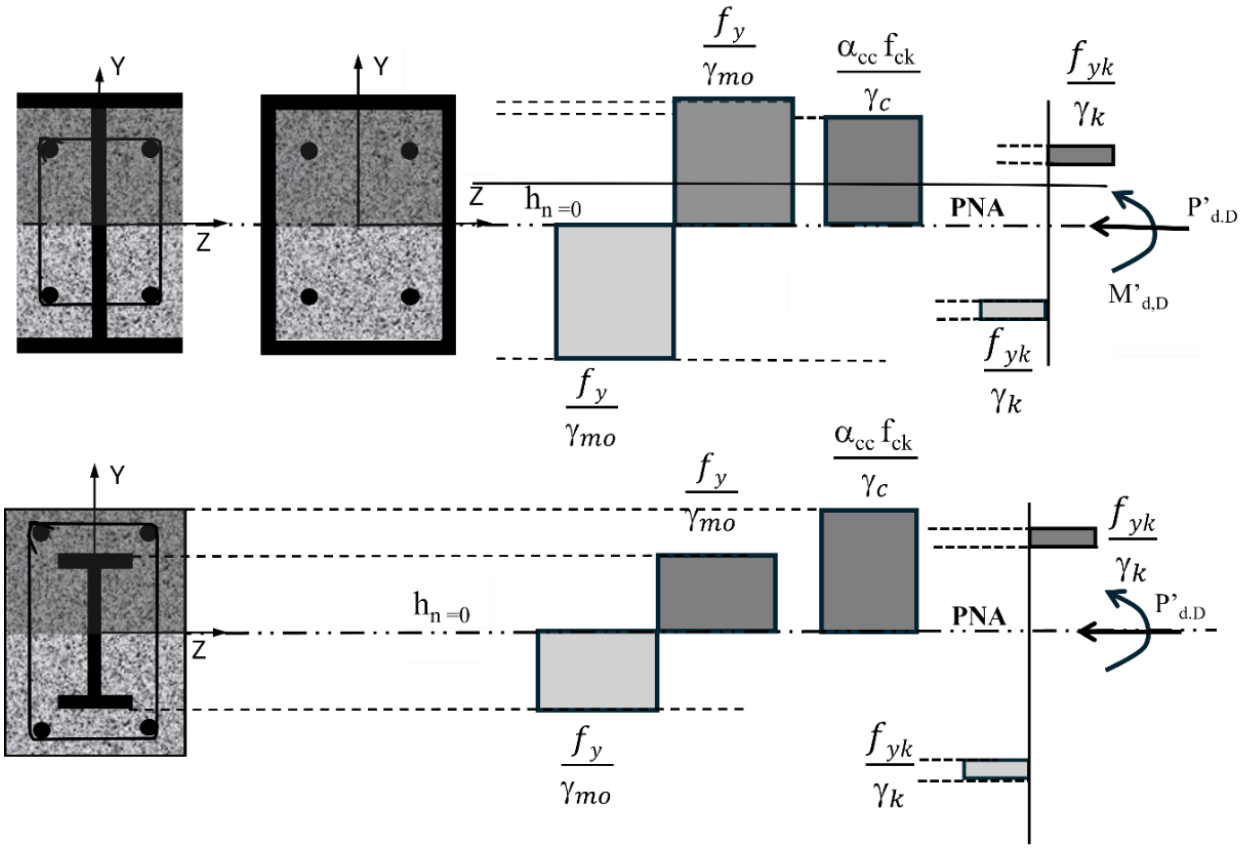

FIG. 47 STRESS DISTRIBUTION AT POINT D AT ULTIMATE STAGE UNDER COMBINED COMPRESSION AND BENDING

$$M'_{d,D} = M_{max} = Z_{ps}\frac{f_y}{\gamma_{m0}} + Z_{pr}\frac{f_{yk}}{\gamma_k} + 0.5 a_c\, Z_{pc}\frac{a_{cc}\, f_{ck}}{\gamma_C}$$

Knowing the values of the compressive and bending design resistances at the points A, B, C and D the values of the resistance at any other point may be obtained approximately by linear interpolation, as shown by dashed lines in Fig. 44, assuming a linear variation.

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ANNEX E

(*Clause* 16)

TESTING

E-1 TESTING OF MATERIALS

Testing of materials shall be done as per standard laid down norms given in **E-1.1** to **E-1.3**.

E-1.1 Concrete

For testing of concrete, reference shall be made to as per good practice [C-6B(2)].

E-1.2 Steel Sections

All the structural steel and accessories like rivets, bolts, nuts, washers, welding consumables, steel forging, casting, etc., shall be tested for mechanical and chemical properties as applicable and shall conform to requirements of appropriate Indian standards referred to in as per good practice [C-6B(3)]. Steel shall conform to the requirements of the accepted standard [C-6B(4)] and the accepted standard [C-6B(19)]. Bolts and bolted connection joints with high strength bolts shall be inspected according to as per good practice [C-6B(20)].

For testing of the strength, flexibility, and other relevant properties of the shear connectors, the test procedures as indicated in **E-1.3** shall be adopted.

E-1.3 Testing of Shear Studs for Steel-Concrete Interface Shear

The nominal static strength of a shear connector may be determined by push-out tests. The conditions and procedures which shall be followed while performing the tests are as indicated below:

a) The dimensions of the standard test are as shown in Fig. 48;

b) The bond at the steel-concrete interface shall be prevented by greasing the flanges or by any other suitable method;

c) The rate of application of load should be uniform and such that the failure load is reached in not less than 10 minutes;

d) The strength of the concrete f_c, at the time of testing, should not differ from the specified cube strength f_{ck} of the concrete by more than $\pm$ 20 percent;

e) Not less than three tests shall be done, and the nominal static strength P_u shall be taken as the lowest value of $f_{ck}.P / f_c$ for any of the tests, where P is the failure loads of the connectors at concrete strength f_c and f_{ck} is the characteristic cube strength of the concrete; and

f) The load-deformation curve, the ultimate resistance of the shear per shear connector, relative slip at ultimate load shall be recorded.

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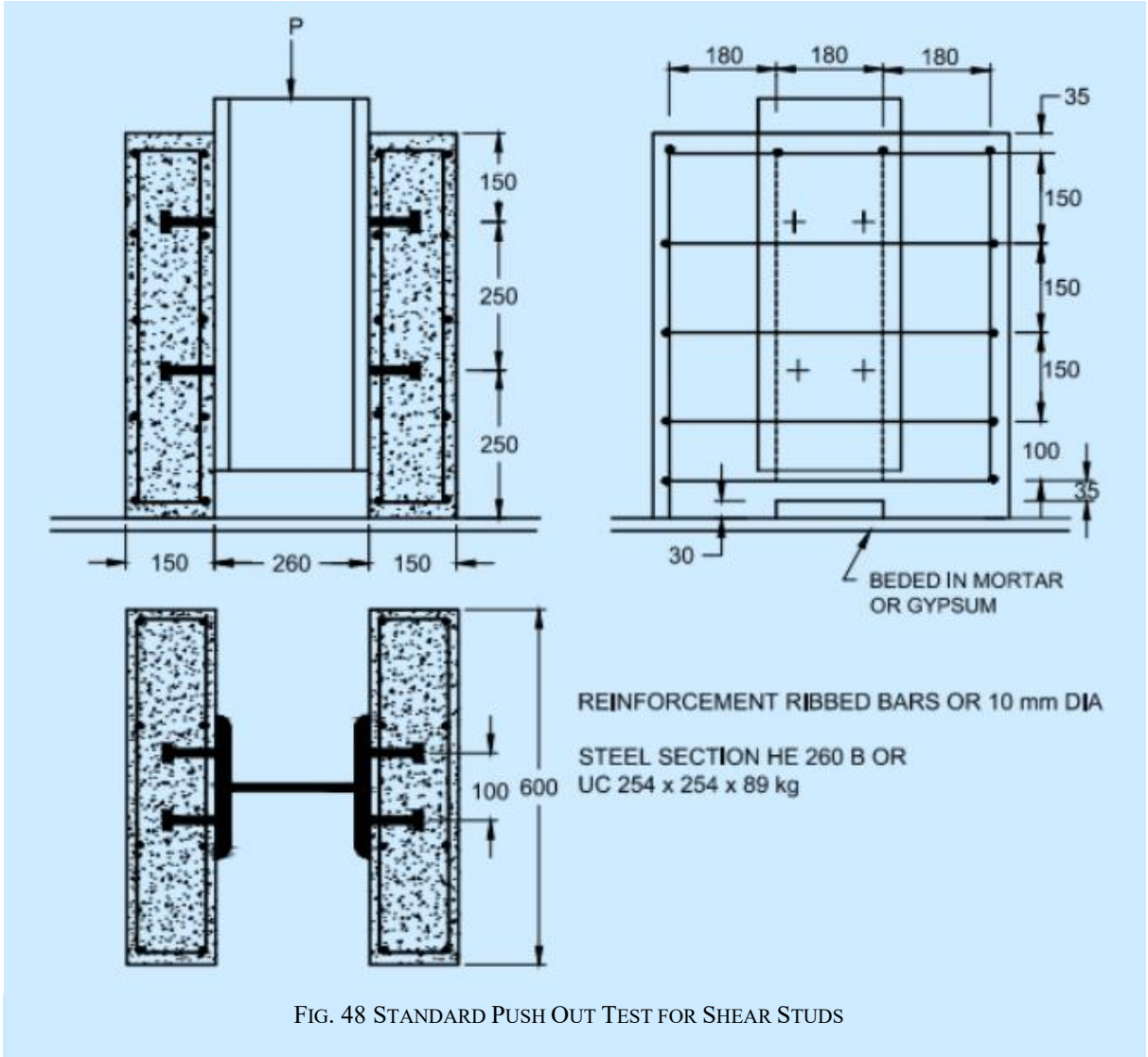

FIG. 48 STANDARD PUSH OUT TEST FOR SHEAR STUDS

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ANNEX F

(*Clauses* 5.3.2, 6.1.2, 8.1.2.1, 9.3 *and* 11.2.1)

STRENGTH DETAILS AND ELASTIC MODULUS OF NORMAL CONCRETE

Sl No.	*Symbol*	*Grade of Concrete*	M20	M25	M30	M35	M40	M45	M50	M55	M60	M65	M70	M75
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
i)	f_{ck},(*MPa*)	Characteristic cube compressive strength	20	25	30	35	40	45	50	55	60	65	70	75
ii)	f_{cm} (*Mpa*)	Mean concrete compressive strength	30	35	40	45	50	55	60	65	70	75	80	85
iii)	f_{ctm} (*MPa*)	Mean tensile strength of concrete	1.91	2.21	2.50	2.77	3.03	3.28	3.52	3.75	3.97	4.42	4.54	4.66
iv)	f_{ctk} (0.05) (MPa)	Characteristic tensile strength at 5 percent fractile of concrete	1.34	1.55	1.75	1.94	2.12	2.29	2.46	2.62	2.78	3.09	3.18	3.26
v)	E_{cm} (*MPa*)	Elastic modulus	28 608	29 962	31 187	32 308	33 346	34 313	35 220	36 076	36 887	37 659	38 395	39 100

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LIST OF STANDARDS

The following list records those standards which are acceptable as 'good practice' and 'accepted standards' in the fulfilment of the requirements of the Code. The latest version of a standard shall be adopted at the time of enforcement of the Code. The standards listed may be used for conformance with the requirements of the referred clauses in the Code.

In the following list, the number appearing in the first column within parentheses indicates the number of the reference in this Part/Section.

Sl No.	*IS No.*	*Title*
(1)	IS 456 : 2000	Plain and reinforced concrete — Code of practice (*fourth revision*)
	IS 800 : 2026	General construction in steel — Code of practice (*fourth revision*)
(2)	IS 456 : 2000	Plain and reinforced concrete — Code of practice (*fourth revision*)
(3)	IS 800 : 2026	General construction in steel — Code of practice (*fourth revision*)
(4)	IS 2062 (Part 1) : 2025	Structural steel: Part 1 Hot rolled medium and high tensile steel (*eighth revision*)
(5)	IS 1608 (Part 1) : 2018	Metallic materials tensile testing: Part 1 Method of test at room temperature (*fourth revision*)
(6)	IS 808 : 2021	Dimensions for hot rolled steel beam, column, channel and angle sections (*fourth revision*)
	IS 1161 : 2014	Steel tubes for structural purposes — Specification (*fourth revision*)
	IS 1239 (Part 1) : 2004	Steel tubes, tubulars and other wrought steel fittings — Specification: Steel tubes (*sixth revision*)
	IS 1239 (Part 2) : 2011	Steel tubes, tubulars and other wrought steel fittings — Specification: Steel pipe fittings (*fifth revision*)
	IS 1730 : 1989	Dimensions for steel plates, sheets, strips and flats for general engineering purposes (*second revision*)
	IS 1732 : 1989	Dimensions for steel bars, round and square for structural and general engineering purposes (*second revision*)
	IS 1852 : 1985	Specification for rolling and cutting tolerances for hot rolled steel products (*fourth revision*)
	IS 2062 (Part 1) : 2025	Structural steel: Part 1 Hot rolled medium and high tensile steel (*eighth revision*)
	IS 4923 : 2017	Hollow steel sections for structural use — Specification (*third revision*)
	IS 11587 : 1986	Specification for structural weather resistant steels
	IS 12779 : 1989	Rolling and cutting tolerances for hot rolled parallel flange beam and column sections
(7)	IS 1875 : 1992	Carbon steel billets, blooms, slabs and bars for forgings (*fifth revision*)
	IS 6911 : 2017	Stainless steel plate, sheet and strip — Specification (*first revision*)
(8)	IS 1030 : 1998	Carbon steel castings for general engineering purposes — Specification (*fifth revision*)
	IS 1875 : 1992	Carbon steel billets, blooms, slabs and bars for forgings (*fifth revision*)
	IS 2004 : 1991	Carbon steel forgings for general engineering purposes — Specification (*third revision*)

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Sl No.	*IS No.*	*Title*
	IS 2644 : 1994	High strength steel castings for general engineering and structural purposes — Specification (*fourth revision*)
	IS 4367 : 1991	Alloy steel forgings for general industrial use — Specification (*first revision*)
(9)	IS 1148 : 2009	Steel rivet bars (medium and high tensile) for structural purposes (*fourth revision*)
	IS 1363	Hexagon head bolts, screws and nuts of product grade C:
	Part 1 : 2019	Part 1 Hexagon head bolts (size range M 5 to M 64) (*fifth revision*)
	Part 2 : 2018	Hexagon Head Screws (Size Range M 5 to M 64) (*fifth r*evision)
	Part 3 : 2018	(Style 1) Hexagon nuts (size range M 5 to M 64) (*fifth revision*)
	IS 1364 (Part 1) : 2018	Hexagon head bolts, screw and nuts products grade A and B: Part 1 Hexagon head bolts (size range M 1.6 to M 64) (*fifth revision*)
	IS 1367	Technical supply conditions for threaded steel fasteners:
	Part 1 : 2014	General requirements for bolts, screws, studs and nuts (*fourth revision*)
	Part 2 : 2002	Tolerances for fasteners — Bolts, screws, studs and nuts — Product grades a, b and c (third revision)
	Part 3 : 2017	Mechanical properties of fasteners made of carbon steel and bolts, screws and studs (fifth revision)
	Part 5 : 2018	Mechanical properties of fasteners made of carbon steel and alloy steel — Set screws and similar threaded fasteners with specified hardness classes — Coarse thread and fine pitch thread (fourth revision)
	Part C : 2018	Mechanical properties of fasteners made of carbon steel and alloy steel — Nuts with specified property classes — Coarse thread and fine pitch thread (fourth revision)
	Part 7 : 1980	Mechanical properties and test methods for nuts without specified proof loads (*second revision*)
	Part 8 : 2020	Prevailing torque type steel nuts — Functional properties (*fifth revision*)
	Part 9/Sec 1 : 1993	Surface discontinuities, Section 1 bolts, screws and studs for general applications (*third revision*)
	Part 9/Sec 2 : 1993	Surface discontinuities, Section 2 bolts, screws and studs for special applications (*third revision*)
	Part 10 : 2002	Surface discontinuities — Nuts (*third revision*)
	Part 11 : 2020	Electroplated coatings systems (*fourth revision*)
	Part 12 : 1981	Phosphate coatings on threaded fasteners (*second revision*)
	Part 13 : 2020	Hot — Dip galvanized coatings on threaded fasteners (*third revision*)
	Part 14/Sec 1: 2018	Mechanical properties of corrosion — Resistant stainless — Steel fasteners, Section 1 Bolts, screws and studs (*fourth revision*)
	Part 14/Sec 2 : 2018	Mechanical properties of corrosion-resistant stainless steel fasteners, Section 2 Nuts (*fourth revision*)
	Part 16 : 2002	Designation system for fasteners (third revision)
	Part 17 : 2005	Inspections, sampling and acceptance procedure (*fourth revision*)

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Sl No.	*IS No.*	*Title*
	Part 18 : 1996	Packaging (*third revision*)
	IS 1929 : 1982	Specification for hot forged steel rivets for hot closing (12 to 36 mm diameter) (*first revision*)
	IS 2155 : 1982	Specification for cold forged solid steel rivets for hot closing (6 to 16 mm diameter) (*first revision*)
	IS 3640 : 1982	Specification for hexagon fit bolts (*first revision*)
	IS 3757 : 1985	Specification for high strength structural bolts (*second revision*)
	IS 4000 : 1992	High strength bolts in steel structures — Code of practice (*first revision*)
	IS 5369 : 1975	General requirements for plain washers and lock washers (*first revision*)
	IS 5370 : 1969	Specification for plain washers with outside diameter 3 x inside diameter
	IS 5372 : 1975	Specification for taper washers for channels (ISMC) (*first revision*)
	IS 5374 : 1975	Specification for taper washer for I-beams (1SMB) (*first revision*)
	IS 5624 : 1993	Foundation bolts — Specification (*first revision*)
	IS 6610 : 1972	Specification for heavy washers for steel structures
	IS 6623 : 2004	High strength structural nuts (*second revision*)
	IS 6649 : 1985	Specification for hardened and tempered washers for high strength structural bolts and nuts (*first revision*)
	IS 7002 : 2018	Prevailing torque type hexagon nuts (with non-metallic insert), style — Property classes 5, 8 and 10 (*third revision*)
(10)	IS 813 (Part 1) : 2018 ISO 2553:2013	Welding and allied processes symbolic representation on drawings-welded joints (*second revision*)
	IS 814 : 2004	Covered electrodes for manual metal arc welding of carbon and carbon manganese steel — Specification (*sixth revision*)
	IS IS 3613 : 1974	Acceptance tests for wire-flux combinations for submerged-arc welding of structural steels
	IS 1395 : 1982	Specification for low and medium alloy steel covered electrodes for manual metal arc welding (*third revision*)
	IS 6419 : 1996	Welding rods and bare electrodes for gas shielded arc welding of structural steel — Specification (*first revision*)
	IS 6560 : 2017	Welding consumables — wire electrodes, wires rods and deposits for gas shielded arc welding of creep resisting steel — Classification (*second revision*)
	IS 7280 : 1974	Specifications for bare wire electrodes for submerged arc welding of structural steel
	IS 812 : 1957	Glossary of terms relating to welding and cutting of metals
	IS 816 : 1969	Code of practice for use of metal arc welding for general construction in mild steel (*first revision*)
	IS 822 : 1970	Code of procedure for inspection of welds
	IS 1024 : 1999	Use of welding in bridges and structures subject to dynamic loading — Code of practice (*second revision*)
	IS 1182 : 1983	Recommended practice for radiographic examination of fusion welded butt joints in steel plates (*second revision*)

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Sl No.	*IS No.*	*Title*
	IS 4853 : 1982	Recommended practice for radiographic inspection of fusion welded butt joints in steel pipes (*first revision*)
	IS 5334 : 2003	Code of practice for magnetic particle flaw detection of welds (*third revision*)
	IS 7307 (Part 1) : 1974	Approval tests for welding procedures: Part 1 Fusion welding of steel
	IS 7310 (Part 1) : 2019	Approval tests for welders procedures: Part 1 Fusion welding of steel (*first revision*)
	IS 7318 (Part 1) : 1974	Approval tests for welders when welding procedure approval is not required: Part 1 Fusion welding of steel
	IS 9595 : 1996	Metal — Arc welding of carbon and carbon manganese steels — Recommendations (*first revision*)
	IS 15977 : 2013	Classification and acceptance tests for bare solid wire electrodes and wire flux combination for submerged arc welding of structural steel — Specification
(11)	IS 1785 (Part 1)	Specification for plain hard-drawn steel wire for pre-stressed concrete: Part 1 Cold drawn stress relieved wire (*second revision*)
	IS 1785 (Part 2)	Specification for plain hard-drawn steel wire for pre-stressed concrete: Part 2 As-drawn wire (*first revision*)
	IS 2266 : 2019	Steel wire ropes for general engineering purposes – Specification (*fifth revision*)
	IS 2315 : 1978	Specification for thimbles for wire ropes (*first revision*)
(12)	IS 432 (Part 1) : 1982	Specification for mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement: Part 1 Mild steel and medium tensile steel bars (*third revision*)
(13)	IS 1786 : 2008	High strength deformed steel bars and wires for concrete reinforcement — Specification (f*ourth revision*)
(14)	IS 13620 : 1993	Fusion bonded epoxy coated reinforcing bars — Specification
(15)	IS 16651 : 2017	High strength deformed stainless steel bars and wires for concrete reinforcement — Specification
(16)	IS 1343 : 2012	Prestressed concrete — Code of practice (*second revision*)
(17)	IS 808 : 1989	Dimensions for hot rolled steel beam, column, channel and angle sections (*third revision*)
(18)	IS 801 : 1975	Code of practice for use of cold — Formed light gauge steel structural members in general building construction (*first revision*)
(19)	IS 11587 : 1986	Specification for structural weather resistant steels
(20)	IS 4000 : 1992	High strength bolts in steel structures — Code of practice (*first revision*)

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 7 PREFABRICATED CONCRETE CONSTRUCTION

BUREAU OF INDIAN STANDARDS

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National Building Code Sectional Committee, CED 46

FOREWORD

This NBCS (Part C/Section 7) gives recommendations regarding modular planning, component sizes, prefabrication systems, design considerations, joints and manufacture, storage, transportation and erection of prefabricated concrete elements for use in buildings and such related requirements for prefabricated concrete.

Prefabrication, though desirable for large-scale building activities, has been gaining importance in the country over the past decade. Two key aspects of prefabrication that need to be considered are the selection of an appropriate system for different categories of buildings and the sizing of their components. In this context, the principle of modular coordination is of significant value and its use is recommended.

Advantages of recent trends in prefabrication have been taken note of and also the hazards attended to such construction. Some of the essential requirements for the manufacture of the prefabricated components and elements are also included in this section.

Since the aim of prefabrication is to effect economy and improve quality and speed of construction, the selection of proper materials for prefabrication is also an important factor in the popularization of this technique. The use of locally available materials with required characteristics and the materials which, due to their innate characteristics such as lightweight, easy workability, thermal insulation, non-combustibility, etc, that effect economy and improve quality, may also be tried. However, this Section pertains to prefabricated elements with cementitious materials.

The design of prefabricated buildings shall include provision for installations of all services and required piping, wiring and accessories to be installed in the building.

This Section was first published in 1970 and was subsequently revised in 1983, 2005 and 2016 as a chapter of National Building Code of India. In the first revision in 1983, the main changes made included: inclusion of brief provision regarding importance of architectural treatment and finishes as applicable to prefabricated buildings; addition of a brief clause on the requirements of materials for use in prefabrication; elaboration of the clause on prefabricating systems and structural elements; revision of the clause on testing of components to include testing of structure or part of structure; and addition of a brief clause on the manufacture of cellular concrete.

In the second revision in 2005, this Section, earlier named as ‘Prefabrication and systems building’, was renamed and restructured as follows:

Section 7 Prefabrication, systems buildings and mixed/composite construction

Subsection 7A Prefabricated concrete

Subsection 7B Systems buildings and mixed/composite construction

In the second revision in 2005, several significant changes were introduced to enhance the standard. The modular coordination and modular dimensions of components were revised to allow greater flexibility in planning. Provisions related to tolerances were updated to address various types of prefabricated components. Additionally, a detailed clause on design requirements was incorporated to ensure the safety of prefabricated buildings against progressive collapse. Furthermore, a new clause specifying the sampling procedure for testing components was added.

In the third revision in 2016, in the Subsections 7A and 7B, several significant modifications had been incorporated. Definitions for new terms had been added and existing terminologies had been modified where necessary. Provisions related to prefabricated systems had been updated and detailed guidelines on the diaphragm action of floor systems had been included. The emulative system had been categorized and elaborated on, covering its definition, analysis, design and detailing. Design considerations had been revised to account for accidental impacts due to vehicles. Provisions and testing procedures for the water tightness of joints had been introduced, along with illustrations of typical precast joint details. Fire resistance testing of prototypes under sustained loads had been addressed and reliance solely on frictional resistance for connections between adjacent prefabricated

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members had been replaced with recommendations for resistance using shear. Updates have also been made to provisions on site prefabrication and vertical stacking guidelines, including restrictions on the number of components stacked on the ground. The time duration for load testing selected components from a lot had been modified. Additionally, new provisions for tolerances during erection and associated design considerations had been added. An annex had been included to address common defects, their causes and remedies, covering issues such as dimensional deviations, cracks, honeycombing, damages, strand slippage and alignment problems.

In this revision, this publication has been renamed as National Building Construction Standards (NBCS).

In this revision, it has been decided to merge subsections 7A and 7B into Section 7, 'Prefabricated Concrete Construction'. The following major modifications have been incorporated in this revision:

a) Definitions of some new terms have been added and existing terminologies modified, wherever required;

b) The emulative design approach has been adopted wherein precast elements are connected at site using wet concrete or grout;

c) Emulative system has been categorized and detailed with respect to its definition, analysis, design, detailing, etc;

d) Provisions relating to prefab systems namely moment resisting frame system, braced frame system, load bearing wall system, cell system and hybrid system are given along with their types and their applications;

e) Provisions relating to the design of corbel, bearing including dapped bearing are given in detail;

f) Provisions relating to quality control and assurance have been included;

g) Provisions for tolerances in erection and associated design considerations have been updated;

h) Provisions and testing procedures for water tightness of joints have been included;

j) A guideline for 3D printed concrete structures has been included in Annex B;

k) A comprehensive guideline on prefabricated prefinished volumetric construction (PPVC) methods, materials, design standards and performance criteria for guidance to building professionals and stakeholders involved in modular construction has been included in Annex C;

m) Guidelines for design for manufacturing and assembly has been included in Annex D;

n) Annex E on common precast connection has been modified based on the current practices in precast industry; and

p) Sustainability in precast concrete construction has been included in Annex G.

This information contained in this section is largely based on IS 15916 : 2026, 'Prefabricated concrete construction — Code of practice (*second revision*)' (*under publication*).

All standards, whether given herein above or cross-referred to in the main text of this Section, are subject to revision. The parties to agreement based on this Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

As the regulation of land development and buildings is a State subject and is dealt by the States and local bodies such as urban local bodies within their jurisdiction through building regulatory documents like building regulations, building byelaws, and fire regulations, this document is voluntary in nature and is non-binding. The implementation depends on adoption by concerned parties or stipulations in a contract or by appropriate adoption by the concerned authorities.

For the purpose of deciding whether a particular requirement of this Section is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 2022 'Rules for rounding off numerical values (*second revision*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this Section.

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Important Explanatory Note for Users of the NBCS

In any Part/Section of this NBCS, where reference is made to **'good practice'** in relation to design, constructional procedures or other related information, and where reference is made to **'accepted standard'** in relation to material specification, testing, or other related information, the Indian Standards listed at the end of the Part/Section shall be used as a guide to the interpretation.

At the time of publication, the editions indicated in the standards were valid. All standards are subject to revision and parties to agreements based on any Part/Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

In the list of standards given at the end of a Part/Section, the number appearing within parentheses in the first column indicates the number of the reference of the standard in the Section. For example:

a) Accepted standard [C-7(1)] refers to the Indian Standard(s) give at serial number (1) of the list of standards given at the end of this Part C/Section 7, that is, IS 13920 : 2016 'Ductile design and detailing of reinforced concrete structures subjected to seismic forces — Code of practice (*first revision*)',

b) Accepted standard [C-7(9)] refers to the Indian Standard(s) give at serial number (9) of the list of standards given at the end of this Part C/Section 7, that is, IS 16172 : 2023 'Reinforcement couplers for mechanical splices of steel bars in concrete — Specification (*first revision*)'.

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 7 PREFABRICATED CONCRETE CONSTRUCTION

1 SCOPE

1.1 This NBCS (Part C/Section 7) covers provisions for prefabricated concrete construction in buildings, including prefabrication systems, design considerations, element tolerances, joints and requirements for manufacture, storage, transportation and erection.

1.2 This Section is limited to the emulative detailing approach, wherein precast elements are connected at site using wet concrete or grout with reinforcement detailing in accordance with this Section and relevant design standards.

1.3 Structural analysis shall consider appropriate computational models, including connections, and the overall performance shall be verified in terms of strength, stiffness, and ductility. The design intent shall be to achieve performance comparable to cast in-situ concrete construction. The design and detailing provisions specified for cast in-situ concrete in good practice [C-7(1)] shall form the basis, and the design engineer shall provide similar detailing to achieve equivalent behaviour compare to cast in situ practice. If required by the contract statement, this can be demonstrated by appropriate testing as specified by the Competent Authority and 12 in agreement with the design team and client.

1.4 Where exact replication of cast in-situ detailing is not feasible, the minimum requirements specified in this Section shall be followed. In cases not covered by this Section, alternative detailing approaches may be adopted, provided their performance is validated through testing (in terms of strength, stiffness, and ductility) as specified and approved by the competent authority in agreement with the design team and client.

2 —

NOTE — This clause is intentionally left blank.

3 TERMINOLOGY

For the purpose of this section, the following definitions shall apply.

3.1 Authority Having Jurisdiction — The authority which has been created by a statute and which, for the purpose of administering the NBCS/part, may authorize a committee or an official or an agency to act on its behalf; hereinafter called the ‘Authority’.

3.2 Basic Module — The fundamental module used in modular coordination, the size of which is selected for general application to building and its components.

NOTE — The value of the basic module has been chosen as 50 mm for the maximum flexibility and convenience. The symbol for the basic module is *M*.

3.3 Cellular Concrete — The material consisting of an inorganic binder (such as lime or cement or both) in combination with a finely ground material containing siliceous material (such as sand), gas generating material (for example, aluminium powder), water and harmless additives (optional); and/or steam cured under high pressure in autoclaves.

3.4 Components — A building product formed as a distinct unit having specified sizes in three dimensions.

3.5 Composite Members — Structural members comprising of prefabricated structural units of steel, prestressed concrete or reinforced concrete and cast in-situ concrete connected together in such a manner that they act monolithically.

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3.6 Diaphragm — It is a horizontal or nearly horizontal structural system (for example, reinforced concrete floors and horizontal bracing systems), which transmits lateral forces to vertical elements that resist earthquake-induced inertia effects.

3.7 Increments — Difference between two homologous dimensions of components of successive sizes.

3.8 Light-Weight Concrete — Concrete of substantially lower unit weight (1 400 kg/m^3 to 1 800 kg/m^3) than that made with gravel or crushed stone.

3.9 Module — A unit of size used in dimensional coordination.

3.10 Modular Coordination – Dimensional coordination employing the basic module or a multimodule.

NOTE — The purposes of modular co-ordination are:

a) to reduce the variety of component sizes produced; and

b) to allow the building designer greater flexibility in the arrangement of components.

3.11 Modular Grid — A rectangular coordinate reference system in which the distance between consecutive lines is the basic module or a multimodule. This multimodule may differ for each of the two dimensions of the grid.

3.12 Multimodule — A module whose size is a selected multiple of the basic module.

3.13 Prefabricate — To fabricate concrete components or assembled units prior to erection or installation in a building.

3.14 Prefabricated Building — The partly/fully assembled and erected building, of which the structural parts consist of prefabricated individual units or assemblies using ordinary or controlled materials, including service facilities; and in which the service equipment may be either prefabricated or constructed in-situ.

3.15 Sandwich Concrete Panels — Panels made by sandwiching an insulation material between two layers of reinforced/prestressed concrete to act as insulation for composite or non-composite concrete panels.

3.16 Self Compacting Concrete — Concrete that is able to flow (650 mm in diameter) under its own weight and completely fill the voids within the formwork, even in the presence of dense reinforcement without any vibration, whilst maintaining homogeneity without segregation.

3.17 Shear Connectors — Structural elements, such as anchors, studs, channels, loops and spirals, intended to transmit the shear between the prefabricated member and the cast in-situ concrete and also to prevent separation at the interface (to transmit the interfacial shear).

3.18 System — It is a particular method of construction of buildings with certain order and discipline using the prefabricated components which are inter-related in functions and are produced based on a set of instructions.

3.19 Unit — Building component formed as a simple article with all three dimensions specified, complete in itself but intended to be part of a larger, compound unit or complete building. Examples are brick, block, floor panel, wall panel and other individual components.

3.20 Emulative Detailing System — A connection detailing system for precast concrete structures that provides a similar level of ductility equivalent to that of a conventionally designed, cast in-situ, monolithic concrete structure subjected to earthquake.

3.21 Jointed Detailing System — A connection detailing system for precast concrete structures that has individual precast components separated from each other but connected using special connections such as welded or bolted plates. These connections are not allowed as per this Section, if adopted appropriate testing needs to be carried out as per **10**.

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3.22 Terminology Relating to 3D Printing

3.22.1 *Extrusion-Based Concrete 3D Printing* — Extrusion based printing refers to the delivery of the concrete through a nozzle in the required shape and dimension, in a layer-by-layer deposition process. The nozzle can either be placed on a gantry or attached to a central robot or a rail mounted mechanism.

3.22.2 *Binder Jetting-Based 3D Printing* —Binder jetting is a 3D printing technology in which a liquid binding agent (water) is sprayed on the surface of a thin layer of the binder (cement and aggregate) to attain the required shape and dimensions. A new layer of the binder is then placed on the top of the existing layer, and the process of spraying the water is repeated. This process is continued until reaching the required height of the object. The formed object is allowed to harden before removing the excess unreacted binder.

3.22.3 *Print Plane* — The plane on which the material is deposited.

3.22.4 *Directions in the Printed Specimens* — The performance of the printed specimen varies with the principal directions of the printed concrete. The directions of the printed concrete are designated as X, Y and Z with reference to the print pathway, where:

a) X direction is parallel to the print direction and in the print plane;

b) Y direction is perpendicular to the print direction in the print plane and could cross different layers; and

c) Z direction is perpendicular to the print direction and print plane, crossing many printed layers.

3.22.5 *Concrete for 3D Printing* — Concrete is a composite material with cement (ordinary Portland cement blended with or without supplementary cementitious materials), water, fine aggregate, coarse aggregate and admixtures. Fine aggregates and coarse aggregate need to comply with the requirements of good practices [C- 7(2)] and [C-7(3)]. Aggregates can be used in the mix subjected to testing and validation, confirming to the required printable properties like pumpability, buildability and extrudability.

3.22.6 *Printable Concrete* — Printable concrete is a mixture of cement (with or without supplementary cementitious materials), water, aggregates and admixtures that can be extruded through the nozzle of the printer and attain the designed dimension within certain tolerance zone on placing additional layers.

3.22.7 *Buildability* — Buildability is the ability of the printed layers to develop adequate green strength to withstand the self-weight as well as load due to subsequent layers without significant geometrical deformation or collapse of the freshly printed component.

3.22.8 *Extrudability* — Extrudability is the ability of the mixture to pass through the nozzle under pressure and attain the designed shape after the extrusion.

3.22.9 *Pumpability* — Pumpability is the ease of transporting concrete mixture from the pump to the nozzle.

3.22.10 *Printability* — The printability of the mixture is a combination of the pumpability and extrudability.

3.22.11 *Open Time* — Open time represents the time over which the 3D printing mixture remains printable. This defines the time period in which the mixture can be delivered through the pumping system and dispensed through the nozzle without clogging of the extrusion system.

3.22.12 *Filament* — The extruded shape is termed as the filament. The cross section of the filament can be circular, rectangular, or square, depending on the shape of the nozzle. The filament is characterised by its diameter (for circular nozzle) or by its orthogonal dimensions – width and thickness. However, it can be of any shape.

3.22.13 *Layer* — A layer is the object formed with a single pass of the printer. Multiple layers are stacked one over the other to build up the full object. Each layer is characterized by the height or thickness, for rectangular or square nozzles.

3.22.14 *Bond* — Bond is an interface between successive layers or filaments of a 3D printed concrete specimen. The interface between layers can be called as ‘horizontal’ bond, while the interface between filaments side by side can be called as ‘vertical’ bond.

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3.22.15 *Shape Stability* — Shape stability is the ability of the printed component to maintain the dimension as subsequent layers are placed.

3.22.16 *Base Layer* — A base layer is the first printed layer on which all following layers are built on.

3.22.17 *Dead Zone Length* — The dead zone length is defined as the length of static deposition of concrete or where the material flow stops such as near the nozzle with rapid change in cross section.

3.22.18 *Extracted Specimen* — Specimen of the required dimension which is extracted (cored/sawn) from the 3D printed concrete elements in any orientation.

3.22.19 *Speed of Extrusion* (*Extrusion Speed*) — The rate at which the extrusion system moves while extruding the concrete. Generally expressed in length/unit time, volume/unit time, or mass/unit time.

3.22.20 *Admixtures*

3.22.20.1 *Superplasticizers* — Superplasticizers are required in 3D printed mixes in order to control the workability of concrete and make it suitable for pumping and extrusion.

3.22.20.2 *Accelerators* — Accelerators are admixtures added to the concrete to decrease the initial setting time and increase the early-age strength.

3.22.21 *Viscosity Modifier* — An admixture that when added to the concrete, modifies the viscosity of the mix.

3.22.22 *Out-of-Plane Deformation* — Distortion/bending of the printed layers from the predefined (vertical) plane of the specimen. Intentional cantilevering or offset shall be allowed and is not considered as out-of-plane deformation.

4 NOTATIONS AND SYMBOLS

The notations used in this section are given at Annex A.

5 MATERIALS

The relevant materials for precast structures shall satisfy the requirements given in the following sections. For addressing sustainability aspects in selecting materials, a reference may be made to Annex G (*see* **G-2** and **G-5**).

5.1 Concrete Ingredients

5.1.1 *Cement*

The cement shall be selected in accordance with good practices [C-7(2)] to provide minimum required strength for lifting, transportation and durability. Each shipment of cement shall be referenced to a certified mill test report that indicates compliance with the specified type of cement. However, accelerated tests shall be carried out before application.

5.1.2 *Mineral Admixtures*

For mineral admixtures, the provisions of Part C ‘Structural Design, Section 5 Structural Concrete’ of this NBCS shall apply, unless otherwise specified by the client or the design.

5.1.3 *Aggregates*

Aggregates used in concrete shall comply with accepted standard [C-7(4)]. All aggregates must be sourced from approved suppliers, with representative samples tested according to the requirements specified in the governing standards. The desired concrete properties must be demonstrated through trial mixes to ensure compliance and satisfactory performance.

The provisions of good practice [C-7(2)] shall apply while deciding the maximum and minimum size of coarse aggregate.

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For stitch joints between precast elements with thickness less than 75 mm, the maximum size of aggregate shall be limited to 10 mm.

5.1.4 *Water*

Water used for concrete mixtures shall conform to Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS. In addition, mixing water shall be free from deleterious matter that may interfere with the setting or strength development of the concrete. Water shall be tested based on the batching requirements at least once a month.

5.1.5 *Chemical Admixtures*

5.1.5.1 All chemical admixtures such as accelerating admixtures, retarding admixtures, water-reducing admixtures, air-entraining admixtures and super-plasticizing admixtures shall comply with accepted standards [C-7(5)].

5.1.5.2 Admixtures containing chloride ions shall not be used.

5.1.5.3 Admixtures must be checked for compatibility with cement and other admixtures to ensure proper performance. A trial mix program shall evaluate slump, stability, workability, air content and strength under typical conditions, including temperature and humidity. The effect of dosage variations and the order of admixture addition shall be determined from supplier recommendations or trial.

5.1.5.4 Synthetic or natural pigments may be used in precast facade or decorative elements to achieve desired colours. The pigments must be insoluble in water, free of soluble salts, acids, calcium sulphate and resistant to sunlight, alkalis and weak acids. The pigment type and amount must not affect concrete setting time or strength.

5.2 Concrete Mix Design

Qualification of a concrete mix for use in precast concrete production shall be through laboratory testing, production trials and quality control testing. The minimum grades of concrete shall be as per good practices [C-7(2)].

5.2.1 *Mix Development*

5.2.1.1 Concrete mixes for precast elements shall be determined through laboratory methods, complying with Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS exposure class requirements. The mix shall meet project needs, service conditions and desired surface finish (colour and texture), with adequate workability for proper placement and consolidation. Water use shall be minimized to the amount necessary for proper placing and vibration consolidation.

5.2.1.2 For standard concrete, the mix design procedure shall be as per good practice [C-7(6)]. The good practices [C-7(2)] shall be used for the correlation between compressive strength and tensile strength, modulus of elasticity, shrinkage, creep coefficient, co-efficient of thermal expansion, etc.

5.2.1.3 Special concretes (for example, high-performance, ultra-high performance) shall be designed based on specialized literature and approved by the competent authority. The literature shall also correlate compressive strength with other properties such as tensile strength, modulus of elasticity, shrinkage, creep and thermal expansion.

5.2.1.4 Steel fibre reinforced concrete, specified in structural elements to enhance mechanical and durability properties, shall be designed using hooked, crimped or straight steel fibres as per specialized literature and with appropriate approvals from competent authority. Any construction joint in precast steel fibre reinforced concrete elements shall have reinforcement bar across the joint to transfer stresses.

5.2.1.5 Lightweight concrete may be used for non-structural precast elements (such as wall panels, facade units, and sandwich panels) to reduce the self-weight, enhance ease of handling and transportation, thermal insulation, and sound absorption. For lightweight concrete, adjustments are required during mix design to account for the lower density and altered workability. Specialized literature shall be referred and appropriate approvals from competent authority shall be obtained.

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5.2.2 *Miscellaneous*

5.2.2.1 All the concrete mixes to be used in the production of precast elements shall be evaluated as trial batches, prepared in accordance with good practice [C-7(7)] under conditions that simulate the actual production and finishing as closely as possible.

5.2.2.2 If the source/manufacturer of cement, source/gradation of aggregates or source/manufacturer of admixture are changed after mix finalization, the proportions of the mix shall be validated and re-designed, if required.

5.2.2.3 When accelerated curing is to be used, the mix proportions shall be based on similarly cured test specimens.

5.2.2.4 When self-compacting concrete is used, the mix proportion shall meet the requirement of good practices [C-7(8)].

5.3 Reinforcement

5.3.1 Reinforcing bars and prestressing strands shall be in accordance with the provisions of Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS, respectively. Reinforcing steel deliveries shall be identified with a heat number least that can be correlated with a mill certificate.

5.3.2 For welding of reinforcement, the good practice [C-7(9)] and Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS shall be referred.

5.3.3 Reinforcement couplers shall comply with the accepted standard [C-7(10)].

5.3.4 If glass fibre reinforced polymer (GFRP) bars are used, the material shall conform to the requirements of accepted standard [C-7(11)]. Such bars shall be used only in precast elements with low risk, based on the decision of engineer-in-charge and client.

5.3.5 Welded wire reinforcement shall conform to accepted standard [C-7(12)].

5.3.6 Welding shall not be performed near any prestressing strand, whether tensioned or not tensioned. The prestressing strand shall not be exposed to spatter, direct heat, or short-circuited current flow.

5.3.7 Prestressing accessories shall comply with the requirements of good practice [C-7(3)].

5.3.8 Plastics for welded wire and bar supports shall be composed of polyethylene, styrene copolymer rubber-resin blends, polyvinyl chlorides and polytetrafluoroethylene. Plastics shall be alkali-resistant and have thermal properties compatible with concrete.

5.4 Grout

5.4.1 *Cementitious Grout*

5.4.1.1 Cementitious grout used in the precast application shall be as per good practice [C-7(2)].

5.4.1.2 Non-shrink non-metallic high-strength and flowable cementitious grouts shall be used for all structural joints between two precast elements having gap less than 50 mm. The grout strength shall exceed the strength of the joining member by at least 5 MPa. If the strength of the members varies, the cementitious grout with the highest required strength shall be used. For special connection, strength of the grout shall be as per **8**.

5.4.1.3 For joint width more than that recommended in the grout manufacturer's guidelines or 50 mm, the grout shall be mixed with 10 mm down aggregate or self-compacting concrete or micro concrete shall be used.

5.4.2 *Chemical Grouts*

Chemical grouts shall be used as per manufacturer's guidelines and its properties shall comply with accepted standard [C-7(13)].

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5.5 Mould

5.5.1 All moulds, regardless of material, shall conform to the profiles, dimensions and tolerances of precast concrete product indicated by the contract documents and the approved shop drawings.

5.5.2 Repeated use of moulds shall not cause any dimensional or planar changes that exceed allowable tolerances of the member. When exposed to temperature fluctuations or moisture, the warping or buckling of moulds shall not exceed to allowable tolerance limits.

5.5.3 Moulds exposed to external vibrations must be capable of transmitting these vibrations evenly across a sufficient area, without flexing, warping or causing plate flutter.

5.5.4 If it is planned to apply a prestressing force by jacking against the mould, the mould shall be sufficiently strong to withstand the force without buckling or wrinkling and maintain the required dimensional tolerances.

5.5.5 When different materials are used to construct the mould, they shall not affect the colour and texture of the precast concrete product intended to be installed unrendered. The mould materials shall be non-absorbent or properly sealed to prevent excessive moisture absorption to minimize variations in finish.

5.5.6 Wooden moulds shall be sealed with suitable materials to prevent moisture absorption.

5.5.7 Steel moulds shall be visually inspected before each use for rust, distortion and tightness of steel sheet joints.

5.5.8 Fibre reinforced polymer moulds shall be used for achieving localized architectural finishes.

5.5.9 Moulds shall be coated with mould release agents that permit its release without damaging or staining the concrete and without affecting subsequent coating, painting, or caulking operations. Debonding agents or wax shall be used during match casting.

5.5.10 Whenever concrete is to be painted or stained, only mould release agents compatible with the coating shall be permitted, unless surface preparation is performed to ensure good adhesion between the coating/stain and the concrete.

5.6 Hardware, Lifting Devices and Miscellaneous Materials

5.6.1 *Hardware*

5.6.1.1 Hardware refers to items:

a) To be placed on or in the structure to receive the precast concrete units (anchor bolts, angles, or plates with suitable anchors);

b) To be embedded in the concrete units for connection or for other mechanical, plumbing, glazing, miscellaneous iron, masonry, roofing works, etc; and

c) Necessary for the installation of the precast concrete units.

5.6.1.2 Hardware shall be made from ductile materials or as specified in the design. For high-strength materials, the design must ensure elastic behaviour. For testing, 12 shall be referred.

5.6.1.3 Welding of steel plates, angles, and other shapes shall be carried out by certified welders in conformance with standard procedures. The size, length, type and location of all welds shall conform to the approved shop drawings, and no unspecified welds shall be added without the approval of the designer.

5.6.1.4 Electrodes used for welding operations shall be bought in sealed (airtight) containers. The selection of a suitable electrode shall be made based on the material to be welded, intricacies based on the geometry of the job, thickness, weld profile and weld position. The maintenance of any electrode used shall be as prescribed by the manufacturer.

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5.6.1.5 All metallic hardware surfaces that are exposed to or within 12 mm of concrete surfaces that are exposed to the weather, corrosive conditions, or condensation preferably be protected against corrosion or be made of non-corrosive materials. Hardware shall be properly cleaned before application of protective treatment. Corrosion protection shall be done as required. Care shall be taken to prevent chemicals, such as acids, from coming in contact with the hardware and causing corrosion.

5.6.1.6 Supplier of hardware shall furnish records of compliance to specification requirements and mill certificates for the material used. Review of hardware fabrication shall be performed by quality control personnel.

5.6.2 *Lifting Devices*

5.6.2.1 Lifting devices shall be capable of supporting the element in all positions planned during manufacture, storage, delivery and erection. Safe loads for lifting inserts or hooks or devices shall be supplied by the manufacturer of proprietary devices. Information on the use and installation of the devices to ensure proper performance shall be supplied by the manufacturer. For factor of safety and capacity of lifting inserts or hooks, 7 and 8 shall be referred, respectively. For testing, 12 shall be referred.

5.6.2.2 If the lifting device is subjected to dynamic loads, it shall preferably be fabricated using ductile materials or shall be designed to remain elastic.

5.6.2.3 Reinforcing bars provided as main reinforcement in the members shall not be used as lifting hooks. Separate reinforcing bars of required grade, bent to the correct size and shape, may be used exclusively as lifting hooks, with adequate embedment or anchorage whining the precast member. Where necessary, smooth rebars with adequate embedment length shall be used as lifting hooks. Lifting hooks made of reinforcing bars shall not be used for multiple handling applications.

5.6.2.4 Each bar size and configuration shall be considered such that it meets load and handling requirements.

5.6.2.5 Coil rods and bolts shall not be welded when used in lifting operations.

5.6.2.6 Connection hardware shall not be used for lifting or handling unless carefully reviewed and approved by the site engineer/designer.

5.6.2.7 Shop drawings shall clearly define insert dimensions and locations for fabrication and placement or refer to standard details.

5.6.2.8 Members incorporating embedded hardware shall be provided with corrosion protection during long-term storage.

5.6.3 *Surface Finishes and Coating*

5.6.3.1 All surface finishes and coating on precast concrete units shall be as stated in the shop drawing.

5.6.3.2 When specified in the project requirements, for finishing processes such as sandblasting, acid etching, surface retardation, the use of mould liners, or the application of veneer facing materials may be employed to achieve the desired surface appearance. These methods shall help control form joints, form blemishes and air voids within acceptable limits.

5.6.3.3 Based on project requirements and client specifications, curing compounds for precast elements shall be chosen for compatibility with coatings, moisture retention, and ease of removal. Only membrane-forming compounds that meet standards for water retention and curing efficiency shall be used. Application should occur immediately after demoulding to prevent moisture loss and ensure proper strength. The surface must be free of dust or laitance, and curing must be uniform to avoid streaking. Curing compounds shall be non-residual or removable when subsequent coatings are applied.

5.6.3.4 Based on project requirements and client specifications, anti-carbonation coatings shall be selected for CO_2 diffusion resistance, breathability and UV stability, conforming to durability standards. They must be tested for permeability and adhesion on precast surfaces. Application should occur on clean, dry, fully cured concrete, free from laitance, dust and form oil. Coatings must be applied evenly to avoid streaks and ensure effective protection.

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5.6.3.5 Based on project requirements and client specifications, paints used on precast elements shall be alkali-resistant, weatherproof, and compatible with the concrete substrate. Selection shall be based on exposure conditions, including moisture, UV and chemical resistance requirements. Before application, the surface must be properly primed and any efflorescence or moisture-related defects must be addressed to prevent peeling or discoloration. Roller or spray application shall ensure uniform coverage and multiple coats shall be applied as required to achieve the desired opacity and aesthetic consistency.

5.6.3.6 Based on project requirements and client specifications, epoxy coatings, selected for chemical resistance, adhesion, impact resistance and fire rating, are used for flooring, structural connections and corrosion protection. Surfaces must be clean and dry, with a primer applied for better bonding. Curing conditions should be controlled, and components must be mixed correctly and applied evenly to avoid uneven gloss or discoloration.

5.6.3.7 Based on project requirements and client specifications, polyurethane or hybrid sealants for external precast joints shall be compatible with concrete, offer high elongation (for grouted joint 200 percent or more, for non-grouted joints 400 percent or more) and resist UV and environmental degradation. Selection should consider joint movement, exposure and adhesion. Joints must be clean, dry and free of contaminants before application, with a primer used if needed. Proper tooling is essential to ensure seamless finishes and prevent air pockets. Sealants in grouted joints act as moisture barriers, so their lifespan shall be verified.

5.6.4 *Insulation Material*

5.6.4.1 Rigid foam insulation for precast elements shall be selected based on thermal resistance, compressive strength and moisture resistance. Expanded polystyrene, extruded polystyrene, polyurethane or polyisocyanurate, etc, are preferred for long-term efficiency. Panels must conform to accepted standard [C-7(14)] or relevant manufacturer guidelines for density, strength and thermal conductivity. Panels shall be precisely cut for a snug fit to minimize thermal bridging. Exposed foam surfaces must be protected with coatings for durability and UV resistance. Foam with strength more than 10 MPa or as per design may be used in flooring to reduce concrete weight and thickness.

5.6.4.2 Mineral wool insulation shall be selected for its fire resistance, acoustic properties, and vapour permeability. Rigid or semi-rigid panels shall be used to withstand the casting process without compression. Mineral wool products conforming to accepted standard [C-7(15)] or any relevant manufacturer's guidelines shall possess thermal conductivity, density, moisture absorption and fire resistance requirements. Before placement, mineral wool boards shall be dry, securely positioned and properly supported to prevent displacement when concrete is poured. Special care shall be taken to prevent fibre shedding, which can affect worker safety and surface finish. If exposed, mineral wool shall be coated or encased to prevent moisture absorption and fibre degradation over time.

5.6.4.3 Lightweight concrete used for insulation layers in precast elements shall be selected based on its density, thermal performance and compatibility with structural concrete. Autoclaved aerated concrete and non-autoclaved cellular concrete conforming to accepted standard [C-7(16)] shall be used or relevant manufacturer's guidelines shall be followed. Proper mix design and curing conditions shall be maintained to achieve uniform density and minimize shrinkage or cracking. During production, the interface between lightweight and normal concrete layers shall be well bonded to prevent delamination. Surface smoothness and uniformity shall be checked, as inconsistencies may affect finishing and overall insulation performance.

5.7 Connections

In addition to the requirements in 5.6.1, the materials of a connection shall be selected such that embrittlement of any part of the assembled connection do not occur. Non-ferrous inserts shall be resistant to electrolytic action and alkali attack. If more than one material is used in a connection, abutting materials shall be selected such that corrosion is not induced. Dissimilar metals shall not be embedded near or in direct contact in moist or saturated concrete unless it is ensured that detrimental chemical or electrochemical (galvanic) reactions do not occur. For continuity of rebar, grouted coupler, grout filled tube or lapping of rebar shall be used. However, depending on seismic requirements the type of continuity method shall be selected. For testing and performance evaluation 12 shall be referred.

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5.7.1 *Wire Loop System — Loop Box*

5.7.1.1 The wire loop system (proprietary product) is a load-transferring, cast-in element consisting of a box and a U-shaped wire loop for connecting prefabricated concrete elements. Generally, the joint with wire loop boxes is considered non-structural and should only be used for durability and gravity load requirements.

5.7.1.2 Relevant manufacturer's guidelines shall be used for selecting the appropriate product. In addition to that, the following points shall be considered:

a) The box and the two flexible wire loops shall be made of carbon steel sheet and high strength wire ropes, respectively. The surface of the box shall be equipped with a profiled surface to ensure sufficient bonding with the surrounding concrete;

b) Proper selection involves ensuring compliance with tensile strength requirements and alignment accuracy to prevent misfits. High-strength, non-shrink non-metallic cementitious grout is recommended to encapsulate loops fully. Care shall be taken in application to maintain uniform joint appearance, especially in exposed architectural elements;

c) The looped wire ropes shall not react with grout and form corrosion. Also, shall not be used if found having corrosion; and

d) The manufacturer shall furnish the detailed technical datasheet along with the test results corroborating the values noted in the technical datasheet. Also, the same shall be verified by the engineer-in-charge by conducting appropriate testing per batch.

5.7.2 *Grouted Coupler*

5.7.2.1 Grouted couplers (proprietary product) provide mechanical splicing of rebars between precast elements, ensuring structural continuity.

5.7.2.2 Their selection shall consider compatibility with reinforcing bar sizes and compliance with applicable standards.

5.7.2.3 Reinforcing bar ends and coupler interiors shall be clean before grouting to ensure proper bonding. A high-performance cementitious grout, characterized by non-shrink non-metallic high strength properties, high early strength shall be used to ensure effective load transfer. The grout shall meet the specified compressive strength and flowability requirements for complete void filling. Misalignment or incomplete grouting shall be addressed through inspection to maintain both functionality and appearance.

5.7.2.4 Class H coupler as per accepted standard [C-7(10)] shall be used with appropriate testing as per **12.3.1**.

5.7.3 *Grout Filled Tubes* (*Dowel Tube*)

5.7.3.1 The tubes shall be corrugated to have proper bond with concrete and grout.

5.7.3.2 The length of the tube shall be as per development length or design requirements as per **7** and **8**.

5.7.3.3 The tube shall be free from oil, grease or loose particles prior to installation and grouting. A high-performance cementitious grout, characterized by non-shrink non-metallic high strength properties, high early strength shall be used to ensure effective load transfer. The grout shall meet the specified compressive strength and flowability requirements for complete void filling. Adequate curing time shall be allowed for strength development. Grout shall be placed using gravity feed to ensure complete filling and venting shall ensure proper air escape. Misalignment or incomplete grouting shall be addressed through inspection to maintain both functionality and appearance.

5.7.4 *Terminators* (*Headed Bars*)

5.7.4.1 Headed bars are used in precast concrete elements to enhance anchorage efficiency by providing an enlarged head at the end of the reinforcement bar, reducing the required development length.

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5.7.4.2 These bars shall be manufactured from high-strength reinforcement steel to ensure adequate ductility and mechanical performance. The heads shall be formed through welding, forging, or mechanical attachment and shall comply with the specified geometric and strength requirements. The selection of headed bars shall be based on design specifications, including grade of bar, head dimensions and load transfer capacity.

5.7.5 *Metal Inserts*

5.7.5.1 Steel plates shall be cast into precast elements to facilitate connections with other structural components for non-structural or gravity load transfer. Plates shall be made from structural-grade steel with appropriate thickness, positioned accurately during casting and securely anchored to prevent displacement.

5.7.5.2 Welded studs shall provide reliable load transfer by creating strong mechanical bonds between steel inserts and concrete. Studs shall be manufactured from high-strength steel and shall conform to applicable material and welding standards. Proper fusion and alignment shall be ensured and weld quality shall be verified through inspection to prevent structural deficiencies.

5.7.5.3 Corrosion protection, such as galvanization or stainless-steel alternatives, shall be applied based on environmental exposure conditions. Protective measures shall not interfere with mechanical performance or bonding with concrete. In exposed applications, surface finishes shall be such as to maintain aesthetic quality. Fire safety requirements shall also be verified for exposed applications.

5.7.6 *Shims*

5.7.6.1 Plastic shims shall be composed of materials that provide sufficient compressive strength (not less than 10 MPa or as required by the design) and demonstrate resistance to moisture, UV exposure, and temperature variations. They shall be dimensionally stable, free from warping or defects and sized to ensure a precise and secure fit for their intended application. Plastic shims shall be used for thicknesses up to 40 mm or less.

5.7.6.2 Metal shims shall be used for any shim thickness; however, use is preferable for thicker shims (typically greater than 40 mm). These shims shall be having higher strength and load-bearing capacity, particularly where greater stability and structural support are needed. For multi-layer shims, tack welding shall be performed between the layers to ensure a strong bond and prevent shifting or separation of the individual layers. Metal shims that are exposed to the environment or are in contact with moisture or corrosive elements shall be provided with appropriate corrosion protection. Care must be taken during installation to ensure that the corrosion protection is intact and not compromised by any mechanical processes.

6 PRECAST CONCRETE SYSTEMS

6.1 General

In precast concrete construction, the design shall address additional requirements including selection of component types, appropriate connection detailing, structural continuity, flexibility and restraint. Structural connections between elements shall be explicitly classified and designed as pinned, rigid, or ductile based on seismic performance criteria. Horizontal elements such as slabs, beams and staircases shall be detailed as simply supported or made continuous using in-situ topping, integrity reinforcement and collector elements to provide diaphragm action for lateral load transfer to the lateral force resisting system (LFRS). Vertical elements (for example, columns and walls) shall be designed as continuous members unless specifically excluded from LFRS. Beam-to-column joints not forming part of LFRS shall be designed to resist gravity loads with compatible displacements, moments based on design requirements and erection eccentricity. Such frames shall incorporate structural bracing to resist lateral forces. For selection of precast concrete system 6.2 to 6.5 shall be referred.

6.2 System Considerations

6.2.1 *Structural Concept*

Precast concrete structural systems shall be explicitly designed to ensure continuity between elements and robustness of the overall system. The selected system shall be robust and shall be designed to withstand the following:

a) Stability against progressive collapse;

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b) Prevention of structural failure under all design load combinations;

c) Fire resistance as per applicable regulations;

d) Adequate ductility and energy dissipation capacity;

e) Redundancy and continuity in load paths;

f) Control of cracking within serviceability limits;

g) Acceptable deformation within serviceability limits; and

h) Durability against environmental exposure.

6.2.1.1 The structural system and each of its components shall maintain stability during all stages, including transportation, erection and service life. The following factors shall be considered for the selection of precast system:

a) Grid distances;

b) Spanning direction of slab and beam;

c) Position of columns and walls;

d) Position of load bearing walls and/or facade and/or facade at stilt;

e) Position and approximate requirements for stabilizing elements; and

f) Position of expansion joints.

6.2.1.2 The appropriate precast structural system shall be selected based on Table 1, considering building usage, architectural constraints, seismic performance requirements and constructability. The response reduction factor (R) shall be considered as per **5** of Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS.

6.2.2 *Standardization of Precast Components/Elements*

The precast components/elements shall preferably be standardized based on the structural, architectural and/or aesthetic requirements. The precast components/elements given below but not limited to, shall be used in accordance with relevant Indian Standards (*see* Part B 'Building Materials' of this NBCS) and the accepted standards, where available:

a) Reinforced/prestressed concrete channel unit;

b) Reinforced/prestressed concrete slab unit;

c) Reinforced/prestressed concrete beams;

d) Reinforced/prestressed concrete columns;

e) Reinforced/prestressed concrete hollow core slab;

f) Reinforced concrete waffle slab/shells;

g) Reinforced/prestressed concrete wall elements;

h) Hollow/solid concrete blocks and battens;

j) Precast planks and joists for flooring and roofing;

k) Precast joists and trussed girders;

m) Light weight/cellular concrete slabs/wall panels;

n) Precast lintels and *Chajjas*;

p) Large panel prefabricates;

q) Reinforced/prestressed concrete trusses;

r) Reinforced/prestressed roof purlins;

s) Precast concrete L-panel units;

t) Prefabricated concrete double-T units;

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u) Prefabricated sandwich concrete panels;

v) Precast concrete foundations; and

y) Precast concrete staircases.

Any other type of components shall be used with approval of the Competent Authority and appropriate testing.

Table 1 Precast Concrete (PC) Buildings with Different Structural Systems

(*Clause* 6.2.1.2)

Sl No.	Structural System
(1)	(2)
i)	*Building with Precast Concrete Moment Frames*: a) Buildings with single or multi-storey cantilever columns with simply supported beams (*see* Note 1) b) Building with ordinary moment resisting frames (OMRFs) (*see* Note 2) c) Buildings with special moment resisting frames (SMRFs)
ii)	*Buildings with Precast Concrete Structural Walls*: a) Ordinary structural walls (*see* Note 2) b) Ductile structural walls
iii)	*Buildings with Precast Concrete Moment Frames and Precast/RC Structural Walls* (*Dual System*) (*see* Note 3): a) Buildings with ordinary structural walls with OMRFs (*see* Note 2) b) Buildings with ordinary structural walls with SMRFs (*see* Note 2) c) Buildings with ductile structural walls with OMRFs (*see* Note 2) d) Buildings with ductile structural walls with SMRFs
iv)	*Precast Cell System*: a) Ordinary precast/RC structural wall system (*see* Note 2) b) Ductile precast/RC structural wall system

NOTES

1 Precast single-storey or multi-storey columns with simply supported beams shall be designed using a response reduction factor (R) of 2, as this system permits only one mode of plastic hinge formation.

2 Precast/RC emulative structures in Seismic Zones III and above shall be designed to be ductile. Hence, precast ordinary moment resisting frame system (OMRF) is not allowed in these seismic zones.

3 Buildings with precast RC structural walls also include buildings having precast RC structural walls and precast RC moment frames, but where:

a) frames are not designed to carry design lateral loads, or
b) frames are designed to carry design lateral loads, but do not fulfil the requirements of 'dual systems'.

4 Various types of moment resisting frame systems are explained in 6.3.4, the structural designer shall select the appropriate system based on the requirements of the structure.

5 For all structures, precautions shall be taken to ensure continuity of load path, adequacy of connections, and compliance with drift limitations.

6 For shear walls with coupling beams, CIS coupling beams shall be used.

7 For elements designed to transfer gravity loads and not part of LFRS, assemblage testing shall be carried out to verify the failure mode of gravity frame elements and to ensure compliance with displacement requirements (equivalent to 'R' times the design storey drift), also the performance criteria with an approval from institute of repute may be verified.

8 The use of precast structural systems in Seismic Zone IV and above shall be permitted subject to approval by institute of repute. The adequacy of such systems shall be established through testing and/or appropriate structural analysis, as deemed appropriate.

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6.3 Basic Precast Concrete Systems

Precast concrete building systems are composed of some basic types of structural systems. These systems shall be combined in various ways to obtain a suitable and effective structural concept that meets the needs of specific buildings. The most common systems are:

a) Precast concrete buildings with moment frames;

b) Precast concrete buildings with structural walls;

c) Precast concrete buildings with moment frames and structural walls (or dual system);

d) Precast concrete buildings with precast or cast-in-situ flat slab and precast concrete moment frames and precast concrete structural walls; and

e) Cell/3D system.

6.3.1 The above list is not unique as many combinations are possible to achieve the same objectives, such as the use of arches and rigid portal frames.

6.3.2 When facade systems act as load-bearing walls, their design shall explicitly consider lateral stability. For non-load-bearing facades, beam deflection and frame movement shall be accommodated in the connection detailing to avoid unintended cracking or overstressing of facade panels.

6.3.3 Where partial fixity is introduced by structural topping or dowels, the resulting flexural stiffness (EI) may be considered in serviceability limit state checks. The contribution of topping shall be analytically substantiated.

6.3.4 *Skeletal Frame Systems*

Skeletal frame systems composed of beams and columns shall be used where architectural flexibility, large spans, or column-free space is required, including in commercial, educational, healthcare and industrial facilities. The following skeletal systems shall be permitted:

a) Moment resisting frames; and

b) Moment frames with stabilizing cores (with precast structural walls).

Depending on the importance of the structure, structural framing system, connection detailing requirement and the level of ductility demanded by the building, the precast beam to column or beam to beam connection shall be selected from the five types as shown in Fig. 1. Type 1 and Type 2 (with minor fixity) shall be adopted with cantilever with single or multi-storey columns with simply supported beam. Types 2, 3 and 4 with detailing as per the good practice [C-7(2)] shall be adopted with OMRF. Types 2, 3, and 4 with detailing as per the good practice [C-7(1)] and detailing of connection as per this NBCS shall be adopted with SMRF. In the case of Type 3 and Type 4, the location of beam-to-beam connection shall be selected such that plastic hinge formation zones are avoided (at least more than 0.15 L_{beam}). In the case of secondary beams or short length beams, the location of the connection can be at the mid span with appropriate detailing.

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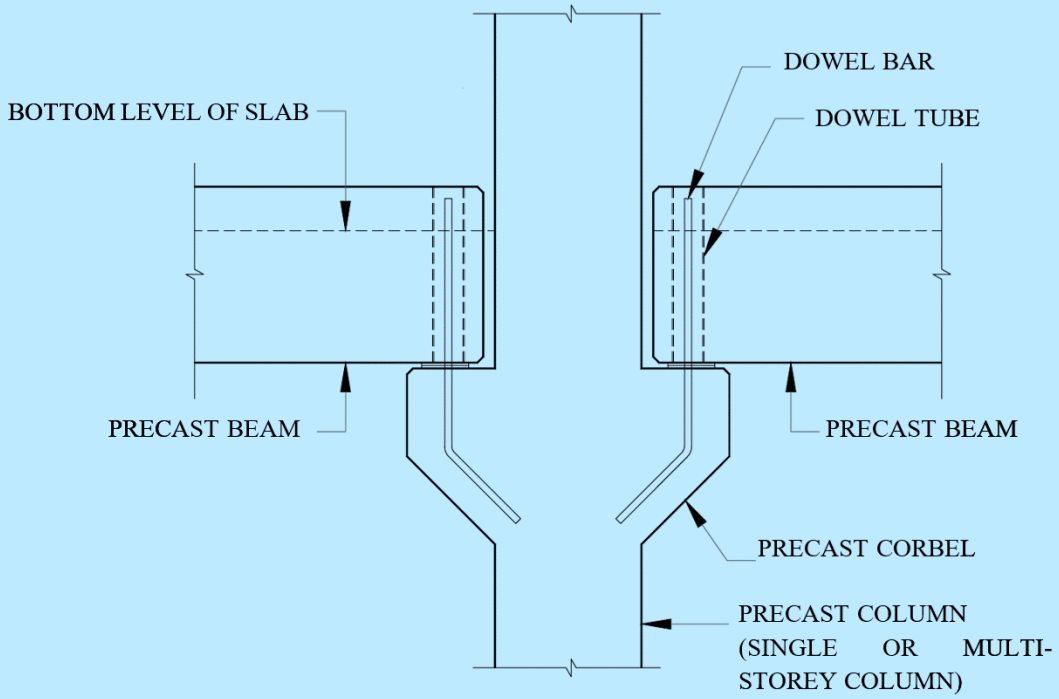
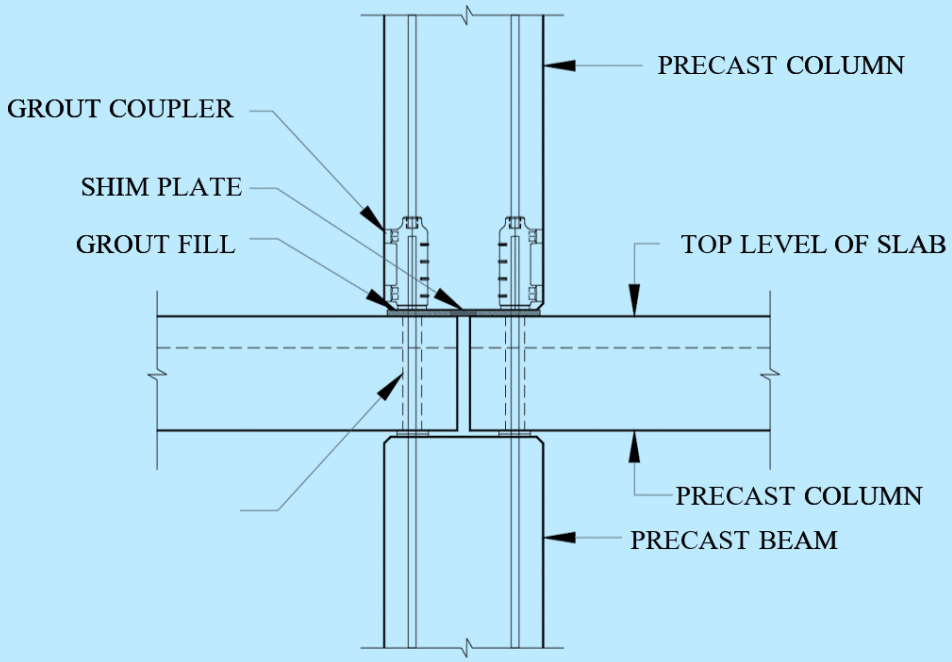


1A) TYPE 1



1B) TYPE 2

FIG. 1 PRECAST BEAM-COLUMN CONNECTION (CONTINUED)

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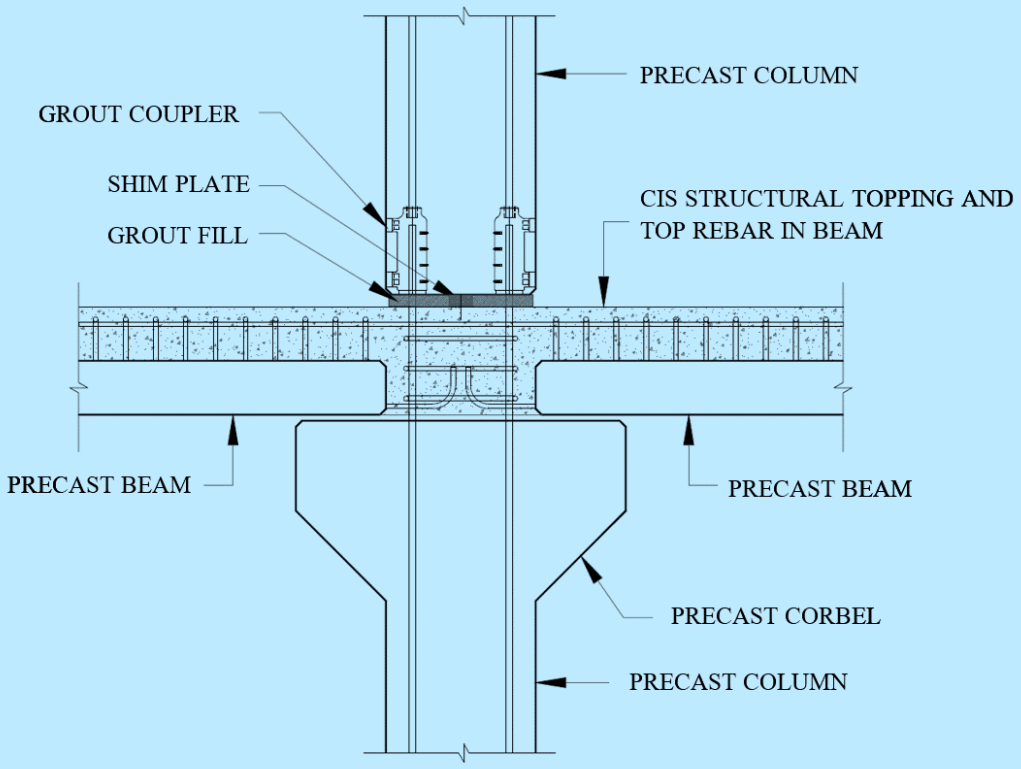
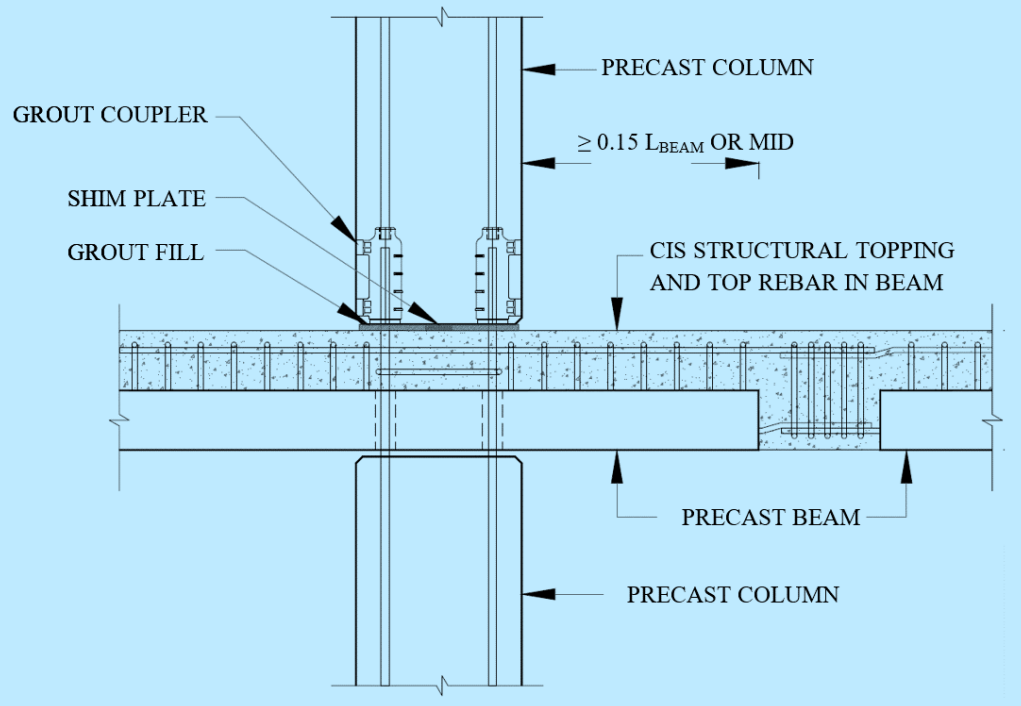


1C) TYPE 3



1D) TYPE 4

FIG. 1 PRECAST BEAM-COLUMN CONNECTION (CONTINUED)

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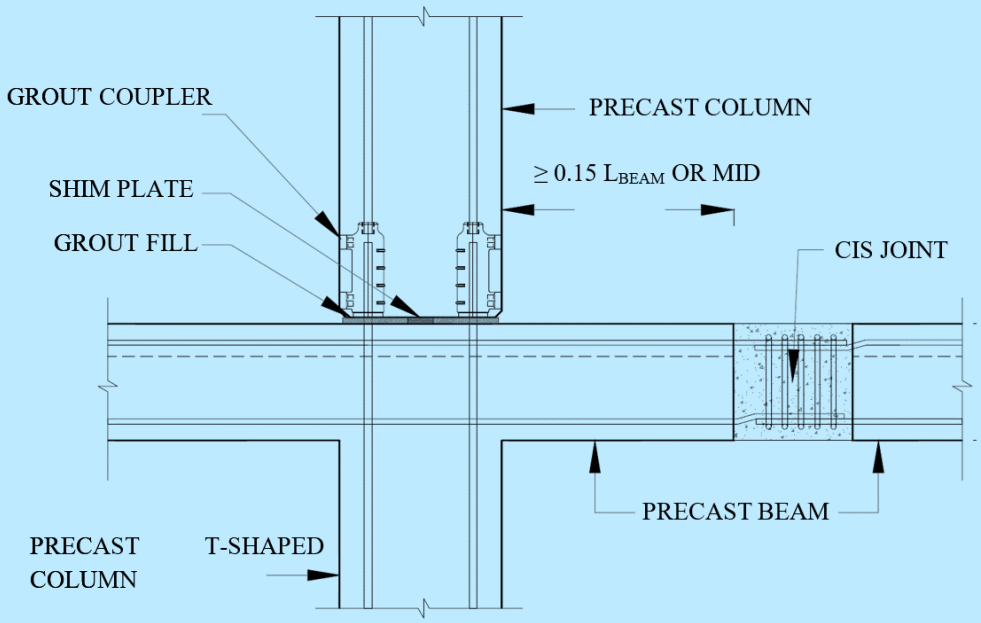

FIG. 1 PRECAST BEAM-COLUMN CONNECTION

6.3.4.1 *Moment resisting frames*

Rigid beam-column connections shall be formed using Type A, Type B, or Type C systems, depending on the required ductility level and building importance. All systems shall include structural topping for diaphragm action in accordance with 6.3.6:

a) *Type-A frames* — Type A frames shall consist of precast beam units between columns/column corbels, where columns can be cast-in-place or precast (*see* Fig. 2). These shall be achieved by adopting either one or a combination of the following arrangements:

   1) System A1 — This system consists of single or multi-storey columns fixed at the base, supporting simply supported beams on corbels or directly above the columns. The slab units shall be simply supported between the ledges or above the beams;

   2) System A2 — This system consists of single storey columns fixed at the base with provision of seating notch or temporary supports or permanent corbels for beams. Depending on design requirements, beams and slabs shall be designed as simply supported for temporary loading and made continuous for permanent loading;

   3) System A3 — This system consists of single or multi-storey columns fixed at the base and staggered joints between the columns at other grid levels with corbels to support the beams. Staggering of columns shall be arranged such that only 50 percent of all columns are spliced at any given storey level. Depending on design requirements, beams and slabs shall be designed as simply supported for temporary loading and made continuous for permanent loading; and

   4) System A4 — This system consists of 3 or 4 storeys columns fixed at the base with corbels to support the beams. Depending on design requirements, beams and slabs shall be designed as simply supported for temporary loading and made continuous for permanent loading.

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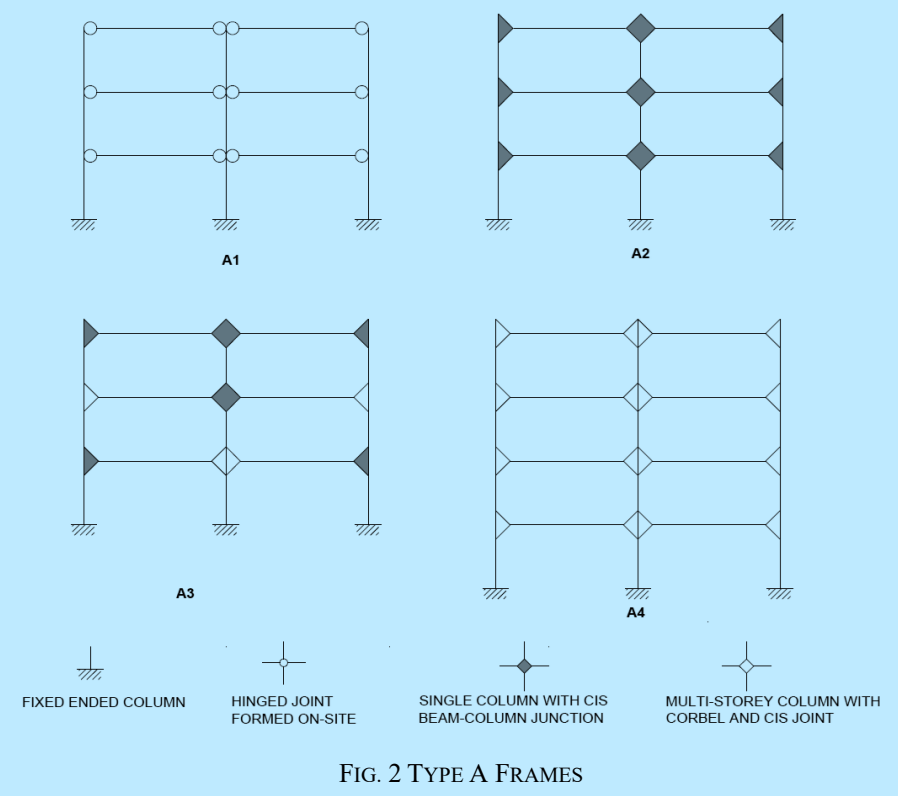

FIG. 2 TYPE A FRAMES

b) *Type-B frames* — Type B frames shall consist of precast beam units passing through columns (*see* Fig. 3). These shall be achieved by adopting the following arrangement:

1) System B1 — Columns shall be 1 storey high and spliced at each floor level. Precast beams run through the columns and are spliced at point of contraflexure. It shall be ensured that beam splices are avoided at end spans. Slab units shall be designed as simply supported.

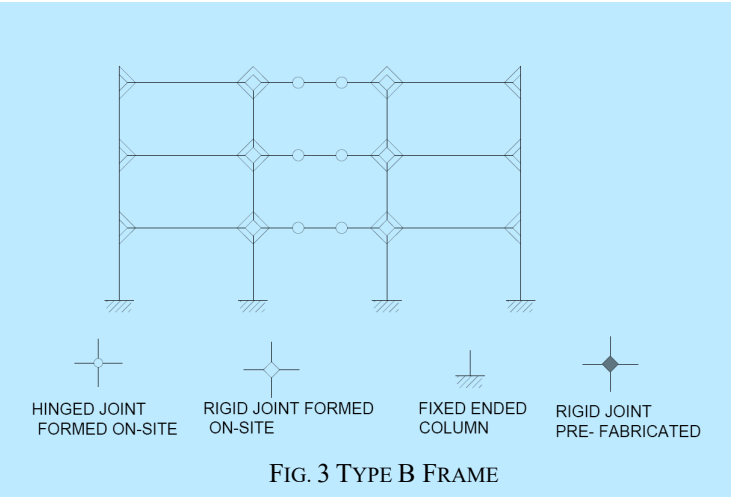

FIG. 3 TYPE B FRAME

c) *Type-C frames* — Type C frames (*see* Fig. 4) shall consist of monolithically fabricated beam column units such as cruciform, T/L-units, H-frames, portal frames, etc. These shall be achieved by adopting either one or a combination of the following arrangements:

1) System C1 — Single storey portal frames shall be placed alternately and simply supported at floor levels. Orthogonal beams and beams connecting the portal shall be simply supported or continuous based on the design requirements. Slab units shall be designed as simply supported; and

2) System C2 — L-shaped and T-shaped frame units shall be placed simply supported on the base and on top of each other. Hinged or moment resisting connections shall be made between frame units at the mid-span of beams. Slab units shall be designed as simply supported.

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3) System C3 — H-shaped frames with a cantilever beam shall be provided with rigid connection at the junction of beam and column. Hinge or moment resisting connections shall be provided at the mid-height of columns and centre of beams. Slab units shall be designed as simply supported.

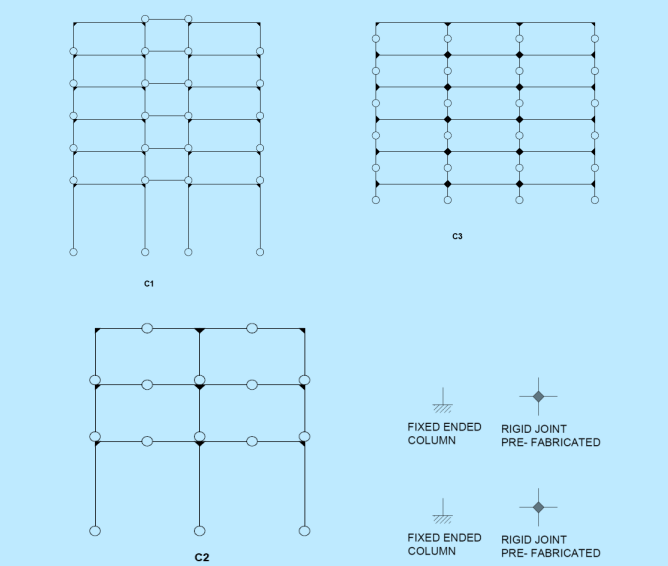

FIG. 4 TYPE C SYSTEMS

6.3.4.2 *Moment frames with structural wall*

In this system, lateral stability shall be achieved through structural walls, structural cores, or bracing. Beam-column and column-base connections shall be designed as pinned or rigid in accordance with the required ductility and seismic performance level (*see* Fig. 5).

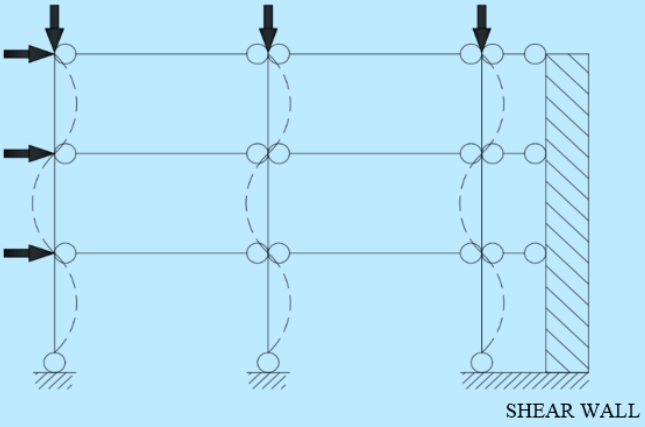

FIG. 5 DUAL SYSTEM

6.3.5 *Wall Systems*

Wall systems in precast construction shall be classified on the basis of the design of connections between individual concrete panels, which when joined together form the structural wall (*see* Fig. 6). The types of emulative wall systems shall be as follows:

a) Wall systems without shear transferring vertical joints; and

b) Monolithic wall systems.

These classifications shall guide the detailing of connections and the overall performance of the system under gravity and lateral loads.

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6.3.5.1 For systems without shear transferring vertical joints, the horizontal joints shall be designed to exhibit ductile behaviour. Vertical joints shall not be relied upon for structural continuity and shall be detailed to allow for movement or minimal shear capacity to be provided. Each panel shall be designed as an independent unit for gravity and lateral loads.

6.3.5.2 For monolithic wall systems, horizontal joints shall be detailed to permit ductile behaviour. Vertical joints shall be designed as strong connections with sufficient capacity to remain elastic under design-level lateral loads, preserving global ductility.

6.3.5.3 For jointed wall systems, both vertical and horizontal joints shall be detailed to provide ductile behaviour. Connections shall be capable of undergoing inelastic deformation with energy dissipation, contributing to system-level ductility.

6.3.5.4 Combination of monolithic and systems without shear transferring vertical joints shall be adopted based on the performance requirements of the building.

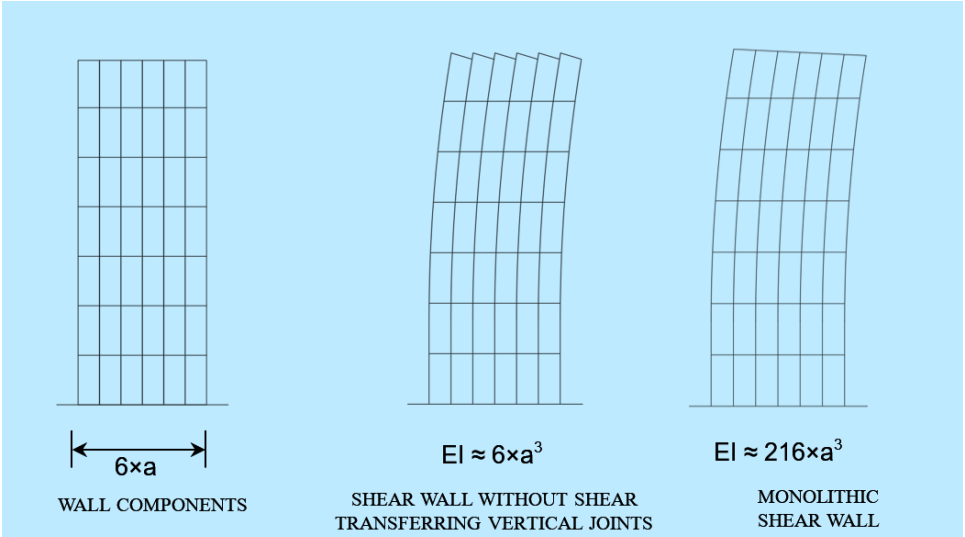

FIG. 6 SHEAR WALL BEHAVIOUR DUE TO CONNECTIONS

6.3.6 *Floor/Roof Systems*

Precast floor and roof systems shall be designed to function as horizontal diaphragms capable of transferring in-plane shear and distributing lateral loads to the lateral force resisting system (LFRS) under all applicable load combinations.

6.3.6.1 A minimum structural topping of 75 mm with reinforcing mesh or welded wire mesh shall be used in Seismic Zones III and above which can be reduced to 50 mm in Seismic Zone II. The diameter of re-bars used in both reinforcing mesh and welded wire mesh shall not be less than 6 mm. Welded wire mesh shall be avoided in Seismic Zones III and above. The minimum structural topping thickness shall be measured above the slab top level even at the highest camber location. Where service conduits of larger diameter are provided, the thickness of structural topping shall be such that a minimum cover of 25 mm is maintained above the conduit. The structural topping shall be provided on the entire floor.

6.3.6.2 Precast floor and roof systems shall be selected from the types defined in **6.3.6.3** to **6.3.6.6**, based on span length, loading and diaphragm requirements**.**

6.3.6.3 *Hollow-core slab* (*HCS*) *system*

This system is made up of prestressed precast concrete slabs with hollow cores (*see* Fig. 7). These systems may be used for multi-story buildings with moderate spans. Structural topping shall be provided to ensure diaphragm action. The top surface of the slab shall be intentionally roughened to amplitude more than 6 mm. The

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individual HCS units shall not be considered for diaphragm force transfer:

a) The thickness of hollow core slabs varies depending on the span and load requirements, but typically range from 150 mm to 500 mm;

b) Standard width of hollow core slab may be 1 200 mm or 2 400 mm; and

c) Standard dimensions of hollow core slabs shall conform to good practice [C-7(17)].

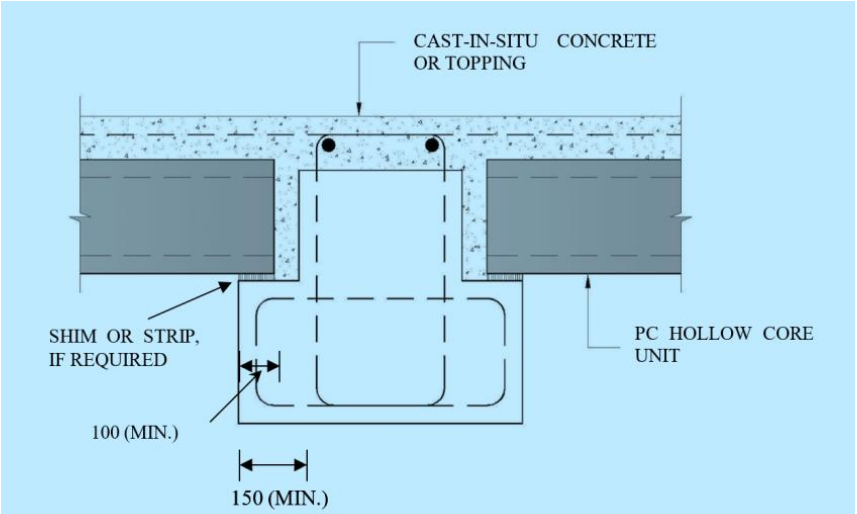

FIG. 7 HOLLOW CORE SLAB

6.3.6.4 *Double tee slab system*

This system is made up of precast concrete slabs with a double tee cross-section. The double tee shape provides greater strength and stiffness than a flat slab, making it suitable for larger spans and higher loads. These shall conform to good practice [C-7(17)]. Structural topping shall be provided to ensure diaphragm action. The double tee units shall not be considered in diaphragm action if the units are not connected for required force transfer, structural continuity and overall stability of the system.

6.3.6.5 *Solid slab system*

Solid precast concrete slab systems shall comprise precast elements that are thinner than equivalent hollow-core slabs. These slabs shall be used in residential or commercial buildings, where individual slab units are intended to span shorter distances.

The slab system shall be considered to act as a structural diaphragm when adequate connections are provided between the slab units and the supporting structural elements and when a structural topping concrete is placed and properly bonded.

In cases where no structural topping is provided, the slab system may still be designed to act as a diaphragm, provided that the connection details ensure adequate transfer of in-plane forces, structural continuity and overall stability of the system.

6.3.6.6 *One-way/two-way composite slab system*

6.3.6.6.1 Composite slab (*see* Fig. 8), also termed as half slab is a precast slab system cast in two stages. The first stage is to manufacture thin precast concrete panels, typically of about 60 mm to 85 mm thick, with the slab's bottom reinforcement and projected shear reinforcement in the form of truss. The projected reinforcement facilitates for multiple point lifting/handling as well as acts as shear connectors for in-situ deck concreting.

6.3.6.6.2 The slab panels shall be designed for overall thickness (precast and in-situ deck) as one-way or two-way in accordance with provisions of **5** of Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS, and Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS.

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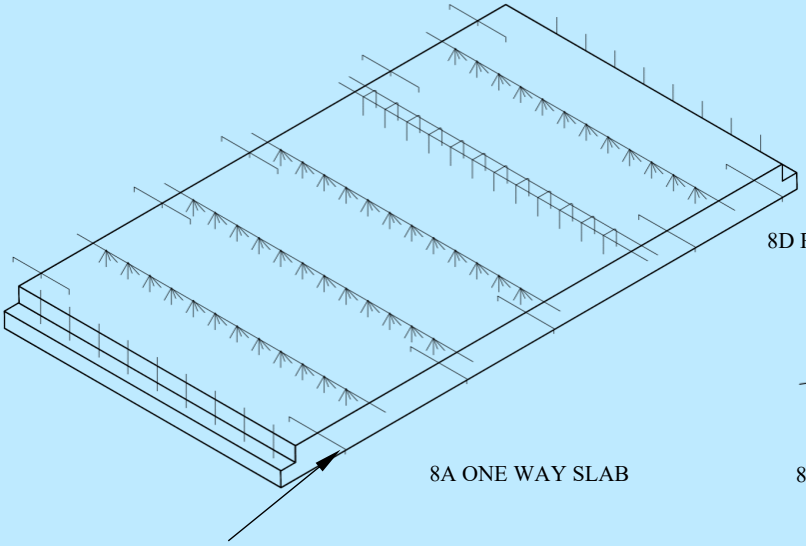
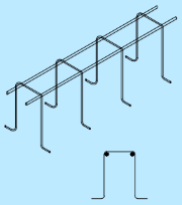
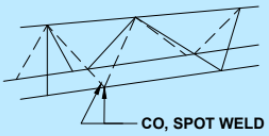


8A ONE WAY SLAB

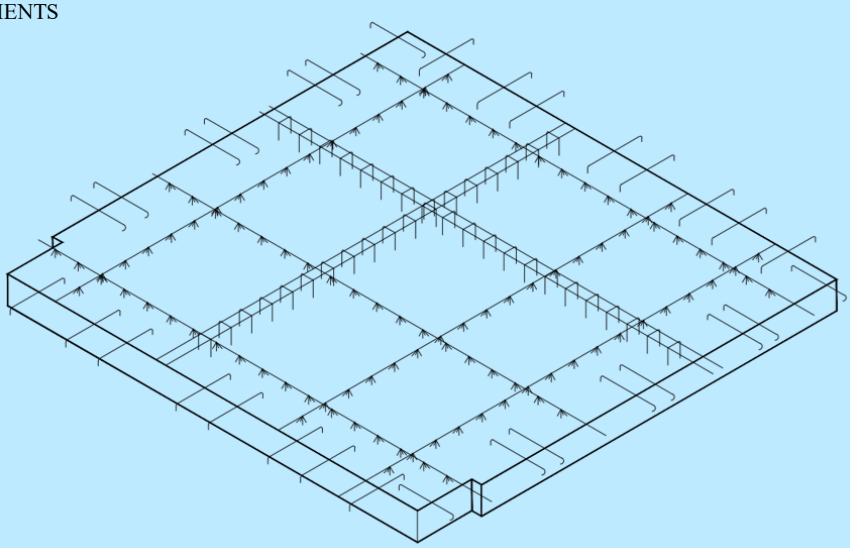


8D RECTANGULAR TRUSS

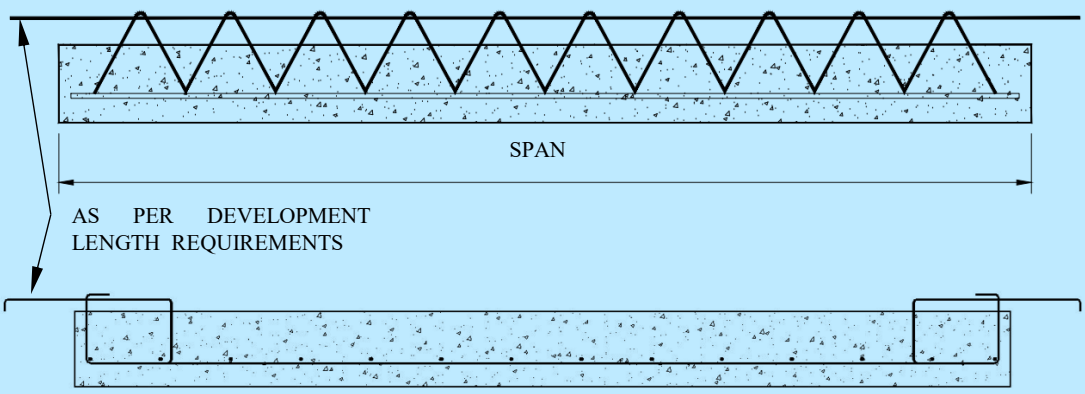


8E TRIANGULAR TRUSS

AS PER DEVELOPMENT LENGTH REQUIREMENTS



8B TWO-WAY SLAB



8C TYPICAL SECTIONAL VIEW

FIG. 8 PRECAST COMPOSITE SLAB PANELS

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6.3.7 *Facade Systems*

Precast facade system shall be made up of following types of non-load bearing cladding panels:

a) Precast insulated concrete panels;

b) Precast solid cladding panels;

c) Architectural precast decorative facade panels; and

d) Precast concrete fins.

All façade connections shall comply with the requirements of the good practice [C-7(18)] and 6.3.2. Design shall account for differential movement, panel dead load and seismic restraint.

6.4 Hybrid System

The following systems are considered in precast hybrid system:

a) Precast concrete seismic structural system (PRESSS) (utilization of bonded or non-bonded prestressing tendons); and

b) Mixed system (utilization of cast-in-situ along with prestressed concrete (PC) members or steel members).

6.4.1 *Precast Concrete Seismic Structural System* (*PRESSS*)

Precast concrete seismic structural system (PRESSS) shall be implemented as an equivalent monolithic system, consisting of precast elements with grouted joints and bonded post-tensioned tendons. The main advantages of post-tensioning are deflection recovery and immediate occupancy of the structure. Two different system of PRESSS are explained in 6.4.1.1 and 6.4.1.2 and application of this system shall be carried out by conducting successful full-scale testing, verification through nonlinear analysis and approvals from competent authorities or institutes of repute.

6.4.1.1 *System* 1 *precast beam elements post-tensioned between columns*

System 1 shall use precast beams post-tensioned between multi-storey columns. Beam length shall be shorter than clear span (20 mm or tolerance requirements, whichever is greater). Post-tensioning and joint grouting shall occur after beam placement (*see* Fig. 9).

6.4.1.1.1 The beams shall be temporarily seated on steel brackets or props. Grout shall then be placed in the gap between beam ends and column faces, followed by post-tensioning operation and grouting of tendons.

6.4.1.1.2 The temporary supports shall be removed once the grout has achieved the required design strength.

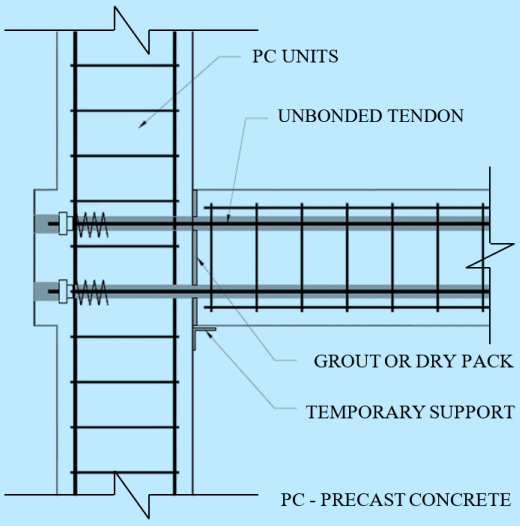

FIG. 9 SYSTEM 1 PRECAST BEAM ELEMENTS POST-TENSIONED BETWEEN COLUMNS

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6.4.1.2 *System 2 supported precast beam elements post-tensioned between columns*

In this system, the beams shall be seated on corbels that transfer gravity loads to the adjacent columns. The beams shall be either built prismatic or with dapped ends, depending on the position of the corbel (*see* Fig. 10) and then post-tensioned.

6.4.1.2.1 The columns in this system shall normally be one-storey in height.

6.4.1.2.2 Columns shall be connected through a mechanical coupler or a grouted steel sleeve.

6.4.1.2.3 This system shall be adopted for buildings designed to demonstrate elastic response, with couplers giving continuity between the column elements, this is utilized for ductile moment resisting frames.

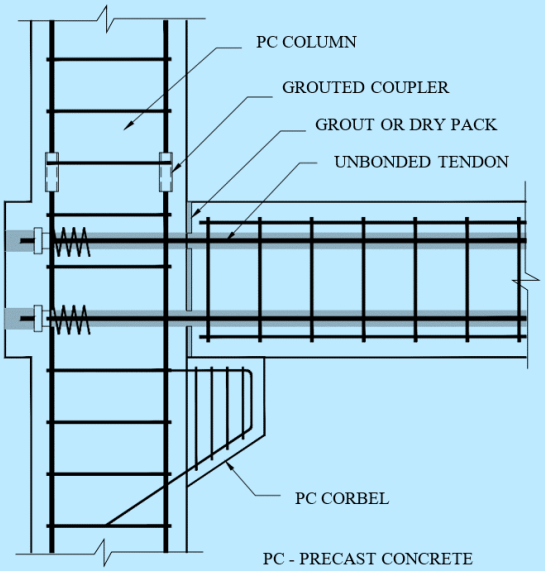

FIG. 10 SYSTEM 2 SUPPORTED PRECAST BEAM ELEMENTS POST-TENSIONED BETWEEN COLUMNS

6.4.2 *Mixed System* (*Cast In-Situ Precast or Structural Steel-Precast*)

This system shall use pretensioned prestressed concrete or reinforced concrete beam shell units or prestressed concrete beams as permanent formwork for cast in-situ beams or structural steel beams (as shown in Fig. 11). Application of this system shall be carried out by conducting successful full-scale testing, nonlinear analysis and approvals from competent authorities or institutes of repute.

6.4.2.1 The precast shells shall typically be left permanently in position after the cast-in-place reinforced concrete core has been cast. The shells can be made from fibre reinforced concrete or reinforced concrete, depending on the design requirements.

6.4.2.2 The precast U-beams or structural steel beams shall be designed to carry self-weight and construction loads and act compositely with the reinforced concrete core when subjected to other loading in the completed structure.

6.4.2.3 Precast U-beams may not be connected by reinforcement to the cast-in-place concrete of the beam or column. If the interfacial shear is found to be not sufficient then shear connector (protruding stirrups or ties) shall be provided between the interfaces.

6.4.2.4 The design shall ensure that the composite beams shall develop plastic hinges during major earthquakes.

6.4.2.5 All precast shell units shall be free of laitance, oil, dust and loose debris before placement of cast in-situ concrete. Construction sequences shall ensure adequate bond between interfaces.

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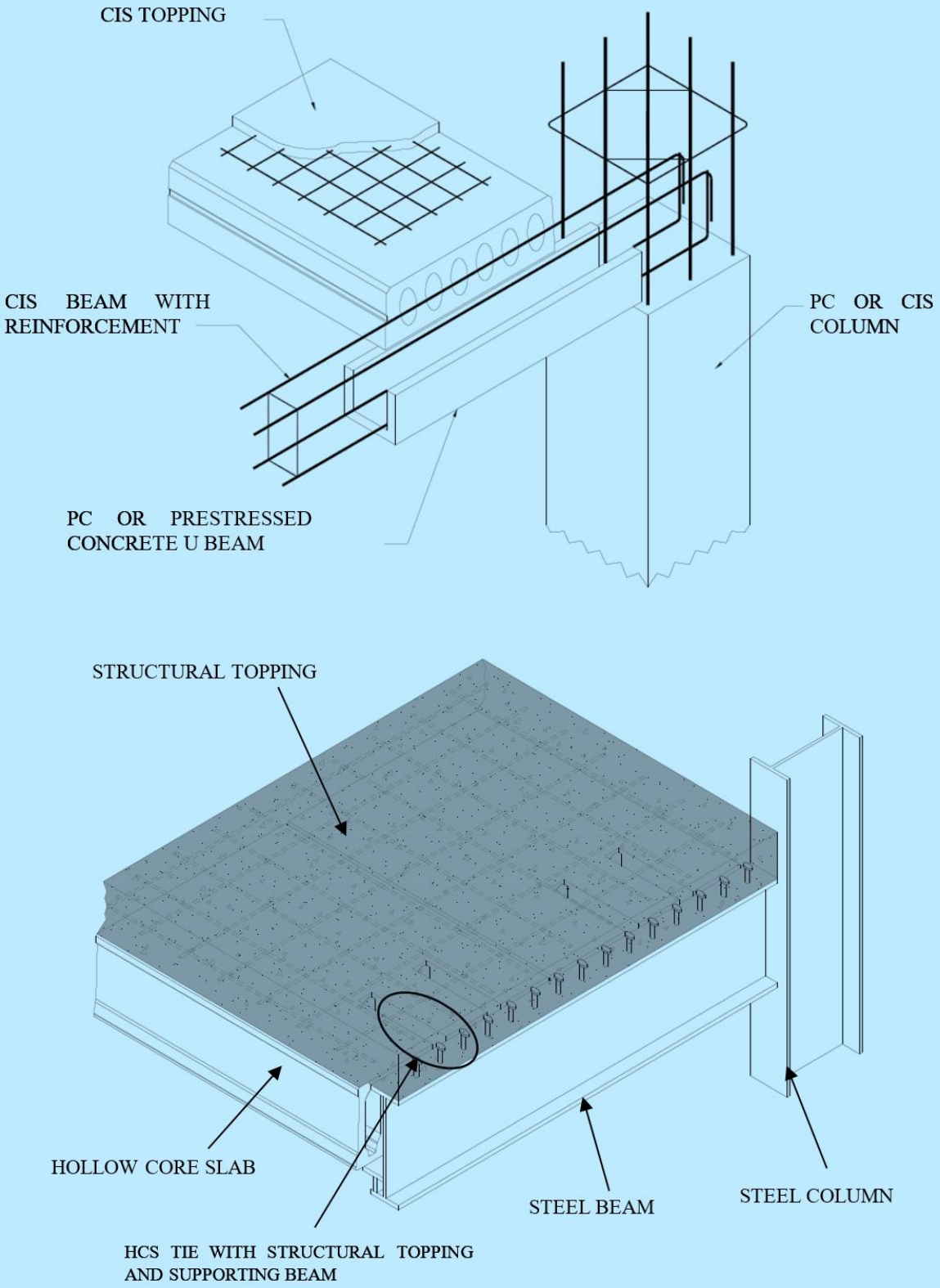

FIG. 11 MIXED SYSTEM

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6.5 Advanced Precast Systems

Advanced precast systems shall include modular 3D precast elements, digitally fabricated units and prefabricated structural modules integrated via design for manufacturing and assembly (DFMA) principles.

6.5.1 *3D Printed System*

3D printed precast systems shall only be used upon approval from the competent authority and shall follow Annex B for design, durability and testing criteria.

6.5.2 *Cell Systems*

Cell systems shall consist of volumetric precast units fabricated as open or closed modules. Connections to surrounding walls and floors shall be verified for strength and serviceability. Annex C shall be referred for further details.

Cell elements for specific area of a structure, such as wet zones (for example, toilets and kitchens), shall be used and integrated with wall and floor systems with appropriate connections.

6.6 Design for Manufacturing and Assembly (DFMA) Method

DFMA-based methods shall conform to Annex D. Designers shall ensure that precast elements are standardized, repeatable and modular to reduce rework and improve assembly efficiency.

7 STRUCTURAL ANALYSIS

7.1 General

Structural analysis shall be carried out to evaluate internal forces and deformations in members and the structural system. The analysis shall verify compliance with strength, serviceability, stability and robustness criteria. These effects include the distribution of forces and moments as well as the calculation of stresses, strains, curvatures, rotations and displacements under both static and dynamic conditions. Advanced analysis methods, such as nonlinear pushover analysis and time history analysis, may be adopted based on project requirements. However, these methods shall be mandatory for irregular structures, critical facilities and structures located in high seismic regions. For sustainable design aspects refer to **G-6**.

7.1.1 All the members and structural system shall be analysed to determine the critical load effects as per methods given in Part C ‘Structural Design, Section 5 Structural Concrete’ and relevant clauses of this Section.

7.1.2 To carry out analysis, the geometry, boundary conditions and behaviour of the structure need to be idealized both for global and local behaviour. The structure is idealized by considering it as made up of elements/ components which can be linear, two dimensional or three dimensional.

7.1.3 Structural analysis shall use methods capable of capturing action effects with sufficient accuracy. These may include classical mechanics or numerical methods such as finite element analysis, provided the model appropriately represents the actual structure.

7.1.4 Precast structures shall be analysed as either emulative systems, where precast performance matches that of monolithic CIS structures, or jointed systems with discrete joint behaviour. Emulative models shall assume monolithic behaviour with joints designed for equivalent force transfer. Lateral force resistance shall be achieved through moment-resisting joints or orthogonally placed structural walls or as per the design requirements.

7.1.5 For precast bearing wall buildings, rotational stiffness of floor-wall and floor-beam joints shall be neglected in structural design. Each component shall be independently designed considering the applicable end conditions, structural loads as specified in 7.2 and the various stages of construction.

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7.1.6 Members shall also be designed to resist handling, erection and impact loads expected during the transportation and erection phases 7.14.

NOTES

1 Jointed construction using welded and bolted connections is not covered in this NBCS. This may be adopted with approval from competent authority and appropriate testing.

2 Rotational stiffness may be accounted for long/short term deflection and serviceability calculations as per 6.3.4, provided that the approach is mutually agreed by the design team and approved by the Authority.

3 Horizontal precast members that transfer gravity loads but are not part of the lateral force resisting system (LFRS) shall be designed assuming pinned end conditions, unless justified otherwise through analysis.

4 Linear model, non-linear model and P-Delta analyses may be used for design, provided the analysis method, assumptions and modelling parameters are clearly defined and justified and approved by the Competent Authority.

7.2 Loads and Forces

The design shall consider all the loading requirements as specified in Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS. Consideration shall also be given to construction loads, any special requirements, such as for plant loads or storage loads and the loading condition should be modified accordingly.

7.2.1 Structural design shall take into consideration the dead loads, live or imposed loads, wind loads, forces such as those caused by earthquake effects, prestressing effects, crane loads, vibration, impact, natural disasters (flood, landslide, etc), blast and effects due to shrinkage, creep, temperature, etc, where applicable.

7.2.1.1 *Dead load*

Dead loads shall be calculated based on unit weights which shall be established taking into consideration the materials specified for construction or **2** of Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS.

7.2.1.2 *Imposed load*

Imposed loads shall be considered as per **3** of Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS. A minimum construction load (as per design requirement) of 1.0 kN/m² or construction traffic load shall be adopted for design, if not specified in the contract document. However, due consideration shall be given to actual possible construction loads.

7.2.1.3 *Wind load*

Wind load shall be considered as per **4** of Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS.

7.2.1.4 *Construction traffic load*

Construction traffic load shall be assumed in accordance with the project requirements. A minimum traffic load (as per design requirement) of 1.0 kN/m^2 or imposed load shall be adopted for design, if not specified in the contract document.

7.2.1.5 *Seismic load*

The seismic forces shall be calculated in accordance with **5** of Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS.

7.2.1.6 *Notional horizontal load*

Notional horizontal load (as shown in Fig. 12) shall be taken as not less than 1.5 percent of the characteristic dead load and accidental loads such as from earth movement, impact of construction vehicles, etc.

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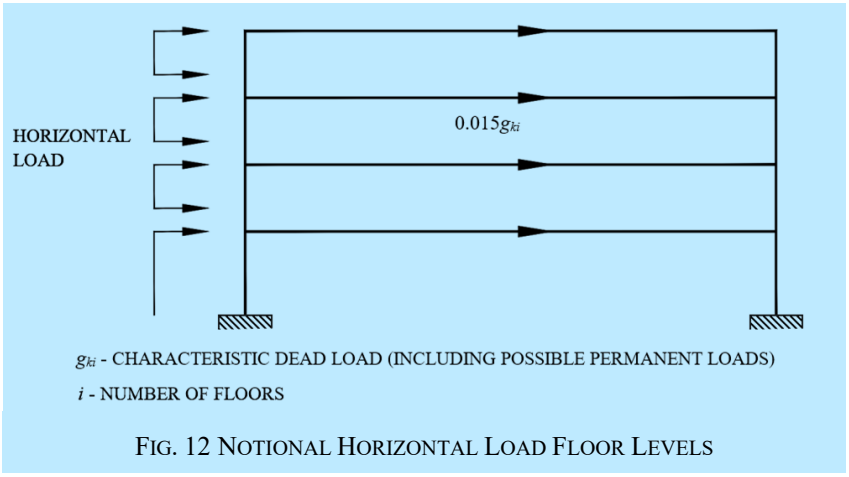

FIG. 12 NOTIONAL HORIZONTAL LOAD FLOOR LEVELS

7.2.1.7 *Shrinkage, creep and temperature effects*

The effects of shrinkage, creep and temperature shall be considered according to the good practice [C-7(2)]. Special literature can be referred to for critical structures.

7.2.1.8 *Environmental loading*

The effects of thermal loading and changes in values of creep and shrinkage due to environmental exposure shall be considered according to the good practice [C-7(2)] or as defined in design basis report.

7.2.1.9 *Other forces and effects*

Relevant clauses of Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS shall be followed for additional forces such as snow load, liquid retaining loads, impact due to equipment, cranes, etc.

7.2.2 *Load Combinations*

Basic load combination shall be as per the good practice [C-7(2)] and Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS. In addition, precast specific load combinations specified by the competent authority or given in this NBCS shall be adopted.

7.2.2.1 *Factor of safety/partial safety factor for load combinations*

Load and material safety factors shall be as per the good practice [C-7(2)]. Precast specific partial safety factors given at various sections of this NBCS shall be adopted accordingly.

7.3 Effective Span

7.3.1 Except for the purely simply supported spans, the effective spans for all elements shall be estimated as per the relevant clauses of the good practice [C-7(2)].

7.3.2 Effective span for all elements during the temporary erection stage are to be considered based on the temporary support condition.

7.3.3 The effective span for simply supported elements shall be considered as the centre-to-centre distance between the bearing pads or shims at each end. If the multiple shims are used, only the shims or pads close to the end of the elements shall be considered as governing case.

7.4 Stiffness

7.4.1 *Relative Stiffness*

The relative stiffness of the member shall be calculated based on the moment of inertia of the section considering gross section, transformed section and cracked section as provided in the good practice [C-7(2)].

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7.4.2 *Stiffness Modifiers*

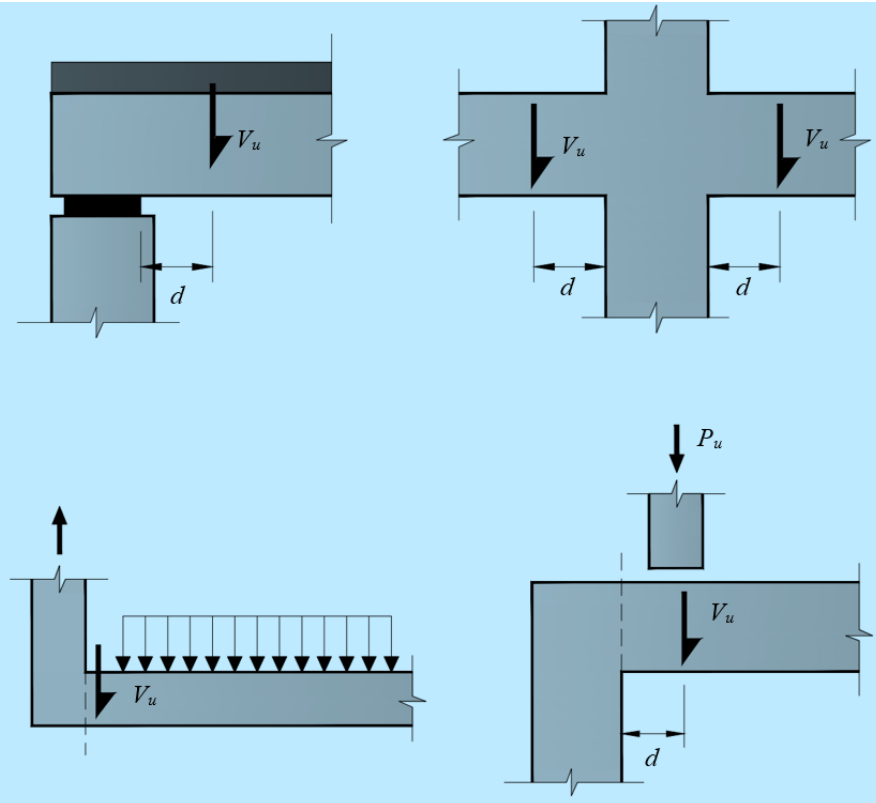
Stiffness reduction factors for members and connections shall be applied as per **5** of Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS, considering structural system type and ductility requirements.

7.4.3 *Critical Sections for Moment and Shear*

The critical sections for moment and shear shall be identified as per the good practice [C-7(2)] and additionally as per **7.4.3.1** and **7.4.3.2**.

7.4.3.1 Critical section for shear as per the good practice [C-7(2)] is shown in Fig. 13.

7.4.3.2 For L-beams and inverted T-beams carrying loads within the bottom 30 percent of section depth (*refer* Fig. 13), the critical section for shear shall be taken at the face of support, unless a monolithic or emulative beam-column joint is specifically detailed to resist shear.



d – effective depth of beam

FIG. 13 CRITICAL SECTION FOR SHEAR

7.5 Concentrated Loads on Floor Slabs

7.5.1 Concentrated loads shall be applied as per Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS. Effective width for slabs carrying concentrated loads shall be calculated as per the good practice [C-7(2)] along with **7.5.2** and **7.5.3**.

7.5.2 The load distribution for hollow core slab depends on the one-way action of the plank and the cast in place structural topping. The effective width for hollow core slabs shall be calculated as per **7.5.3**.

7.5.3 Moments and shears induced by line and concentrated loads shall be resisted by an effective section as described in Fig. 14. If the total deck width, perpendicular to the span, is less than the span, modification may be

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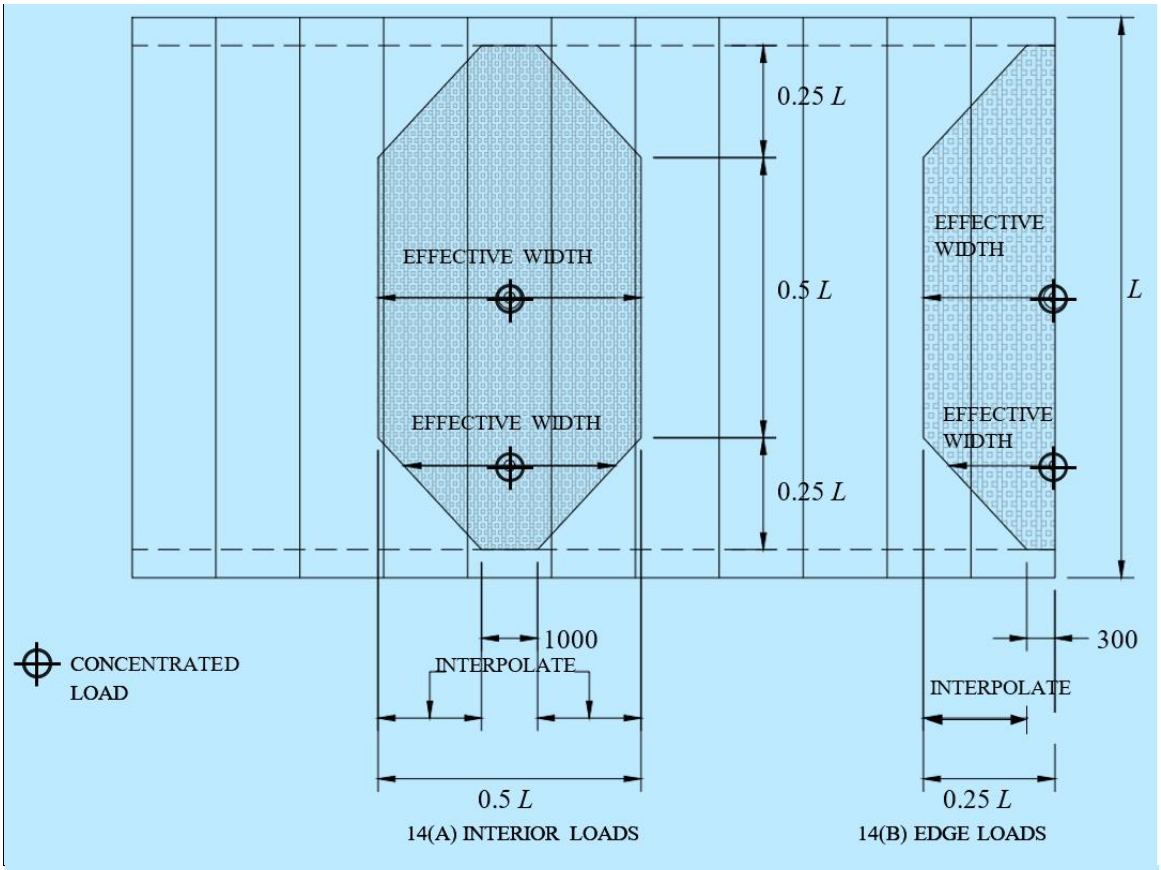
required. In addition, the following limitations would apply:

a) As the width of the system becomes narrower than the span length, the effective resisting widths will become narrower;

b) For extremely high span-depth ratios (more than approximately 50), the effective section at mid-span shall be reduced by 10 percent to 20 percent;

c) For spans less than about 3 m, the effective width at the support may become narrower; and

d) Local load concentrations can cause longitudinal splitting failures due to transverse bending in the system and punching shear failure. The magnitude of concentrated loads shall be limited to minimise such failures.



All dimensions in millimetres.

FIG. 14 MOMENT AND SHEAR DISTRIBUTION WIDTHS FOR HCS

7.6 Stability Design

7.6.1 *Design Philosophy*

The following factors shall be considered in designing for stability of a precast structure:

a) The overall structural system shall be checked for global and local stability under all applicable loading conditions including temporary and final stages;

b) Precast structural systems shall be designed to resist wind and all applicable lateral loads;

c) Cross walls or sway frames shall be so arranged to provide lateral stability;

d) When lateral loads are low, stability shall be verified using a notional horizontal force equal to 1.5 percent of total vertical load, as shown in Fig. 12;

e) Lateral stability shall be maintained during all construction and erection stages. Temporary bracing, shoring, or props shall be detailed and installed to match the staged loading conditions. A temporary

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works scheme including propping layout, sequencing and dismantling plan shall be submitted and approved before execution;

f) Inverted T and L-beams shall be analysed for asymmetric loading during both construction and service stages, using all applicable load combinations;

g) The strength of grout shall be at least 50 percent of the design strength or as per design requirement before application of any other loads (temporary or permanent) on the element. Stress and cracking shall be evaluated and verified before proceeding;

h) Long-span and high-rise precast structures shall include engineered bracing and verified stability measures during all construction and service stages; and

j) Temporary anchors shall be designed for proper alignment and safety of the elements. Appropriate washers shall be used.

7.6.1.1 *Overturning*

Stability against overturning shall be checked in accordance with the requirements in Part C/Section 5 of this NBCS. For L and inverted T beams, lateral support shall be provided during construction to avoid stability related failures.

7.6.1.2 *Sliding*

Stability against sliding shall be checked in accordance with the requirements in the good practice [C-7(2)]. All the precast members during construction shall be propped or anchored to prevent sliding from the supporting structure. Minimum bearing as per 7.11 shall be followed.

7.6.1.3 *Probable variation in dead load*

Probable variations in dead load during casting of in-situ components shall be accounted for in the design. Pouring of structural topping above the long span elements (such as HCS) shall be monitored to have the required pouring depth measured from top of the highest point of the element (camber point) instead of global level.

7.6.1.4 *Moment connection*

Moment connection shall be used for lateral force resistance system frames. Where frames are pin-jointed, transverse stability shall be ensured through structural walls or bracing system. Integrity ties shall be provided to maintain system continuity.

7.6.1.5 *Lateral sway*

Stability against lateral sway shall be evaluated using the good practice [C-7(2)] and **5** of Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS, including during temporary construction stages and final occupancy.

7.6.1.6 *Lateral force resistance system design*

The lateral force resistance system design shall be in accordance with the relevant provisions of Part C 'Structural Design, Section 1 Loads and Load Effects', Part C 'Structural Design, Section 5 Structural Concrete' and Part C 'Structural Design, Section 6 Steel and Steel-Concrete Composite Construction, Subsection A Steel' of this NBCS with the guidelines stipulated in **6**. Additional impact of lateral loads such as earth fill, or liquid/snow retention shall be analysed as per the project requirements.

7.7 Durability Design

7.7.1 *Design Philosophy*

The following factors shall be considered for achieving the design life and durability of a precast structure:

a) Geometry and detailing of precast elements;

b) Concrete mix design, including constituent materials;

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c) Nominal and minimum cover as per environmental class;

d) Environmental exposure classification in accordance with the good practice [C-7(2)];

e) Fire resistance rating based on intended use;

f) Access provisions for inspection and maintenance;

g) Controlled production quality;

h) Handling, transport and erection constraints; and

j) Watertight and durable joint detailing.

In addition to above requirements, the recommendations specified in the good practice [C-7(2)] and relevant accepted standards shall be followed.

7.7.2 *Service Life*

7.7.2.1 The service life of any structure shall be specified in the design basis report (DBR) or considered based on the good practice [C-7(2)].

7.7.2.2 For temporary structure, the service life shall be considered as the period the structure is required to satisfy the intended function, or as mentioned in the contract document.

7.7.2.3 The service life specified for a structure does not apply to architectural finishes, replaceable materials such as sealants, paints, non-structural grout. The service life of such items shall be considered as per the manufacturer's guidelines and mutually agreed upon by all the stakeholders.

7.7.3 *Shape of Precast Units*

7.7.3.1 Sharp corners or sudden changes in geometry, re-entrant corners, or grooves in a section causes stress concentrations that may lead to cracking or spalling of concrete and therefore shall be avoided. For sections with practical constraints, stress concentrations shall be checked and strengthening or local detailing shall be provided, as necessary.

7.7.3.2 Extra reinforcements shall be provided around the cut-outs or opening, if any, present in an element. Larger cut-outs shall be considered in the analysis to characterize the stress concentration and its effects on the design as per the good practice [C-7(2)].

7.7.3.3 Buckling and instability must be prevented during the lifting and erection of long, slender precast units, as per the good practice [C-7(2)]. For thin sections (thickness less than 75 mm), the positioning of lifting inserts should be such that it prevents buckling of the compression flange, especially during the handling and manoeuvring of the elements.

7.7.3.4 The precast unit shall be designed and detailed to have sufficient drainage to avoid standing water or excessive moisture.

7.7.4 *Concrete Cover*

Concrete cover for precast concrete elements shall comply with the requirements of the good practice [C-7(2)], unless reduced values from Table 2 are adopted. Reduced cover values shall only be permitted if the elements are produced in a controlled environment.

7.7.4.1 The minimum concrete cover for reinforcement to ensure durability against fire and corrosion shall comply with the provision of the good practice [C-7(2)]. Fire resistance of joint fillers and other components shall meet or exceed the fire resistance rating of the connected precast members, or as prescribed in **7.8**.

7.7.4.2 The cover to all brackets, fixings, etc shall comply with the minimum cover requirements specified for reinforcement as per the above table or the good practice [C-7(2)].

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Table 2 Nominal Cover in Precast Elements

(*Clause* 7.7.4)

Sl No.	Exposure	Elements	Nominal Concrete Cover, (mm, *Min*)
(1)	(2)	(3)	(4)
i)	Mild	All	20
ii)	Moderate	Slabs and walls	25
		For the ties/stirrups of beams and columns exposed to weather	25
		For ties/stirrups of beams and columns non-exposed to weather	Greater of d_b or 20
iii)	Severe	All	40
iv)	Very Severe	All	45
v)	Extreme	All	70

NOTES

1 The prescribed covers for precast elements are lesser than the covers noted in the good practice [C-7(2)] to account for the greater control for proportioning, placing and curing inherent to precast concrete produced in a controlled environment.

2 Controlled environment does not imply that the precast members shall be manufactured in a factory. Elements manufactured at the jobsite or in a temporary factory also qualify under these requirements, if the controls are equal to that normally adopted in a factory. The client and engineer-in-charge shall verify the temporary factory and certify the same, if the cover requirements of this Table are adopted.

3 d_b is bar diameter.

7.7.5 *Protection and Maintenance of Joints and Connections*

7.7.5.1 The connection between the precast element shall be properly filled with high strength, non-shrink, non-metallic cementitious grout (minimum 50 MPa or 5 MPa more than the strength of the element, whichever higher) to prevent corrosion, cracking and spalling of concrete. Where the joint gap between the precast elements is more than 50 mm, the provisions of 5.4.1.3 shall apply.

7.7.5.2 Steel plates or inserts used in permanent connections shall be provided with adequate concrete cover or protected with corrosion-resistant coatings. Inserts for temporary use shall be removed or treated to prevent corrosion.

7.7.5.3 Where minimum cover cannot be ensured due to geometric or project constraints, only corrosion-resistant materials (such as stainless steel or galvanized steel) shall be used for connectors and fixings.

7.7.6 *Other Effects*

7.7.6.1 Joint spacing and width shall account for cumulative movements including production tolerances, creep, shrinkage, settlement and thermal expansion to prevent cracking and spalling.

7.7.6.2 In addition to satisfying general requirements for durability, strength, stability, control of cracking and limitation of deformation, the following provisions shall be met:

a) Structural components shall be designed to limit cracking and deformation from early thermal movements, creep and shrinkage; and

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b) Bearings or movement joints shall be provided to minimize structural restraints. Where restraint is unavoidable, the design shall include measures to resist resulting stresses, with reinforcement provided as per as per the good practice [C-7(2)].

7.8 Fire Resistance

7.8.1 *Fire Resistance Requirement*

7.8.1.1 The fire resistance of a structural element shall be expressed in terms of time in hours in accordance with good practices [C-7(19)]. General requirements for fire protection are given in good practice Part F 'Fire and Life Safety' of this NBCS.

7.8.1.2 Elements and joints wider than 50 mm shall comply with fire resistance provisions of the good practice [C-7(2)], except where otherwise specified in this section.

7.8.1.3 All joints in fire-prone structures shall be filled with mortar, grout, or concrete in accordance with the design requirements.

7.8.1.4 The joints between slab elements or hollow core elements with structural topping of more than 50 mm (minimum thickness of structural topping concrete) may not require any additional treatment for fire rating.

7.8.1.5 For joints between structural wall elements with a width less than or equal to 50 mm shall be filled with grout of strength minimum 50 MPa or 5 MPa more than the strength of the element, whichever higher. For non-structural wall elements, the grout strength shall be at least equal to that of the element. For joints with a width exceeding 50 mm, the provisions of 5.4.1.3 shall apply.

7.9 Structural Design of Precast Concrete Components

7.9.1 *Footings*

7.9.1.1 All the design and detailing requirements of the good practice [C-7(2)] shall be followed for members/elements along with any specific criteria given in this Section.

7.9.1.2 Precast footing with multiple elements stitched using appropriate connection arrangement shall be adopted, wherever required. However, footing subjected to vibration shall be without any joints, preferably a single unit or cast in-situ or the system shall be tested prior to application for the required level of vibration.

7.9.1.3 The minimum thickness at the edge of a precast footing shall be as per the good practice [C-7(2)].

7.9.1.4 The connections shall be detailed as per 8 to ensure effective force transfer between elements.

7.9.1.5 Detailing of column, pedestal and wall connections to the footing elements shall be as per 8.

7.9.2 *Walls*

7.9.2.1 Design and detailing of wall elements shall comply with the good practice [C-7(2)], in addition to the specific requirements of this Section.

7.9.2.2 As the elements are manufactured in controlled environment, walls subjected to marginal shear forces, minimum longitudinal and transverse reinforcement shall be 0.1 percent of gross area. For other cases, reinforcement shall be as per the good practice [C-7(2)].

7.9.2.3 The connections shall be detailed as per 8 and shall ensure proper force transfer between the elements.

7.9.2.4 Walls shall be tied to intersecting elements such as floors, columns, other walls and footings using appropriate ties or mechanical connections that ensure force transfer and structural continuity.

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7.9.3 *Shear Wall*

7.9.3.1 All the design and detailing requirements of the good practice [C-7(2)] shall be followed for members/elements along with any specific criteria given in this Section.

7.9.3.2 Minimum reinforcement for structural wall shall be as per the good practice [C-7(2)] and as per good practice [C-7(18)].

7.9.3.3 At least one continuous vertical tie of 12 mm diameter or higher shall be provided from foundation to terrace at each end of the structural wall using Class H coupler as per [C-7(9)] or any other connection with appropriate lapping or welding as per the design requirements.

7.9.3.4 The connections shall be detailed as per **8** and to ensure adequate transfer of axial, shear and bending forces between connected elements.

7.9.3.5 Shear walls shall be structurally tied to intersecting elements, including floors, columns, other walls and footings, using mechanical ties, dowels, or other engineered connections capable of transferring expected design loads.

7.9.4 *Floors/Slabs*

7.9.4.1 All the design and detailing requirements of the good practice of [C-7(1)] shall be followed for members/ elements along with any specific criteria given in this Section.

7.9.4.2 Minimum reinforcement for slab elements and structural topping the minimum reinforcement shall be as per good practice [C-7(1)].

7.9.4.3 The thickness of structural topping shall be considered if the maximum shear stress at any location is less than 0.4 MPa. Also, the top surface of the slab shall be roughened to a minimum amplitude of 6 mm. Where the shear stress exceeds 0.4 MPa, adequate shear reinforcement or studs shall be provided to ensures the composite action between the topping and the floor slab elements.

7.9.4.4 The minimum grade of concrete for the structural topping shall be equal to M30 or as per design (whichever is higher).

7.9.4.5 The connections shall be detailed as per **8** and shall ensure proper force transfer between the elements.

7.9.4.6 For pre-tensioned and post-tensioned members, stress limits shall be as per good practice [C-7(3)].

7.9.4.7 For simply supported slab at least one third of the maximum positive reinforcement shall be extended till the centre of the bearing. For other cases, at least one fourth of the reinforcement shall extend into the support for a length of 150 mm. Adequate development length shall be provided to the bottom reinforcement as per good practice [C-7(2)]

7.9.4.8 For requirements relating to design and detailing for precast structures with hollow core slabs, Annex H may be referred.

7.9.5 *Columns*

7.9.5.1 All the design and detailing requirements of the good practice [C-7(1)] shall be followed for members/elements along with any specific criteria given in this NBCS.

7.9.5.2 Minimum reinforcement requirements shall be as per the good practice [C-7(2)].

7.9.5.3 The connections shall be detailed as per **8** and shall ensure proper force transfer between the elements.

7.9.5.4 At least four continuous vertical ties of 12 mm diameter or higher shall be provided from foundation till the roof using Class H coupler as per accepted standard [C-7(9)] or any other connection, with appropriate lapping or welding.

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7.9.5.5 Transverse reinforcement as per the good practice [C-7(2)] shall be provided in the members as well as the splice locations (grouted coupler, grouted duct, mechanical connections, etc).

7.9.5.6 For connection using grouted coupler of height less than 250 mm, minimum cover to the confinement reinforcement shall be 25 mm at the location of coupler. For all other cases cover shall be as per the good practice [C-7(2)].

7.9.5.6 For connection using grouted duct/tube or any other, minimum cover shall be as per the good practice [C-7(2)].

7.9.6 *Beams*

7.9.6.1 All the design and detailing requirements of good practice [C-7(1)], [C-7 (2)] and [C-7(3)] shall be followed for members/elements along with any specific criteria given in this Section.

7.9.6.2 Minimum reinforcement requirements shall be as per the good practice [C-7(2)].

7.9.6.3 The connections shall be detailed as per **8** and shall ensure proper force transfer between the elements.

7.9.6.4 Transverse reinforcement as per good practice [C-7(2)] shall be provided in the members as well as the splice locations (grouted coupler, grouted duct, mechanical connections, etc).

7.9.6.5 For connection using mechanical coupler of length less than 500 mm, minimum cover of 20 mm shall be maintained to the transverse reinforcement at the coupler location. In all other cases the cover shall be as per the good practice [C-7(2)].

7.9.6.6 For beam made composite to the cast in-situ topping (T-action or L-action, in compression), the thickness of cast in-situ shall be considered in the height of the beam and the width of flange shall be calculated according to the good practice [C-7(2)].

7.9.6.7 Stress limits for pre-tensioned and post-tensioned beams shall be as per good practice [C-7(3)].

7.9.6.8 For simply supported beam at least one third of the maximum positive reinforcement shall be extended till the centre of the bearing. For other cases, the reinforcement shall extend to the other member or to the support and shall develop f_y at the tension face of the support.

7.9.7 *Staircase/Landing*

7.9.7.1 All the design and detailing requirements of Part C ‘Structural Design, Section 5 Structural Concrete’ of this NBCS shall be followed for members along with any specific criteria given in this Section.

7.9.7.2 Minimum reinforcement requirements shall be as per Part C ‘Structural Design, Section 5 Structural Concrete’ of this NBCS and shall ensure proper force transfer between the elements.

7.9.7.3 The connections shall be detailed as per **8**.

7.9.8 *Composite Concrete Construction*

7.9.8.1 All the design and detailing requirements of Part C ‘Structural Design, Section 5 Structural Concrete’ of this NBCS shall be followed for members along with any specific criteria given in this Section.

7.9.8.2 The connections shall be detailed as per **8** and shall ensure proper force transfer between the elements.

7.9.9 *Insulated Panels/Thermal Comfort*

7.9.9.1 All the design and detailing requirements of Part C ‘Structural Design, Section 5 Structural Concrete’ of this NBCS shall be followed for members along with any specific criteria given in this Section.

7.9.9.2 Minimum thickness of the concrete wythe shall be 50 mm.

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7.9.9.3 Individual wythe shall be connected to each using wythe connectors by providing solid zones or special reinforcements.

7.9.9.4 Maximum spacing between the reinforcements in the wythe shall be 250 mm or as per design.

7.9.9.5 For walls made from composite wythe, the solid zone shall be designed to transfer the shear demand on the wall and for moment transfer complete composite wythe section shall be considered.

7.9.9.6 Insulation material if provided shall have adequate bond with the concrete wythes to avoid splitting of junction during handling.

7.9.9.7 The connections with other elements shall be detailed as per 8 and shall ensure proper force transfer between the elements.

7.9.10 *Composite Floors*

7.9.10.1 All the design and detailing requirements of Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS shall be followed for members along with any specific criteria given in this Section.

7.9.10.2 No shear reinforcement/studs shall be provided if the maximum shear stress at any location is less than or equal to 0.4 MPa.

7.9.10.3 The minimum grade of concrete for the structural topping shall be equal to M30 or as per design (whichever is higher).

7.9.10.4 The connections shall be detailed as per 8 and shall ensure proper force transfer between the elements.

7.9.11 *Others Structural Elements*

7.9.11.1 Any structural element shall be designed and detailed as per Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS.

7.9.11.2 The connections shall be detailed as per 8 and shall ensure proper force transfer between the elements.

7.9.11.3 Any connection scheme found effective in force transfer shall be used, subjected to approval of competent authority.

7.10 *Design of Diaphragm*

For the stability of the precast concrete building horizontal loads due to wind and earthquake shall be transmitted to the lateral force resistance system through the floor acting as horizontal deep beam. However, in a precast slab system, the floor comprises individual units generally not connected to each other. A structural topping concrete is provided to act as the diaphragm, or the individual elements are connected. In this system, the diaphragm action shall ensure the following:

a) Collect and transfer lateral forces at each level to the lateral force resisting elements such as shear walls, moment frames, etc; and

b) Tie individual lateral force resisting elements into a single lateral force resisting system.

7.10.1 Diaphragm action arises from the in-plane stiffness inherent to the floor system. As a result, substantial in-plane forces can develop in the diaphragm during lateral loading. In the context of precast concrete construction, the panelled nature of floor systems can pose challenges in the seismic design of diaphragms. These challenges may involve ensuring sufficient strength, stiffness, and in some cases, ductility in both the diaphragm and its connections.

7.10.2 In case of buildings whose floors can provide rigid horizontal diaphragm action, the total shear in any horizontal plane shall be distributed to the various vertical elements of lateral forces resisting system, assuming the floors to be infinitely rigid in the horizontal plane.

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7.10.3 In case of building whose floor diaphragms cannot be treated as infinitely rigid in their own plane, the lateral shear at each floor shall be distributed to the vertical elements resisting the lateral forces, considering the in-plane flexibility of the diaphragms.

NOTE — Reinforced concrete monolithic slab-beam floors or those consisting of prefabricated/precast elements such as solid slabs, hollow core slabs or double tees with reinforced topping shall satisfy the requirements of 7.10.10. In all cases, the precast elements need not be considered as part of the diaphragm except solid slabs with proper connectivity. The gravity load design, however, shall be composite including the structural topping as well as the precast element provided the interface shear at the topping and element face is analysed and designed for the same.

7.10.4 *Rigid Diaphragms*

A diaphragm is classified as rigid if it can distribute the horizontal forces to the vertical components in proportion to their relative stiffness. Diaphragms of concrete slabs of concrete-filled metal deck with span-to-depth ratios of 3 or less in structures that have no horizontal irregularities are permitted to be idealized as rigid. The minimum thickness of structural topping concrete shall be as per 7.10.10.

Where the diaphragm is very stiff compared to the vertical elements, as in a low aspect ratio, cast-in-place diaphragm supported by moment frames, it is acceptable to model the diaphragm as a completely rigid element.

7.10.5 *Flexible Diaphragms*

A floor diaphragm shall be considered to be flexible, if it deforms such that the maximum lateral displacement measured from the chord of the deformed shape at any point of the diaphragm is more than 1.5 times the average displacement of the entire diaphragm or as per good practice [C-7(18)] and **5** of Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS. Flexible diaphragm can be adopted in modular structures having frequent openings such as substation, electrical rooms, industrial structures, etc.

7.10.5.1 Where the diaphragm is flexible compared with the vertical elements, as in some jointed precast systems supported by structural walls, it may be acceptable to model the diaphragm as a flexible beam spanning between rigid supports.

7.10.5.2 In other cases, it may be advisable to adopt a more detailed analytical model to account for the effects of diaphragm flexibility on the distribution of displacements and forces. Examples include buildings in which diaphragm and vertical element stiffness have approximately the same value, buildings with large force transfers, and parking structures in which ramps connect between floors and act essentially as bracing elements within the building.

7.10.6 *Topped Diaphragms*

Topped floor and roof diaphragms include systems with in-situ concrete slab over precast RC planks and joists. The in-situ concrete slab is considered to act composite with the precast RC planks with sufficient surface roughness to transfer horizontal shear, so that the flexural strength of the precast component is enhanced by the added structural depth of the in-situ slab.

NOTE — In the computational model of the structure, the diaphragm shall be modelled considering the thickness of the topping/structural topping with extra dead load applied on top of the diaphragm to consider the precast planks or elements.

7.10.7 *In-plane Diaphragm Design Loads and Forces*

Topped floor and roof diaphragms shall be designed to resist design in-plane inertial forces due to earthquake forces and wind loads from the structural analysis. Earthquake forces shall be calculated in accordance with good practices [C-7(1)] and [C-7(20)] and **5** of Part C 'Structural design/Section 1 Loads and load effects' of this NBCS. Wind loads shall be estimated in accordance with Part C 'Structural design/Section 1 Loads and load effects' of this NBCS.

7.10.8 *Standard Analysis Methods for Diaphragm Design*

7.10.8.1 The magnitude of the inertia force at each floor level is estimated using a capacity design approach or a diaphragm design force distribution based on equivalent lateral loads. A key design step is to transform the diaphragm design forces into internal actions. For regular floor plans, a 'horizontal plate girder' analogy is typically used in design. In this approach, diaphragm design forces are typically applied at each level as a

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distributed in-plane load along the length of the diaphragm. The resulting girder internal forces are used to determine the amount of flexural reinforcement (chord steel), shear (web) reinforcement, and lateral system reaction load path reinforcement (collector steel). An irregular mass distribution in plan may require the design forces to take on a smeared load configuration as the mass distribution with respect to diaphragm depth has relevance in determining the proper web reinforcement. For more complex diaphragm configurations, a strut-and-tie method can be used. The strut-and-tie method accurately predicts the behaviour of floor diaphragms, in general, and may become the preferred method of diaphragm analysis (*see* Fig. 15 and Fig. 16).

7.10.8.2 Calculation of diaphragm in-plane design moments, shears and axial forces shall be consistent with requirements of equilibrium and with design boundary conditions. It shall be permitted to calculate design moments, shears, and axial forces in accordance with one of (a) through (d):

a) A rigid diaphragm model if the diaphragm can be idealized as rigid;

b) A flexible diaphragm model if the diaphragm can be idealized as flexible;

c) A bounding analysis in which the design values are the envelope of values obtained by assuming upper bound and lower bound in-plane stiffness for the diaphragm in two or more separate analyses; and

d) A finite element model considering diaphragm flexibility (semi rigid).

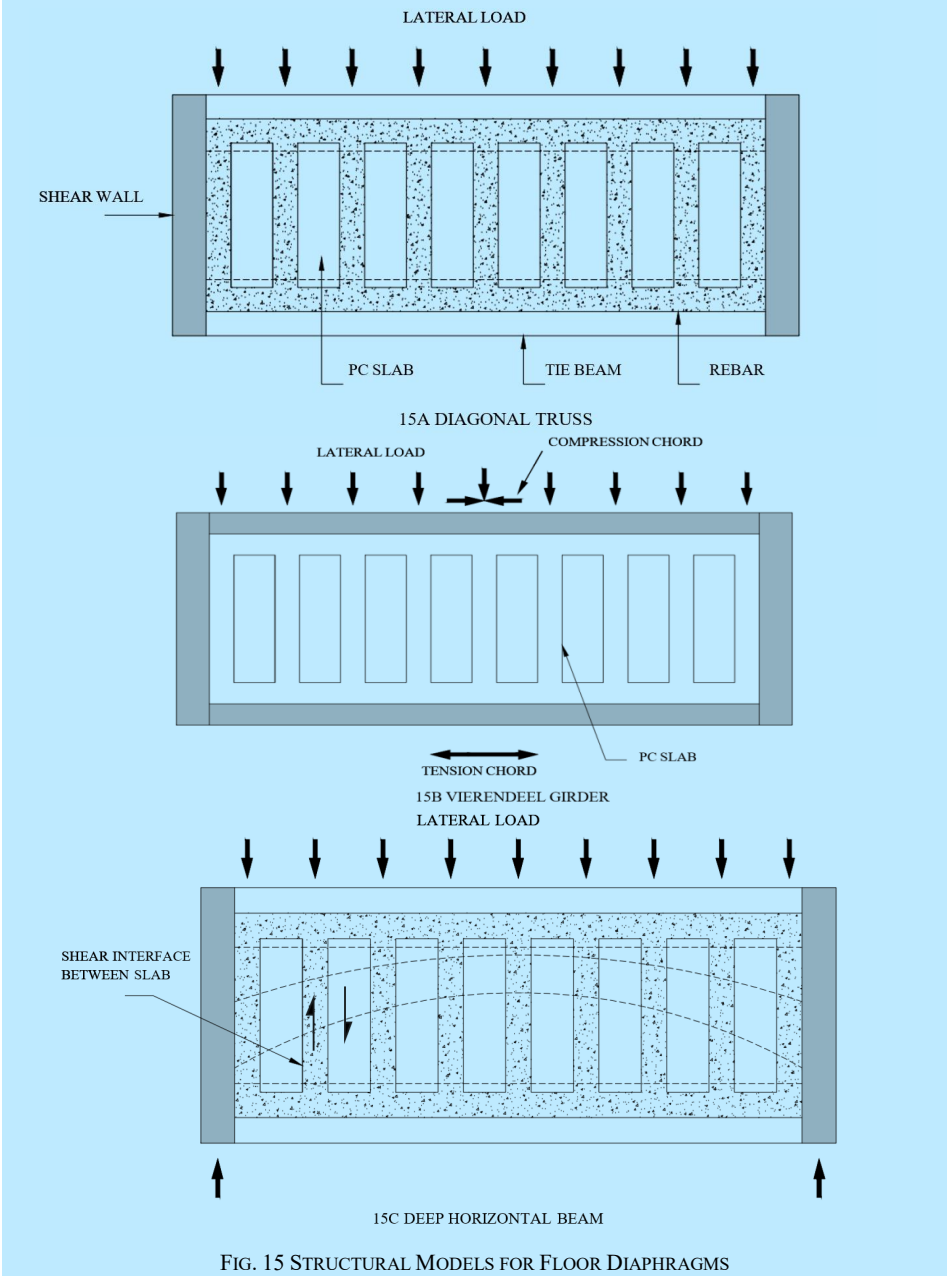

FIG. 15 STRUCTURAL MODELS FOR FLOOR DIAPHRAGMS

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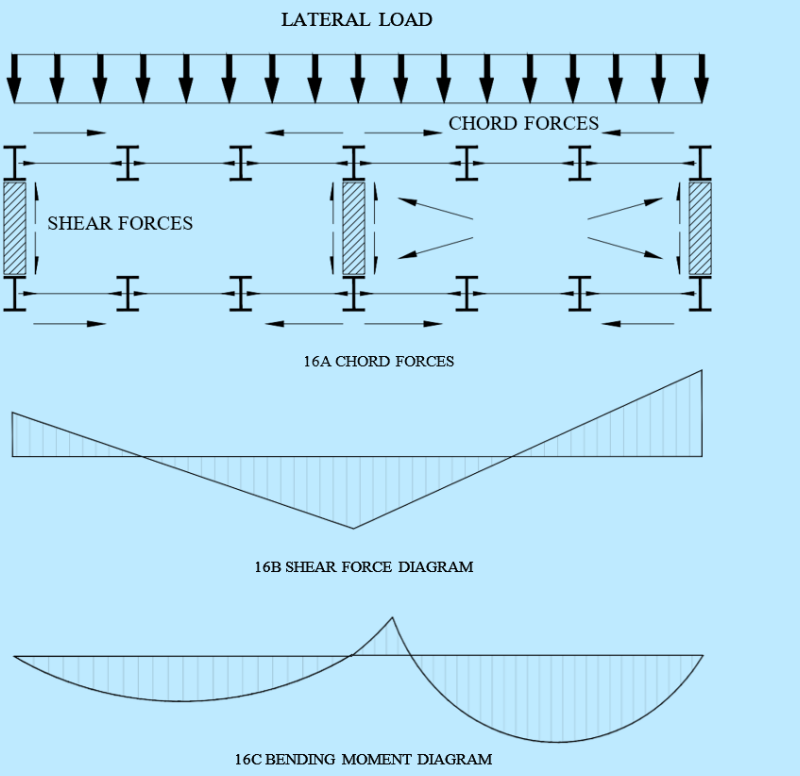

FIG. 16 DESIGN FORCES ON A DIAPHRAGM

7.10.9 *Materials*

7.10.9.1 Design properties of in-situ concrete for the topped diaphragms and diaphragm reinforcement shall be considered as per Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS.

7.10.9.2 Minimum grade of structural topping concrete shall be 40 MPa or as per design (whichever is higher).

7.10.10 *Minimum thickness of topped diaphragms*

7.10.10.1 Topped floor and roof diaphragms shall have thickness as required for stability, strength and stiffness under factored load combinations.

7.10.10.2 Topped floor and roof diaphragms that comprises RC planks composite with in-situ concrete slabs, the full thickness of the entire diaphragm must be sufficient to resist the design actions. In system with special precast slab elements such as hollow core slabs or double tees, the diaphragm thickness shall be limited to the in-situ structural topping concrete.

7.10.10.3 The minimum thickness of in-situ structural topping concrete shall be as per 6.3.6.1.

7.10.11 *Diaphragm Connections*

Connections between individual precast RC planks shall not be designed for any diaphragm forces in a topped diaphragm where the diaphragm thickness is limited to cast-in-place structural topping only. Nominal connections required to facilitate the vertical alignment of the RC planks and flanges shall be provided.

7.10.12 *Opening and Voids in the In-situ Concrete Topping Slab*

The effects of openings and voids in the in-situ concrete topping slab shall be considered in the diaphragm design. Detailing around the openings and voids must be done in accordance with Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS.

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7.10.13 *Collectors*

7.10.13.1 Collectors shall extend from the vertical elements of the lateral-force-resisting system across all or part of the diaphragm depth as required to transfer shear from the diaphragm to the vertical element (as shown in Fig. 17). It shall be permitted to discontinue a collector along lengths of vertical elements of the lateral-force-resisting system where transfer of design collector forces is not required.

7.10.13.2 Collectors shall be designed as tension members, compression members, or both, in accordance with the criteria for axial member in Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS.

7.10.13.3 Where a collector is designed to transfer forces to a vertical element, collector reinforcement shall extend along the vertical element to at least the greater of (a) and (b):

a) The length required to develop the reinforcement in tension; and

b) The length required to transmit the design forces to the vertical element through shear-friction in accordance with 7.12, through mechanical connectors, or through other force transfer mechanisms.

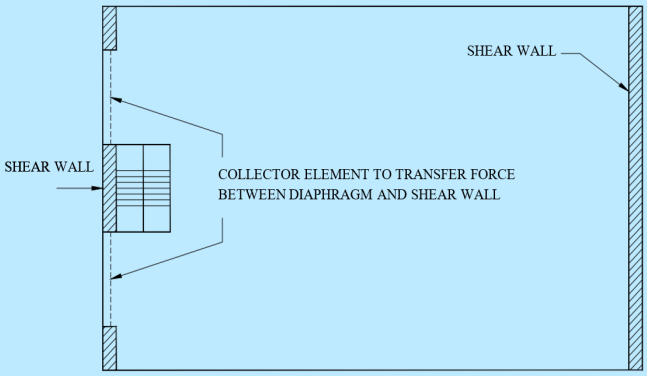

FIG. 17 COLLECTORS BETWEEN FRAMING ELEMENTS

7.10.14 *Reinforcement Limits*

7.10.14.1 Reinforcement to resist shrinkage and temperature stresses shall be estimated according to relevant clauses of Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS.

7.10.14.2 Reinforcement designed to resist diaphragm in-plane forces shall be in addition to reinforcement designed to resist other load effects, except that reinforcement designed to resist shrinkage and temperature effects shall be permitted to also resist the diaphragm in-plane forces.

7.10.15 *Reinforcement Detailing*

7.10.15.1 Concrete cover for reinforcement shall be in accordance with good practice [C-7(2)].

7.10.15.2 Development lengths of deformed and prestressed reinforcement shall be in accordance with good practice [C-7(2)] and [C-7(3)], respectively, unless longer lengths are required by high seismic zone requirements.

7.10.15.3 Splices of deformed reinforcement shall be in accordance with Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS.

7.10.15.4 Bundled bars shall be in accordance with Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS.

7.10.16 *Reinforcement Spacing*

7.10.16.1 Minimum spacing of reinforcement shall be in accordance with Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS.

7.10.16.2 Maximum spacing of deformed reinforcement shall be the lesser of five times the diaphragm thickness and 450 mm.

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7.10.17 *Diaphragm and Collector Reinforcement*

7.10.17.1 Tensile or compressive force in reinforcement at each section of the diaphragm or collector shall be developed on each side of that section.

7.10.17.2 Reinforcement provided to resist tension shall extend beyond the point at which it is no longer required to resist tension for at least a length of L_d, except at diaphragm edges and at expansion joints.

7.10.17.3 Welded wire mesh of diameter 6 mm or higher shall be used in the structural topping concrete to achieve the diaphragm reinforcement.

7.10.18 *Other Parameters*

7.10.18.1 Heavy duty PVC or metal conduits with diameter up to 20 mm are preferable. Larger diameter conduits (25 mm or 32 mm or higher) may be permitted as per project requirements with proportionately higher thickness of structural topping.

7.10.18.2 Single junction box shall be used to conduct the conduits.

7.11 Designing of Bearing Region

7.11.1 *General*

The bearing region shall ensure structural integrity through the following measures:

a) Overlapping reinforcement in reinforced bearing zones;

b) Physical restraint to prevent loss of bearing due to movement; and

c) Tolerance provisions to accommodate cumulative deviations due to production and erection.

7.11.2 *Net Bearing Width of Non-Isolated Members*

The net bearing width of a member shall not be less than 40 mm and shall be determined by:

$$\text{Net bearing width} = \frac{\text{ultimate support reaction}}{\text{effective bearing length} \times \text{ultimate bearing stress}}$$

The net bearing width should be increased to cater for any free movement permitted or rotation of the bearing about the support.

7.11.3 *Effective Bearing Length*

The effective bearing length of a member shall be taken as the least of:

a) Actual bearing length;

b) One-half of bearing length plus 100 mm; or

c) 600 mm.

7.11.4 *Design Ultimate Bearing Stress*

The design ultimate bearing stress shall be based on the weaker bearing surface and shall be:

a) $0.4f_{ck}$ for dry concrete bearings;

b) $0.6f_{ck}$ for mortar-bedded concrete bearings; or

c) $0.8f_{ck}$ for steel bearing plates embedded in concrete, provided the plate dimensions do not exceed 40 percent of the supporting concrete dimension.

An intermediate value of bearing stress between dry and bedded bearings may be used for flexible bedding.

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7.11.5 *Net Bearing Width of Isolated Members*

For isolated members, the net bearing width shall be increased by 20 mm over that required for non-isolated members.

7.11.6 *Minimum Dimensions of Bearing*

The minimum requirement for bearing length shall be as per Fig. 18.

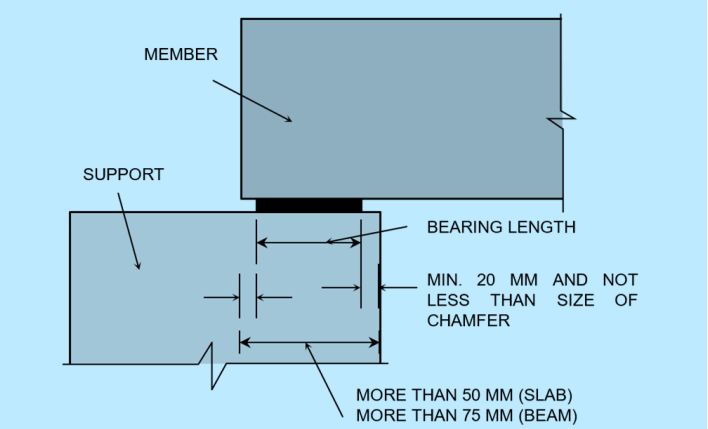

FIG. 18 BEARING LENGTH ON SUPPORT

7.11.7 *Detailing for Simple Bearing*

Over and above the net bearing width calculated above, allowances for spalling and constructional inaccuracies should be made in the nominal width selected (*see* Fig. 19). The effects of accidental displacement of a supported member during erection shall also be considered. Minimum anchorage lengths shall comply with requirements specified in **9**.

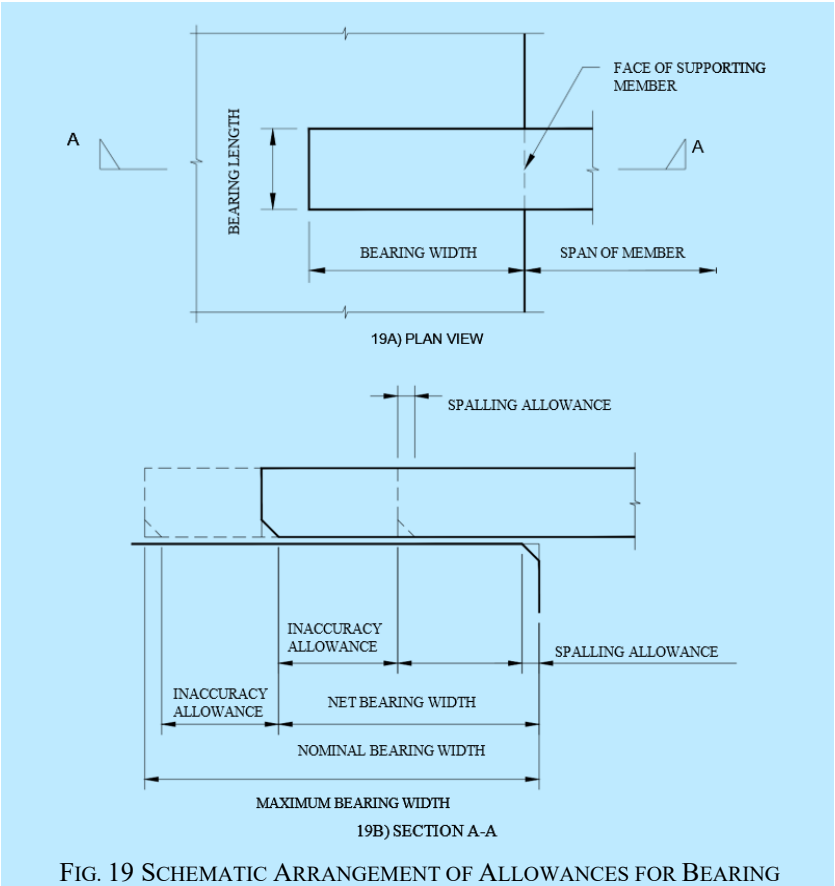

FIG. 19 SCHEMATIC ARRANGEMENT OF ALLOWANCES FOR BEARING

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7.11.8 *Allowances for Spalling at Supports*

The design shall account for reduction in effective bearing area due to potential spalling, in accordance with Table 3 and Table 4. When determining the outer edge of a support or the end of a supported member, chamfers occurring within the areas subjected to spalling may be excluded.

Table 3 Allowances for Effects of Spalling at Supports

(*Clause* 7.11.8)

Sl No.	Material of Support	Ineffective Distance, Measured from Outer Edge of Support mm
(1)	(2)	(3)
i)	Steel	0
ii)	Concrete ≥ M30	15
iii)	Concrete < M30	25
iv)	RC with outer edge less than 300 mm deep and vertical loop bar not greater than 12 mm	Nominal cover to reinforcement on outer face of support
v)	RC with outer edge less than 300 mm deep and vertical loop bar is 16 mm or above	Nominal cover plus inner radius of bend of bars

Table 4 Allowances for Effects of Spalling at Supported Members

(*Clause* 7.11.8)

Sl No.	Reinforcement at Bearing of Support Member	Ineffective Distance, Measured from Outer Edge of Support mm
(1)	(2)	(3)
i)	Straight bar, horizontal loops or vertical loops of 12 mm diameter or less	Greater of nominal cover or 10 mm
ii)	Tendons or straight bars exposed at end of member	0
iii)	Vertical loops of 16 mm diameter or above	Nominal end cover plus inner radius of bend of bars

7.11.9 *Shims*

Plastic or metal shims shall be used as per structural and durability requirements and shall be specified as per design requirements.

7.11.9.1 Steel shims shall not be used at areas susceptible to spalling. Load transfer through shims shall be included in the structural design.

7.11.9.2 The total thickness of individual shims shall not exceed 40 mm.

7.11.9.3 For joint gaps exceeding 40 mm, steel shims shall be used and shall be tack welded when shim thickness exceeds 40 mm to ensure load transfer and stability. Minimum thickness of shim plates shall be 5 mm.

7.11.10 *Allowance for Construction Inaccuracies*

The design shall include allowances for construction inaccuracies due to deviations in layout, manufacturing and erection. In absence of statistical deviation data, the minimum allowances shall be:

a) 15 mm or 3 mm/m for steel or precast concrete supports;

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b) 20 mm or 4 mm/m for masonry supports; and

c) 25 mm or 5 mm/m for in-situ concrete supports.

7.11.11 *Bearings Transmitting Compressive Forces*

The horizontal joints between load-bearing walls or columns shall be designed to resist all the forces and moments implicit in the assumptions made in analysing the structure as a whole and in designing the individual member to be joined. In the absence of more accurate information derived from a comprehensive programme of suitable tests, the area of concrete to be considered in calculating the strength of the joint in a wall or column shall be as per 8.3.2. Joints and joint reinforcement shall be detailed to prevent premature splitting or spalling at member ends.

7.11.12 *Other Forces at Bearings*

a) *Horizontal forces at bearing* — The horizontal forces at bearings may be induced by creep, shrinkage, temperature effects, misalignment, lack of plumb or other causes. When these forces are significant, the structural capacity of the supporting member may be impaired. A minimal force of 0.2 V_u (vertical or shear force on the bearing) shall be used in design. Allowances should be made by the provision of:

   1) Sliding bearings which allow longitudinal and lateral movement;

   2) Additional lateral reinforcement at the top of the supporting member; or

   3) Continuity reinforcement which ties the ends of the supported members together.

b) *Rotation at bearing* — Suitable bearings should be used to accommodate the rotations at end supports in particular for flexural members. Allowances should be made for any consequential increases in bending moments or bearing stresses due to rotations.

7.11.13 *Design of Bearing on Plain Concrete*

7.11.13.1 Plain concrete bearing (*see* Fig. 20) shall be used in situations where the bearing is uniform and the bearing stresses are low.

7.11.13.2 In other situations, and in thin-stemmed components where the bearing area is less than 12 500 mm^2, a minimum reinforcement equal to $N_u/0.65f_y$ (but not less than one bar of 10 mm diameter) is recommended. If horizontal load is not present, a minimum recommended value of $N_u = 0.2\ V_u$ shall be used.

where

N_u = factored axial force (minimum shall be 0.2 V_u); and

V_u = factored shear force.

7.11.13.3 The design bearing strength of plain concrete shall be calculated based on the following equation as given below:

Supporting area is wider in all sides than the loaded area (lesser of)

$$V_{uR} = (0.45 f_{ck} A_1)\sqrt{\frac{A_2}{A_1}}$$

$\sqrt{\frac{A_2}{A_1}}$ shall not exceed 2,

$$V_{uR} = 2(0.45 f_{ck} A_1)$$

Other cases

$$V_{uR} = 0.45 f_{ck} A_1$$

where

V_{uR} = design shear resistance;

A_1 = loaded area (bearing constant area) (*see* Fig. 20); and

A_2 = area of the lower base of pyramid, cone or tapered wedge (*see* Fig. 20).

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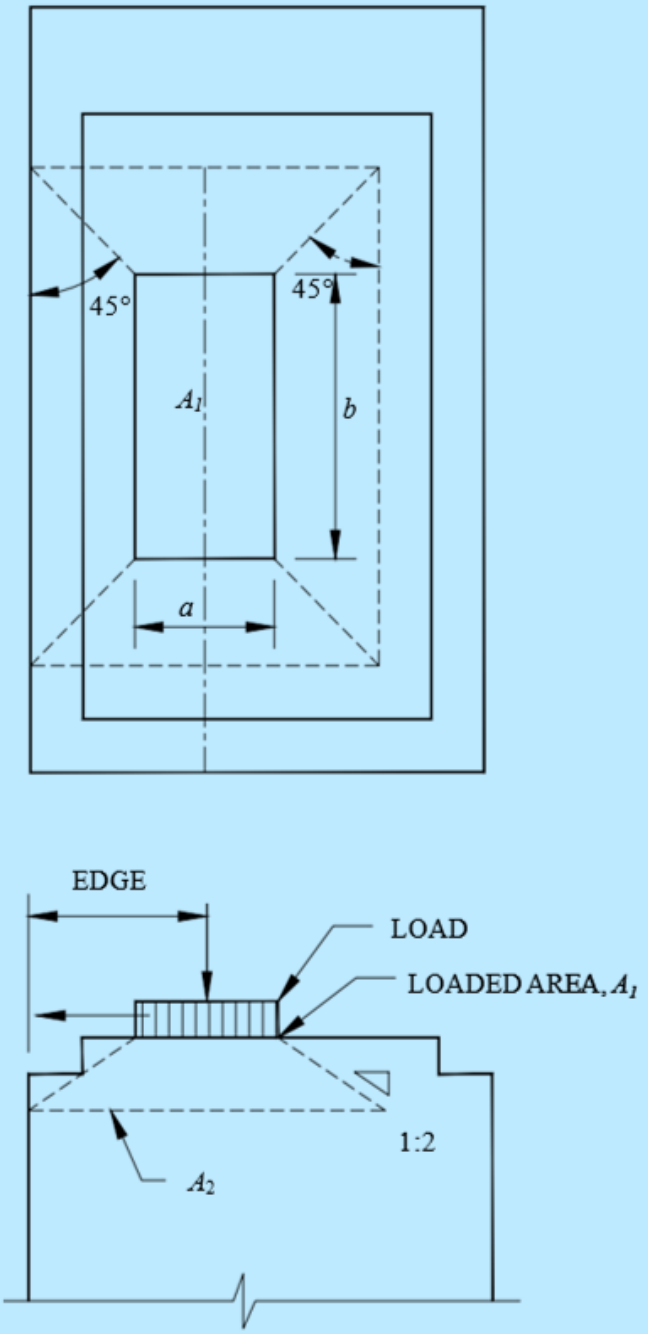

A_1 = LOADED AREA (BEARING CONTACT AREA)
A_2 = AREA OF THE LOWER BASE OF PYRAMID, CONE OR TAPERED WEDGE

FIG. 20 BEARING ON PLAIN CONCRETE

7.11.14 *Design of Reinforced Concrete Bearing* (*for thin Stem Elements and Rectangular Beams*)

7.11.14.1 If the applied load V_u, exceeds the design bearing strength V_{uR}, as calculated by equation in 7.11.13.3, reinforcement shall be provided in the bearing area as per Fig. 21.

7.11.14.2 The area of reinforcement $A_{vf} + A_n$ shall be calculated as per 7.12 and 7.11.16. This reinforcement shall be appropriately developed on each side of the vertical crack by development length, hooks, or welding to an anchor bar, bearing plate, or angle.

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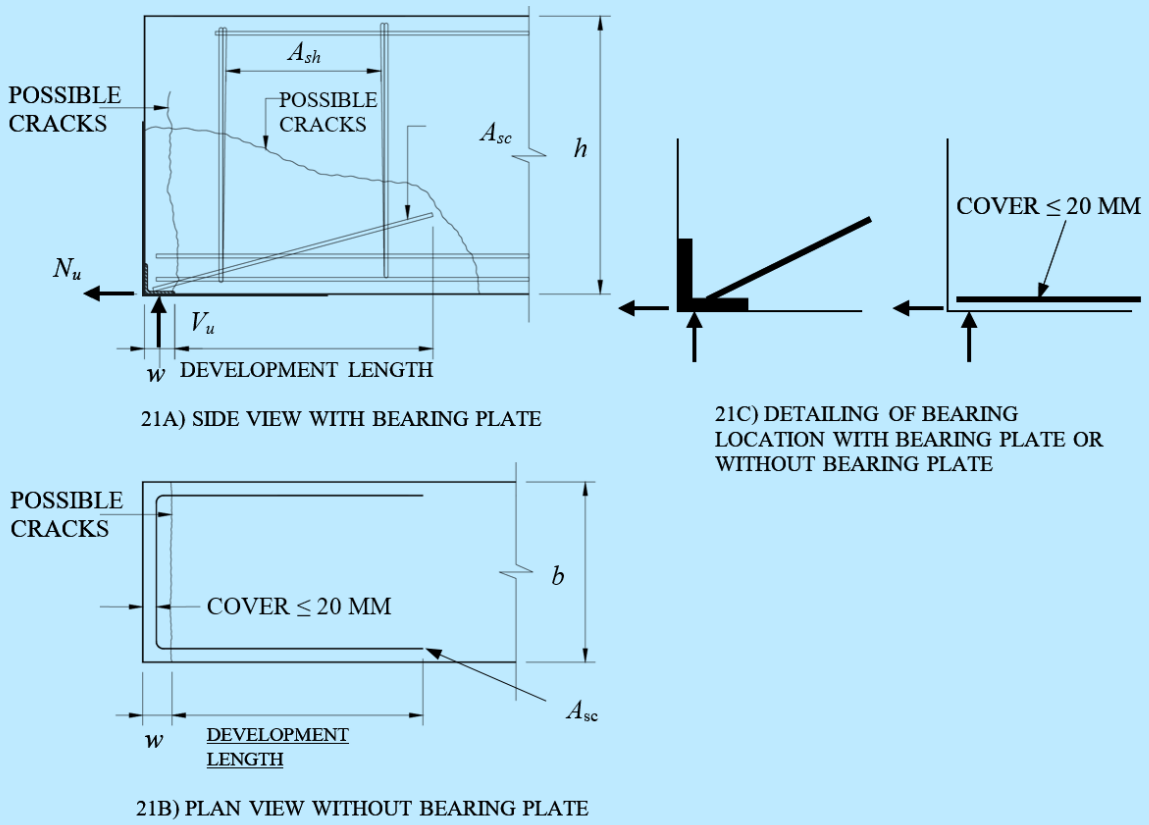

FIG. 21 REINFORCED CONCRETE BEARING

7.11.14.3 The area of vertical shear reinforcement shall be calculated as follows:

$$A_{sh} = \frac{(A_{vf} + A_n)f_y}{\mu f_{ys}}$$

where

A_{vf} = area of shear friction reinforcement;

A_n = area of tension reinforcement;

f_y = yield strength of tension reinforcement;

μ = coefficient of friction as per Table 6; and

f_{ys} = yield strength of shear reinforcement.

7.11.14.4 Diagonal tension reinforcement such as stirrups or welded-wire mesh may be counted as A_{sh} but shall not be added to separately designed shear reinforcement.

7.11.14.5 When components are subjected to bearing force more than $0.45f_{ck}A_1$ confinement reinforcement in all directions shall be provided. A_1 shall be considered as per Fig. 20.

7.11.15 *Dapped End Bearing* (*Thin Stem and Regular Beams*)

7.11.15.1 Dapped end members shall be designed for the potential failure modes shown in Fig. 22 to Fig. 24 with the reinforcement required for each consideration.

7.11.15.2 Dap reinforcement shall be provided in all cases where one or more of the following conditions:

a) The depth of the recess exceeds the lesser of 0.2 H or 200 mm;

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b) The length of the recess (L_p in Fig. 22) exceeds 300 mm for non-thin-stemmed components and the nib (the concrete section above the dap) is not designed for the full flexural and shear capacity; and

c) For non-thin-stemmed components, less than 1/3 of the main flexural reinforcement extends to the end of the component above the dap.

7.11.15.3 Thin-stemmed components such as double tees, T beam, etc, shall be designed with the following additional consideration:

a) Stem width is 125 mm or less;

b) Vertical hanger reinforcement or inclined reinforcement at an angle greater than 90° measured from horizontal; and

c) Dap height ≤ 0.5 h.

7.11.15.4 The design equations given in this section shall be considered for cases where shear span-to-depth ratio (a_v/d) do not exceed 1.0.

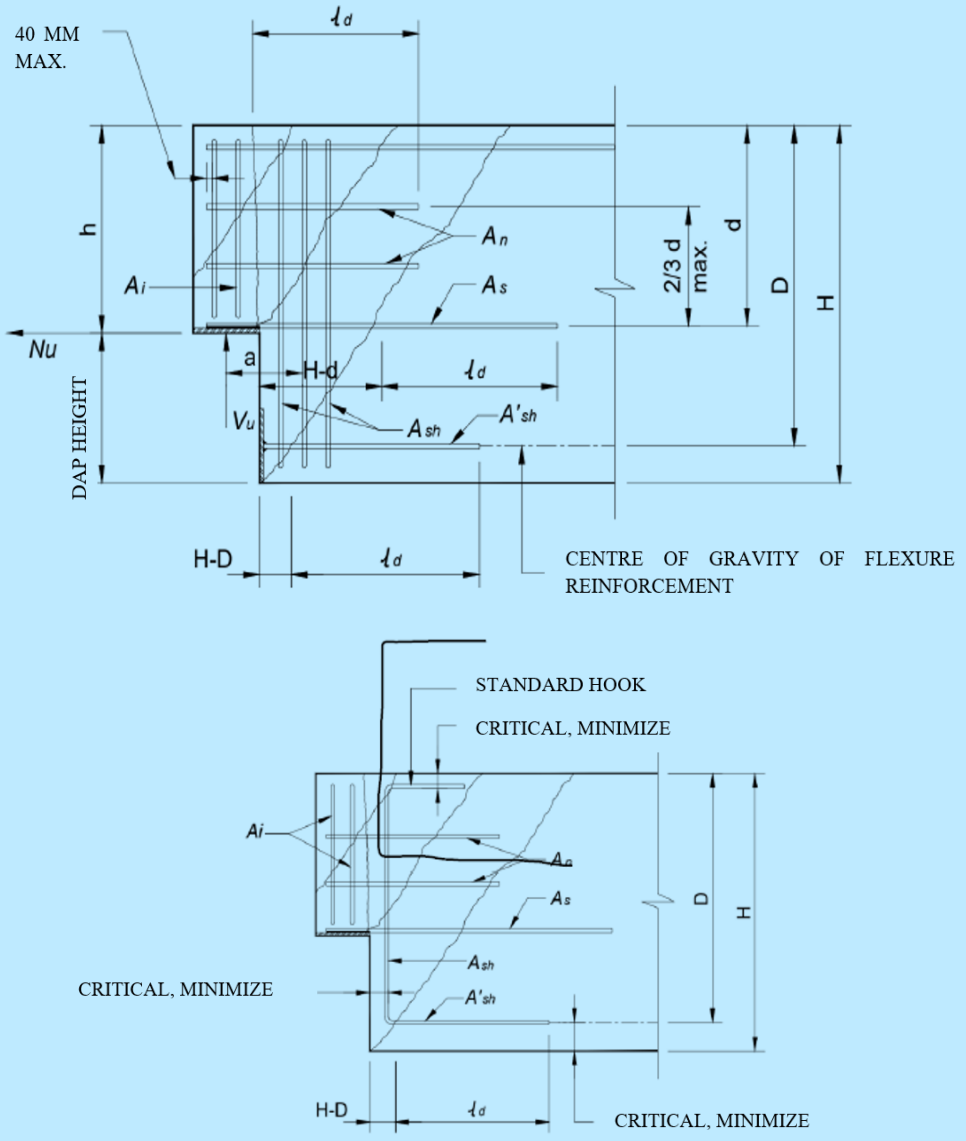

FIG. 22 POTENTIAL FAILURE MODES AND REQUIRED REINFORCEMENT IN NON-THIN STEMMED DAPPED ENDS

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7.11.15.5 High strength rebars may be used, however, the yield strength of reinforcement shall be restricted to 415 MPa.

7.11.15.6 For short, shallow recesses, the hanger reinforcement A_{sh}, and A'_{sh}, shall not be necessary. Confinement reinforcement A_v and flexural reinforcement $A_{vf} + A_n$, in accordance with 7.11.14 shall be provided.

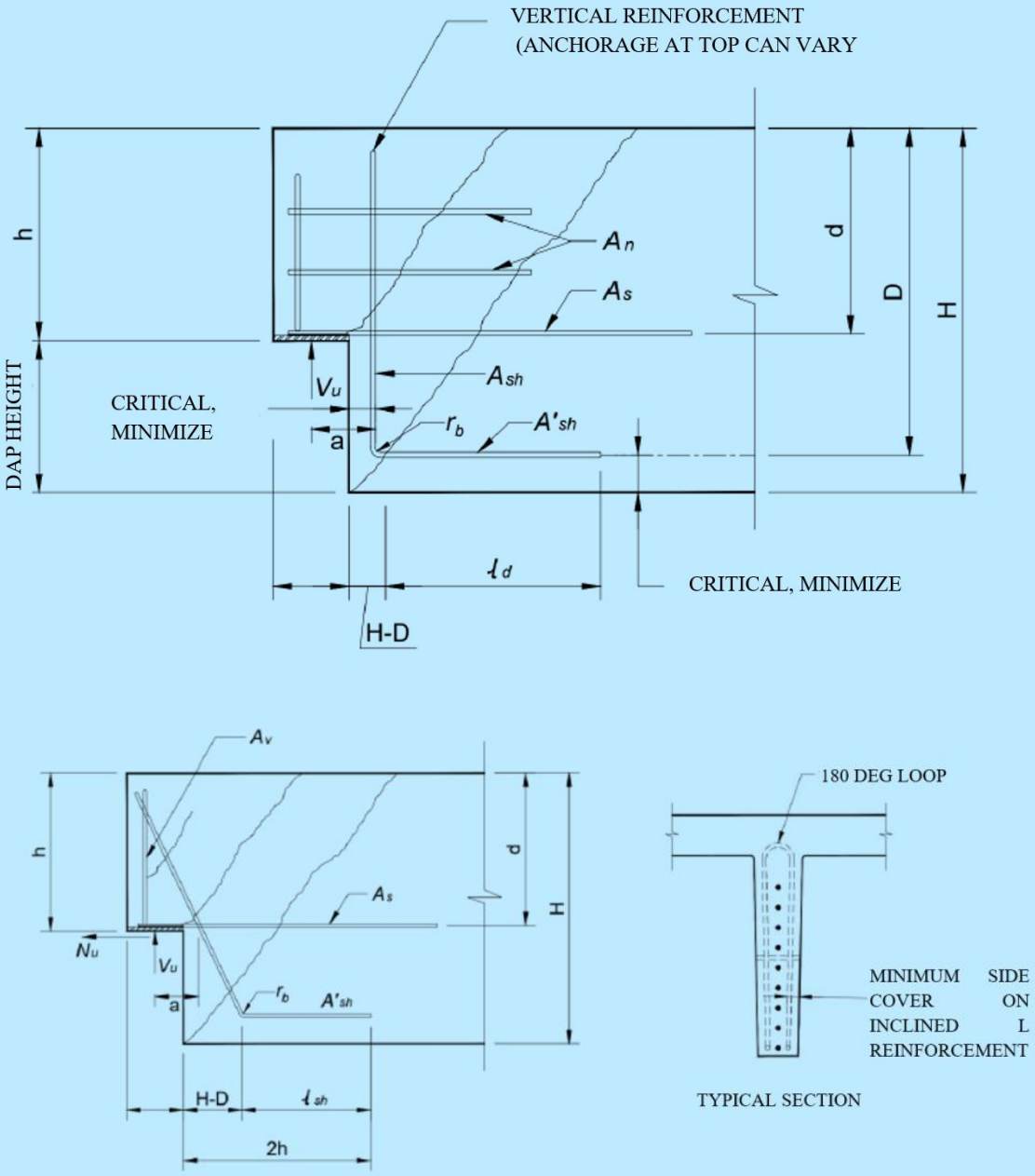

FIG. 23 POTENTIAL FAILURE MODES AND REQUIRED REINFORCEMENT IN THIN-STEMMED DAPPED ENDS

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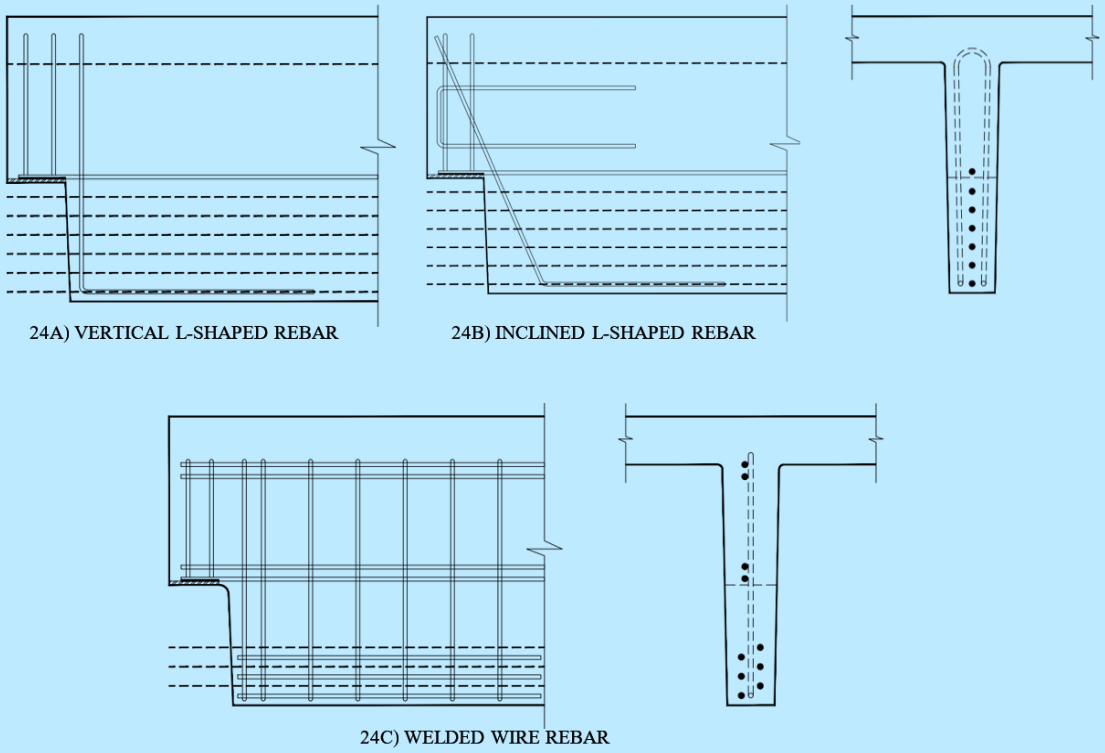

FIG. 24 CONFIGURATION OF DAP REINFORCEMENT FOR THIN-STEMMED DAPPED ENDS

7.11.15.7 The area of horizontal reinforcement A_s (area of reinforcement for flexure and axial tension in the extended end) shall be the larger of the two values calculated using the equations given below. The values of A_f, A_n shall be calculated as per 7.11.17 and A_{vf} is as defined under 7.12:

$$A_s = A_f + A_n \text{ , or}$$

$$A_s = \frac{2}{3}A_{vf} + A_n$$

7.11.15.8 The area of horizontal reinforcement A_h (area of direct shear reinforcement) shall be provided in the nib to arrest Crack 2 as per the following and shall be distributed within 2/3 d measured from the primary tension reinforcement A_s:

$$A_h = 0.5(A_s - A_n)$$

7.11.15.9 The area of hanger reinforcement A_{sh} shall be provided to arrest Crack 3, shall be as per the following:

$$A_{sh} = \frac{V_u}{0.75 f_y}$$

7.11.15.10 The area of vertical shear reinforcement A_v shall be provided to arrest Crack 4 as per the following:

$$V_{uR} = 0.75(A_v f_y + A_h f_y + 0.15\sqrt{f_{ck}}bd)$$

$$\text{Minimum } A_v = \frac{1}{2f_y}(1.34V_u - 0.15\sqrt{f_{ck}}bd)$$

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7.11.15.11 The anchorage of reinforcement shall be as per the following:

a) Horizontal bars A_s shall be extended a minimum of L_d beyond Crack 5 and anchored at the end of the component by welding to cross bars, plates, or angles;

b) Horizontal bars A_h shall be extended a minimum of L_d, beyond Crack 2 and anchored at the end of the component by hooks or other suitable means;

c) To ensure development of hanger reinforcement A_{sh}, the rebar shall be bent and continued parallel to the bottom of the component, or a separate horizontal anchored reinforcement with $A'_{sh} \geq A_{sh}$ shall be provided. The extension of reinforcement (A'_{sh}) at bottom of the component must be at least L_d beyond Crack 5. The A'_{sh}, reinforcement may be anchored on the dap side by welding it to a plate, angle, or cross bar;

d) The beam flexure reinforcement may also be used to ensure development of A_{sh} reinforcement, provided that the flexural reinforcement is adequately anchored on the dap side and the A_{sh} reinforcement is wrapped around the flexural reinforcement;

e) In thin-stemmed components, it is recommended that the extension of the hanger reinforcement be greater than L_d. A'_{sh} shall extend a distance that is the greater of 1.5 L_d of the strand past the face of the dap or 2.0 L_d of A'_{sh} beyond the diagonal Crack 5;

f) Vertical reinforcement A_v should be properly anchored by hooks, as required by Part C/Section 5 of this NBCS;

g) Thin-stemmed components, except for the vertical 'Z' configuration, which has the bar welded to the plate, should have at least one 12 mm diameter rebar welded to the bearing plate or have equivalent development length (L-rebar); and

h) Welded-wire reinforcement in place of bars may be used for reinforcement. It should be anchored in accordance with Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS. In thin-stemmed components the use of welded-wire reinforcement should be limited to situations where it is placed symmetrically in the width of the stem.

7.11.15.12 *Other considerations*

In addition to the reinforcement for the dapped end, it is important that the following be considered:

a) The depth of the extended end should not be less than about one half the depth of the beam, unless the beam is deeper than necessary for other than structural reasons;

b) The hanger reinforcement A_{sh} should be placed as close as practical to the re-entrant corner and as symmetrically as possible around the centroid of the section being considered. This reinforcement requirement is not additive to other shear-reinforcement requirements but should be in addition to torsion reinforcement;

c) The shear strength of the full-depth beam behind the dap must be verified;

d) For thin-stemmed components, for a distance of 2H beyond the dap the concrete contribution to shear strength should be limited to:

$$V_c = 0.15\sqrt{f_{ck}}b_w d_p$$

or

$V_c = 0.23\sqrt{f_{ck}}b_w d_p$, where at least two bonded strands in the extended end are present

where

b_w = average width of the stem for the full depth of the section; and

d_p = distance from the top to the centroid of the prestressing strand for the full depth section and shall not be less than 0.8 H.

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e) For thin-stemmed components, for a distance 2 *H* beyond the dap, the reinforcement contribution to shear strength V_s shall be limited as per the following:

$$V_s = 0.15\sqrt{f_{ck}}b_w d_p$$

7.11.16 *Ledges*

7.11.16.1 *General*

Beam and wall ledges (as shown in Fig. 25) shall be designed as per the following guidelines. Hanger steel requirements for the ledge in a wall need not be satisfied except where penetrations occur just below ledges. The design procedure explained in this section shall be applicable for the following cases:

a) $N_u \leq V_u$;

b) Ledges with heights of 200 mm to 450 mm;

c) Ledge projections of 150 mm to 250 mm;

d) Bearing widths of components on the ledge ranging from 100 mm to 300 mm;

e) Distances of end load to end of ledge ranging from 100 mm to 900 mm; and

f) Design concrete compressive strengths f_{ck} of 30 MPa to 80 MPa.

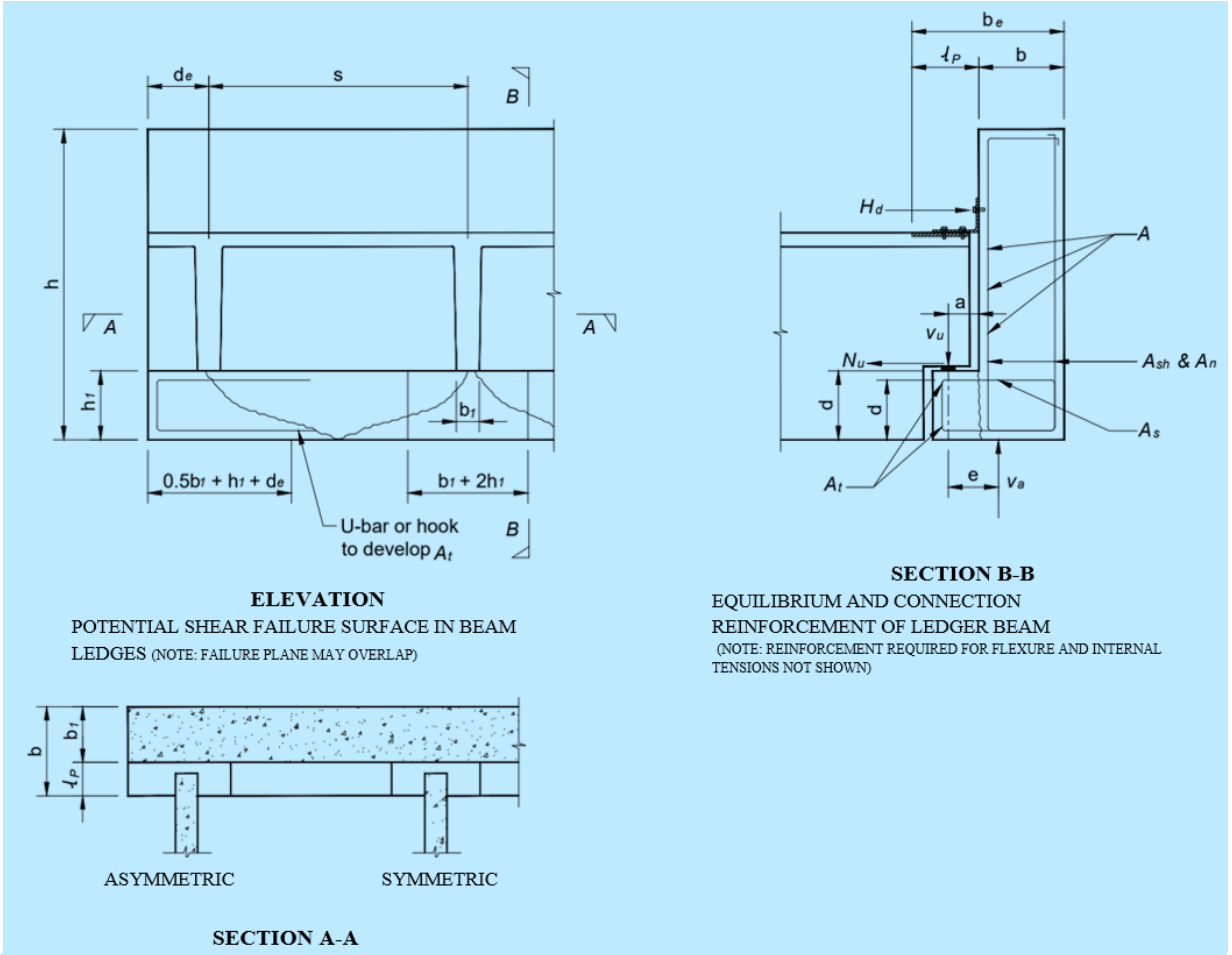

FIG. 25 DESIGN OF BEAM LEDGES

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7.11.16.2 For shear strength of ledge for concentrated loads, not near the ends, where $d_e \geq 0.5\ b_t + h_l + l_p$, a symmetric failure controls the design punching shear strength of the ledge shall be determined as the lesser of following two equations:

$$V_{uR} = 0.056\gamma\beta\sqrt{f_{ck}}h_l(b_t + 2h_l + 2l_p)$$

or

$$V_{uR} = 0.028\gamma\beta\sqrt{f_{ck}}h_l(b_t + 2h_l + s + 2l_p)$$

For concentrated loads that are near ends, where $d_e \leq 0.5\ b_t + h_l + l_p$, an asymmetric failure controls the design punching shear strength of the ledge and shall be as per the following:

$$V_{uR} = 0.056\gamma\beta\sqrt{f_{ck}}h_l(0.5b_t + h_l + d_e + l_p)$$

or

$$V_{uR} = 0.56\gamma\beta\sqrt{f_{ck}}h_l(0.5b_t + h_l + d_e + l_p)$$

where

$\gamma = \sqrt{1 + 12.5\frac{f_{pc}}{f_{ck}}}$, a factor for the effect of prestressing, 1 for precast beams;

β = coefficient of shear strength based on R (as per Fig. 26);

h_l = height of beam ledge;

b_t = bearing width;

l_p = length of ledge projection;

s = minimum spacing of concentrated loads;

d_e = distance from centre of load to the end of the ledge; and

f_{pc} = average prestress force after losses divided by gross concrete area of beam.

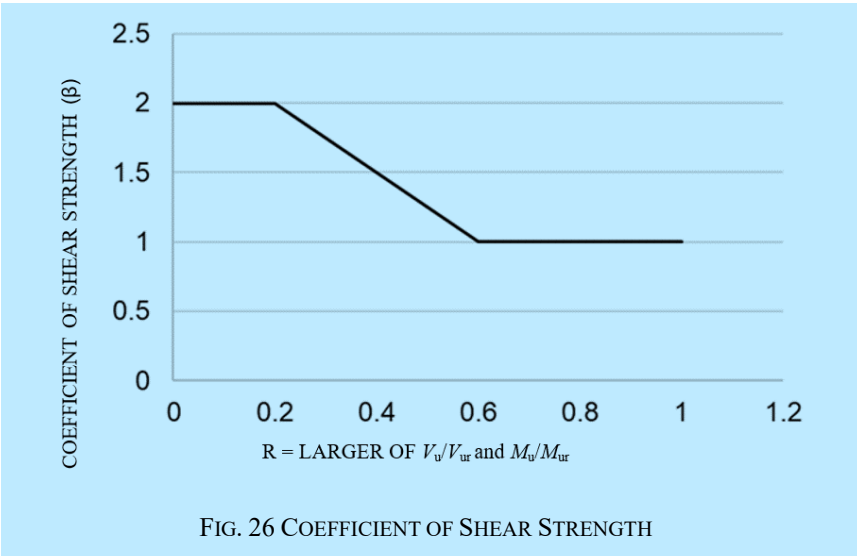

FIG. 26 COEFFICIENT OF SHEAR STRENGTH

7.11.16.4 For ledges supporting a continuous load or a series of closely spaced concentrated loads, punching shear strength per metre run shall be determined as per the following:

$$V_{uR} = 0.675\sqrt{f_{ck}}h_l$$

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7.11.16.5 If the applied factored load exceeds the strength as determined above, the ledge shall be designed for shear transfer and diagonal tension in accordance with 7.11.15.8 through 7.11.15.10.

7.11.16.6 Transverse (cantilever) bending of the ledge requires flexural reinforcement ($A_s = A_f + A_n$), shall be computed using 7.11.17.5. Such reinforcement shall be uniformly spaced over a width of $6h_l$ on either side of the bearing, but not to exceed half the distance to the next load. Bar spacing shall not exceed the ledge depth h_l or 450 mm whichever is less.

7.11.16.7 Longitudinal reinforcements shall be placed in both the top and bottom of the ledge portion of the beam and calculated as per the following:

$$A_l = \frac{200 l_p d_l}{0.75 f_y}$$

where

d_l = design depth of A_l reinforcement in Fig. 25.

7.11.16.8 Hanger steel A_{sh} shall be computed using following equation and distributed in the web of the beam. The reinforcement shall be uniformly spaced over a width of 6 h_l on either side of the bearing, but shall not exceed half the distance to the next load. The area of reinforcement shall not be additive to shear and torsional reinforcement.

$$A_{sh} = \frac{Vu}{0.75 f_y}(m)$$

where

m for L beam

$$m = \frac{(d_s + a) - \left(3 - 2\frac{h_l}{h}\right)\left(\frac{h_l}{h}\right)^2 \left(\frac{b_l}{2}\right) - e\gamma_t \frac{x_l^2 y_l}{(x_l^2 y_l + x_w^2 y_w)}}{d_s} \geq 0.6, and$$

m for inverted tee-beam

$$m = \frac{\left(\frac{b}{2} + e\right) - \left(3 - 2\frac{h_l}{h}\right)\left(\frac{h_l}{h}\right)^2 \left(\frac{b}{2}\right) - e\gamma_t \frac{x_l^2 y_l}{(x_l^2 y_l + x_w^2 y_w)}}{d_s} \geq 0.4$$

where

d_s = distance from outside face of web to A_{sH} reinforcement;

b_l = width of web plus ledge being designed;

x_l, y_l, x_w, y_w, = defined in Fig. 27.

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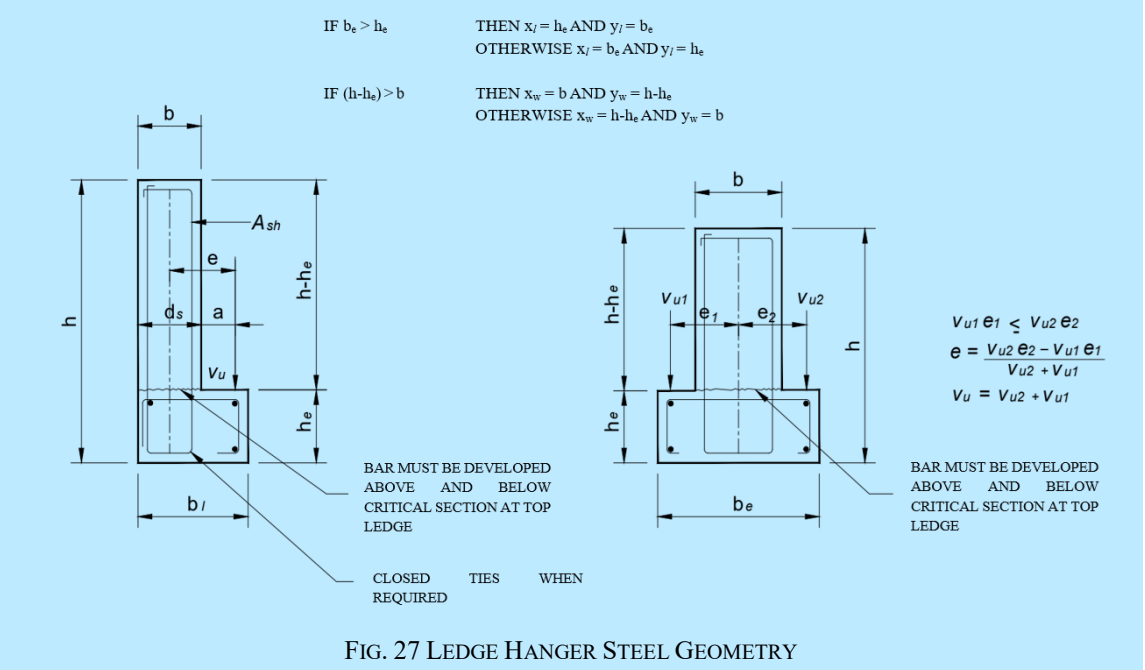

FIG. 27 LEDGE HANGER STEEL GEOMETRY

7.11.17 *Concrete Corbels*

7.11.17.1 *General*

Concrete corbels are short cantilevers that support other components and transmit the load to the supporting component and tend to act as simple trusses or deep beams (*see* Fig. 28).

7.11.17.2 The design based on strut and-tie method shall be as per Part C/Section 5 of this NBCS. For steel brackets, Part C/Section 6A of this NBCS shall be referred.

7.11.17.3 The cantilever beam method of designing a corbel presented in this section shall be used according to following limitations:

a) $a_v/d \leq 1$; and

b) $N_U \leq V_U$.

7.11.17.4 The dimensions of a corbel shall be selected such that the nominal shear strength shall not exceed the strength calculated as per Table 6.

7.11.17.5 The area of primary tension reinforcement A_{sc} shall be greater of $(A_f + A_n)$ and $(2/3)\ A_{vf} + A_n$

7.11.17.6 The area of reinforcement A_f shall be calculated as per the following equation:

$$A_f = \frac{V_u a_v + N_u(h-d)}{0.7 f_y d}$$

7.11.17.7 The area of reinforcement A_n shall be calculated as per the following equation:

$$A_n = \frac{N_u}{0.75 f_y}$$

7.11.17.8 The area of reinforcement A_{vf} shall be calculated as per shear friction (*see* 7.12).

7.11.17.9 Unless tensile forces are prevented from being applied to the bracket or corbel, N_u shall be considered as $0.2V_u$ or as specified in the project.

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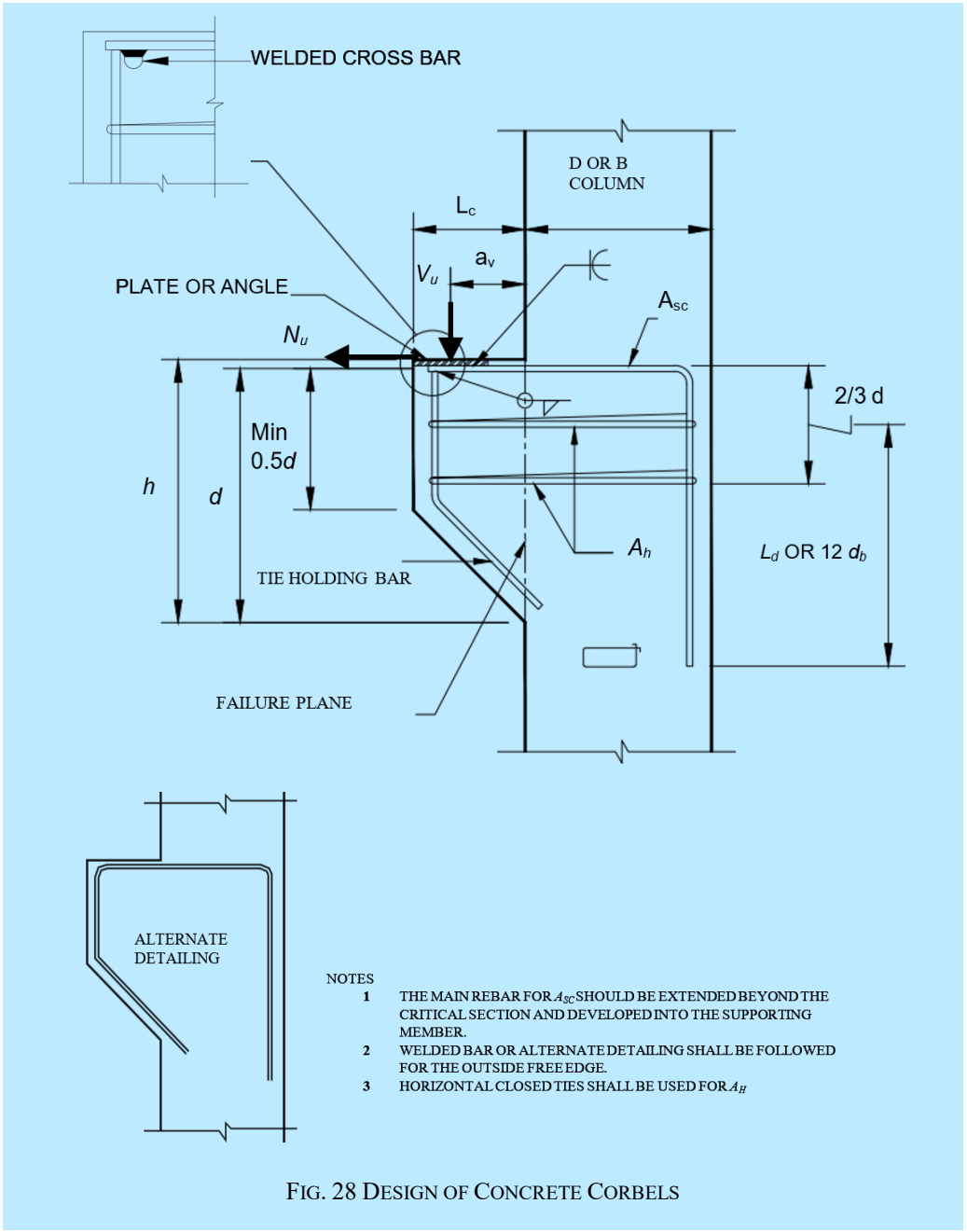

FIG. 28 DESIGN OF CONCRETE CORBELS

7.11.17.10 If bearing pads are used to avoid tensile forces, the value of N_u that corresponds to the force causing the pad to slip shall be considered.

7.11.17.11 Minimum reinforcement (A_{sc}) shall be calculated as per the following equation. Where the corbel depth is larger than required by design a reduced depth d may be used consistently in all the design equations:

$$A_{sc,min} = 0.032 \frac{f_{ck}}{f_y} bd$$

7.11.17.12 The area of stirrups (A_h) shall be calculated as per the following equation:

$$A_h \geq 0.5(A_{sc} - A_n)$$

7.11.17.13 The shear reinforcement shall be provided as closed stirrups and uniformly distributed within 2/3 d measured from the primary tension reinforcement.

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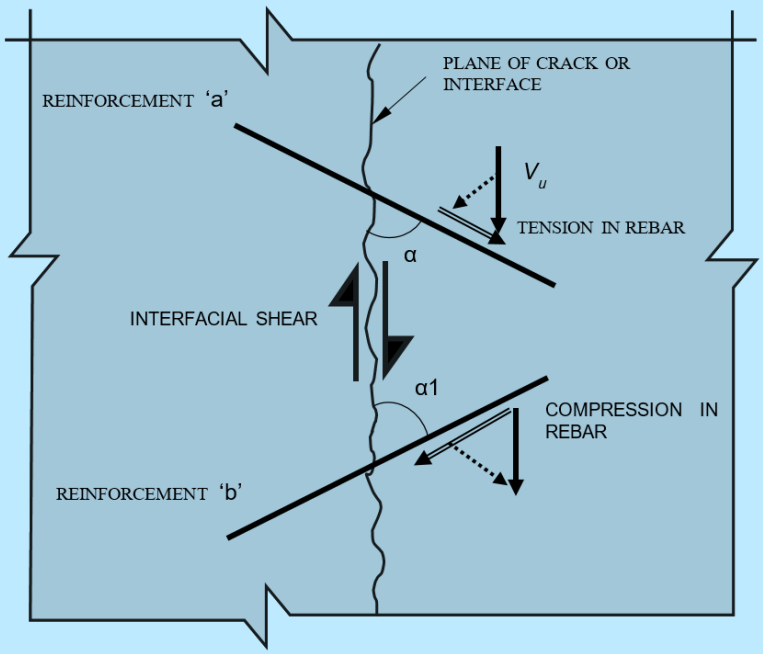
7.11.17.14 The development length for primary tension reinforcing bar shall be as per good practice [C-7(2)]. Development length for tension reinforcement shall be measured from the assumed failure plane (face of the supporting member). For member not having enough embedment length, hooked bars shall be used.

7.11.17.15 The main tension steel within the concrete corbel shall be developed at the outer edge of the corbel by welding to a cross bar, bearing plate with cross bar, angle section or bending along the depth of corbel.

7.12 Shear Friction

Shear friction shall be applied to consider shear transfer across interface between dissimilar materials or interface between concrete cast at different time or across a shear plane as shown in Fig. 29, reinforcement 'a'. The following clauses shall be used.



NOTE – Reinforcement 'a' shall be considered in shear transfer and reinforcement 'b' shall be neglected.

FIG. 29 SHEAR FRICTION REINFORCEMENT

7.12.1 *Nominal Shear Strength*

7.12.1.1 Value of V_{uR} across the assumed shear plane shall be calculated using equation in 7.12.1.2 or 7.12.1.3 depending on the angle of shear reinforcement.

7.12.1.2 If shear-friction reinforcement is perpendicular to the shear plane, nominal shear strength across the assumed shear plane shall be calculated by:

$$V_{uR} = 0.75\mu A_{vf} f_y$$

7.12.1.3 If shear-friction reinforcement is inclined to the shear plane and the shear force induces tension in the shear friction reinforcement, nominal shear strength across the assumed shear plane shall be calculated by:

$$V_{uR} = 0.75\ A_{vf} f_y\ (\mu\ sin\ sin\ \alpha + cos\ cos\ \alpha)$$

where

A_{vf} = area of reinforcement crossing the assumed shear plane to resist shear;

μ = coefficient of friction in accordance with Table 6; and

α = angle between shear-friction reinforcement and assumed shear plane.

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7.12.2 The value of V_{uR} across the assumed shear plane shall not exceed the limits in Table 5. Where concretes of different strengths are cast against each other, the lesser value of f_{ck} shall be used in Table 5.

Table 5 Maximum V_{uR} Across the Assumed Shear Plane

(*Clause* 7.12.2)

Sl No.	Surface Condition	Maximum V_{uR}	
(1)	(2)	(3)	
i)	Concrete placed monolithically or placed against intentionally roughened surface (amplitude of 6 mm or more)	$0.12f_{ck}A_{cr}, or$ $(2.475 + 0.049f_{ck})\ A_{cr}$, or $8.25A_{cr}$	Equation given in 7.12.1.2 least of the three
ii)	Other cases	$0.12f_{ck}A_{cr}$ or $4.125A_{cr}$	Equation given in 7.12.1.3 least of the two

Table 6 Coefficient of Friction

(*Clauses* 7.11.14.3, 7.11.17.4 *and* 7.12.1.3)

Sl No.	Contact Surface Condition	Coefficient of Friction
(1)	(2)	(3)
i)	Concrete placed monolithically	1.4
ii)	Concrete placed against hardened concrete that is clean, free of laitance and intentionally roughened to a full amplitude of approximately 6 mm	1.0
iii)	Concrete placed against hardened concrete that is clean, free of laitance and not intentionally roughened	0.6
iv)	Concrete placed against as-rolled structural steel that is clean, free of paint and with shear transferred across the contact surface by headed studs or by welded deformed bars or wire	0.7

7.12.3 The frictional component of compression load (if any) present at the interface shall be considered along with the shear friction capacity.

7.12.4 Shear friction capacity shall not be considered for a joint producing compression in the joint reinforcement as shown in Fig. 29 reinforcement 'b'.

7.13 Serviceability Design Checks

7.13.1 *Deflection*

Deflection calculations and permissible limits shall comply with the provisions of Part C/Section 5 of this NBCS. Partial fixity provided by the structural topping and nominal connections may be considered for serviceability checks.

7.13.2 *Crack*

The relevant provisions of good practice [C-7(2)] shall be followed for crack width calculations and limits. If structural cracks are observed on the tension face of the element, testing of the element shall be carried out in accordance with **10** or as directed by engineer-in-charge and the client.

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7.13.3 *In-Service Considerations*

The relevant provisions of good practice [C-7(2)] shall be followed for service consideration calculations and limits.

7.14 Design of Demoulding, Handling Systems and Temporary Forces

7.14.1 *Demoulding Forces*

Forces on the elements during lifting from the casting bed due to suction or adhesion between the precast elements and the mould should be accounted while deciding the strength of concrete. These are accounted for by applying an equivalent load factor to the self-weight of the member and treating it as an equivalent static force to evaluate the stresses in the precast element. Table 7 gives recommended minimum values of equivalent load factor for demoulding forces for different product types and finishes.

Table 7 Recommended Load Factors to Account for Forces in Demoulding of Elements

(*Clauses* 7.14.1, 10.8 *and* 10.8.3)

Sl No.	Product Type	Finish	
		Exposed Aggregate with Retarder	Smooth Mould (form oil only)
(1)	(2)	(3)	(4)
i)	Flat with removable side forms. No formed rebates or reveals	1.2	1.3
ii)	Flat with removable side forms with formed rebates or reveals	1.3	1.4
iii)	Fluted with proper draft	1.4	1.6
iv)	Sculptured	1.5	1.7

NOTES

1 These factors are to be applied to the flexural design of precast elements only. For lifting inserts, *see* Table 10.

2 The above values are recommended values only. Guidance should also be sought from the precast manufacturer based on their pre-casting experience if the load factors need to be increased.

3 The associated imposed loads or wind loads, if any, are to be assessed and considered under the appropriate load combination.

7.14.2 *Handling Stresses*

Precast elements shall be designed and handled to prevent permanent damage during demoulding, storage, transportation, and erection. Design shall account for all loads acting during these stages. Minor cracks within acceptable design crack widths shall be repaired using methods approved by the contract or chemical supplier guidelines.

7.14.3 *Handling and Transportation*

An allowance should be made for dynamic loads and impact forces arising in handling, transportation and erection. Similar to demoulding force consideration, Table 8 gives recommended values for equivalent load factor to be applied to the self-weight of members to allow for these forces.

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Table 8 Recommended Load Factors to Account for Dynamic Forces Arising During Handling, Transportation and Erection of Elements

(*Clause* 7.14.3)

Sl No.	Stage	Load Factor
(1)	(2)	(3)
i)	Yard handling	1.2
ii)	Transportation	1.5
iii)	Erection	1.2

NOTES

1 These factors are to be applied to the flexural design of precast elements only. For lifting inserts, refer **7.14.6**.

2 The above values are recommended values only. Under certain conditions higher factors may apply that is certain unfavourable load conditions.

3 The associated imposed loads or wind loads, if any, are to be assessed and considered with the appropriate load combination.

7.14.4 *Early Demoulding/Handling of Precast Element*

For precast elements lifted and handled prior to gaining full strength the elements together with any lifting inserts should be designed accordingly. Recommended minimum concrete strengths for lifting and handling of precast elements are given in Table 9. Precast elements should be designed, using these lower strengths, to span between lifting points without excessive cracking or deflection. For prestressed concrete, consideration should also be given to stresses resulting from transfer of prestressing forces.

Table 9 Recommended Minimum Concrete Strength for Lifting and Handling

(*Clauses* 7.14.4 *and* 10.8.4.1)

Sl No.	Application	Minimum Concrete Strength N/mm^2
(1)	(2)	(3)
i)	None specified, fine controlled crane, non-prestressed	10*
ii)	Lifting which involves significant impact or high acceleration	15*
iii)	All units where concrete strength for lifting is specified in the specification	As specified
iv)	Concentrically prestressed elements (piles, wall panels, or thin floor slabs)	20
v)	Eccentrically prestressed (T beam, deep flowing units)	25
vi)	Highly stressed prestressed elements	30 or as specified

*Depending on anchor length or as recommended by insert manufacturer.

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7.14.5 *Temporary Stages/Erection Sequence*

The design of precast elements at temporary stages can be critical compared to the permanent stages. Consideration should be given to the loading imposed on the precast elements during each phase of construction. The following shall be verified:

a) Precast sections of composite elements which are required to support self-weight plus construction load prior to casting of an in-situ topping;

b) Lower precast floor slabs or precast stair flights which support propping to upper levels during installation; and

c) Bearing or halving joints which support higher temporary construction loads because of back propping to upper levels.

The design should also take into consideration that the structural action and framing might be different during the temporary stages resulting in higher stresses in individual members.

7.14.6 *Lifting Inserts*

The number of lifting inserts and location of inserts shall be decided by the following:

a) Lifting insert capacity (safe working load);

b) Total weight of the element;

c) Strength of concrete at age of lifting;

d) Shape of the unit;

e) Location of the inserts so that the failure of any one insert does not cause failure of the entire lifting system thereby ensuring the element can still be safely supported;

f) Position of any cut-outs and/or openings;

g) Rigging arrangement; and

h) Anchorage of inserts.

All lifting inserts shall be designed by accounting the factors of safety given in Table 10. Typically lifting inserts shall be designed with a factor of safety of 4.

Table 10 Recommended Factors of Safety for Lifting Inserts

(*Table 7 and Clause 7.14.6*)

Sl No.	Item	Recommended Factor of Safety
(1)	(2)	(3)
i)	Bracing members	2
ii)	Bracing connections	3
iii)	Bracing inserts cast into precast members	3
iv)	Lifting inserts, normal circumstances	4
v)	Lifting inserts, multiple usage	5
vi)	Rebar inserts, single usage	2
vii)	Rebar inserts, multiple usage	4

7.14.7 *Bracing Design*

Bracing should be designed for both construction and wind loading. The construction load should be a minimum value of 1.5 kN/m² but should be increased if considered appropriate.

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Wind loads shall be calculated in accordance Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS and the bracing shall be designed for it.

8 JOINTS AND CONNECTIONS

8.1 Basic Design Considerations

A joint shall be considered based on the action of force transfer (tension, compression, shear, moment, etc) at an interface (one or many) between the precast components. A connection, which is a zone that includes various joints, shall be designed in combination of any of the following forces:

a) Compressive force;

b) Tensile force;

c) Shear force; and

d) Flexural and torsional moments, as applicable.

8.1.1 *Transfer of Compressive Force*

8.1.1.1 The precast or cast in-situ member supporting the other elements shall be designed to safely resist all the compressive forces generated by various load combination.

8.1.1.2 The joint shall be designed for all other forces arising from the external forces along with the predominant compression. Shear forces more than 0.1 P_u (axial load) shall not be neglected in the design of the joint.

8.1.1.3 The members may rotate because of temperature change, creep and shrinkage, hence bearing pads or strips shall be provided to account for the rotation. Minimum thickness of the joint shall be decided based on tolerance requirements, and shall not be less than 10 mm.

8.1.1.4 Appropriate edge distance for the bearing pad shall be provided to avoid crushing or cracking of edges.

8.1.1.5 Connection having bearing materials harder than concrete shall be checked for ultimate limit state as per the provisions of good practice [C-7(2)]. Busting reinforcement shall be provided in the supporting member, if required.

8.1.1.6 Connection having bearing materials softer than concrete shall be checked for serviceability limit states as per the provisions of good practice [C-7(2)]. Splitting reinforcement shall be provided in the supporting member, if required.

8.1.1.7 Joint using steel inserts for connection shall be properly designed and executed as per the design requirements. The steel components shall be anchored to the concrete members for proper transfer of compressive forces to the main reinforcement.

8.1.1.8 Connections shall be provided with reinforcement as follows or with equivalent connection:

a) For walls elements, dowels shall be placed at a maximum spacing of 1/4th of the length of the wall panels or 450 mm or as required by design. Also, at least two extreme edge dowels shall be continuous from foundation till roof using Class H coupler as per [C-7(9)] or any other connection;

b) For column minimum four corner bars of 12 mm diameter or as per design requirements shall be continuous from foundation till roof using Class H coupler as per [C-7(9)] or any other connection;

c) For beam at least one 12 mm dowel at centre of the bearing region of the beam shall be provided to avoid accidental failure; and

d) For any other member appropriate reinforcement shall be provided to avoid accidental lateral loads.

8.1.2 *Transfer of Tensile Forces*

8.1.2.1 Tensile forces generated at the joint shall be resisted by tie bars sufficiently anchored at both the sides of the joint.

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8.1.2.2 Adhesive bond between the members shall not be accounted in force transfer.

8.1.2.3 The tensile stresses generated in the connecting members shall also be linked to the reinforcement of the member by proper provision of ties or any other connection.

8.1.2.4 Premature brittle failure shall be avoided for tie members and material selected for making tie shall be ductile.

8.1.2.5 Joints with grout filled tubes or sleeves shall be utilised with adequate development length. International practice indicates that the confinement of dowel tube enhances the rebar anchorage performance, reducing the required development length. Hence, for determining development length reduced development length shall be adopted based on test results as per 12. In absence of test results, the bond strength of concrete shall be considered. Alternatively, other joint configurations may be adopted, provided their performance is evaluated through assemblage and system level testing along with nonlinear analysis and approved from competent authorities or institutes of repute.

8.1.2.6 A joint using bolting, welding or lap splicing shall be properly anchored to the precast components and transfer of forces in the member shall be verified.

8.1.2.7 Reinforcement loops (U-bars) protruding from precast elements may be used to provide tie between the elements as shown in Fig. 30. Reinforcement of 8 mm or higher diameter shall be provided as longitudinal reinforcement (preferably four numbers with U-bars) as shown in the Fig. 30. Lap length, spacing of loop bars and overlapping length shall be as per the Fig. 30.

8.1.2.8 Anchoring of reinforcement shall be such that high stress concentration in the members are avoided.

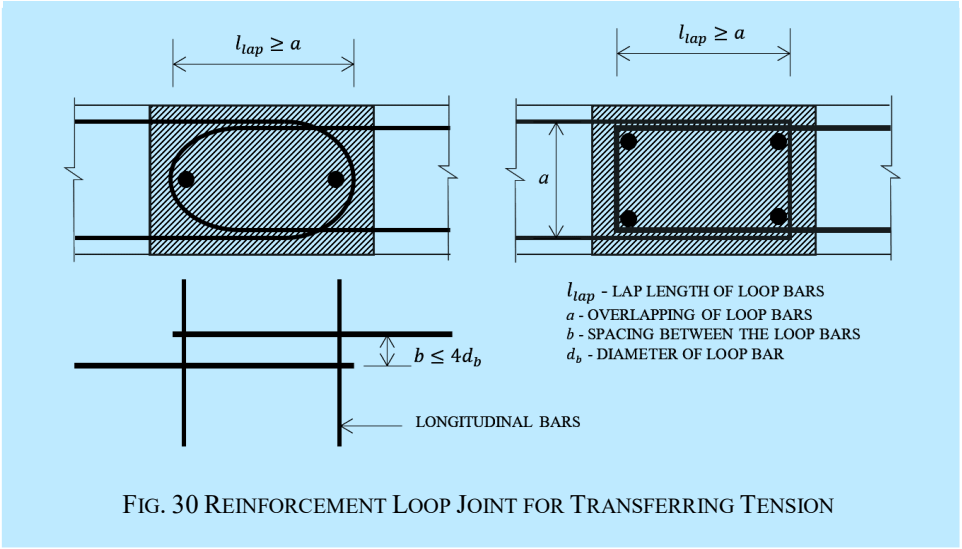

FIG. 30 REINFORCEMENT LOOP JOINT FOR TRANSFERRING TENSION

8.1.3 *Transfer of Shear Force*

8.1.3.1 Shear forces may be transferred between concrete elements by adhesion or friction at joint interfaces, shear-key effect at indented joint faces, dowel action of transverse steel bars, pins and bolts, or by other mechanical connection devices. The frictional resistance can be enhanced by the pull-out resistance of tie bars properly placed across the joint.

8.1.3.2 Adhesive bond or adhesion between the interfaces of elements shall not be considered in the calculation of shear strength of the joint.

8.1.3.3 Shear resistance of joint between two elements with reinforcement may be calculated using shear friction (*see* 7.12) equation or as per good practice [C-7(2)]. Shear friction shall not be used if the reinforcement crossing the joint is subjected to compressive forces by the displacement of the members.

8.1.3.4 Permanent compressive stresses acting on the joint shall also be considered along with shear friction.

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8.1.3.5 Unreinforced shear keys or castellation or indented joint face shall be used for members subjected to very high shear forces. This connection shall be designed to remain elastic under ultimate limit states to avoid sudden brittle failure of the joint. Transverse reinforcement across the interfaces shall be provided to prevent sudden separation of the members.

8.1.3.6 The dimension of shear keys shall be as per Fig. 31.

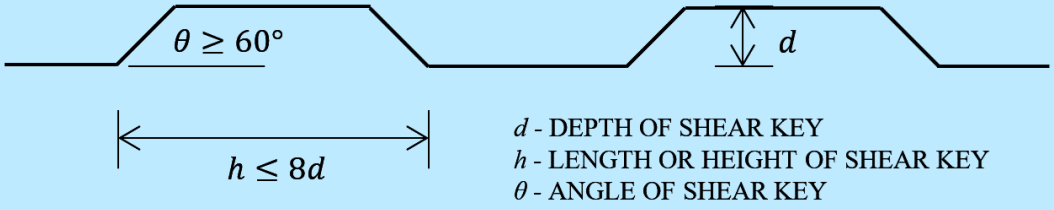

FIG. 31 DETAILING OF SHEAR KEY WITHOUT REINFORCEMENT

8.1.4 *Transfer of Bending and Torsional Moment*

8.1.4.1 Strong connections shall be used possessing stiffness, strength and ductility confirming to that of cast in-situ construction.

8.1.4.2 Moment resisting connections shall be designed such that ductile failures occur without any local modes of brittle failure (shear, pull-out, etc).

8.1.4.3 Continuity within the joint may be obtained by any of the following or combination:

a) Lapping of bars;

b) Reinforcement grouted into apertures;

c) Overlapping loop or U-bars;

d) Reinforcement connected using threaded couplers; and

e) Any other type of connection with appropriate testing of the connection.

8.2 Connection Based on Precast Concrete Seismic Systems

The connection of seismic moment resisting frames and structural walls incorporating precast concrete elements generally fall into two broad categories, either ‘equivalent monolithic (emulative)’ systems or, ‘jointed’ systems. The distinction between these types of construction is based on the design of the connections between the precast concrete elements as provided in the following sections.

8.2.1 *Equivalent Monolithic Systems* (*Emulative*)

A precast concrete structural system satisfying the requirements of this clause shall have strength and toughness equivalent to that provided by a comparable monolithic reinforced concrete structure.

8.2.1.1 *Connections in monolithic systems*

The connections between precast concrete elements of equivalent monolithic systems can be subdivided into two categories:

a) *Strong connections of nominal ductility*

In moment resisting frames and structural walls these connections are protected by a capacity design approach which ensures that flexural yielding occurs away from the connection region.

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b) *Ductile connections*

Ductile connections of equivalent monolithic systems typically comprise longitudinal reinforcing bars in the connection which are expected to enter the post-elastic range in a severe earthquake.

In moment resisting frames yield penetration may occur into the connection end-region. The potential plastic hinge region may extend a distance along the end of the member as in cast-in-place construction.

8.2.2 *Jointed Systems*

This system is not in the purview of this NBCS.

8.3 Design of Connections

The design of connections shall follow those design methods and considerations for reinforced concrete, prestressed concrete and structural steel. Proprietary connections shall be based on tests and approved from competent authority or institutes of repute. The following types are the broad category of the connections based on connecting members:

a) Column to foundation;

b) Column to column;

c) Beam to column;

d) Beam to beam;

e) Beam to slab;

f) Beam to wall;

g) Wall to wall;

h) Wall to slab; and

j) Slab to slab.

Examples of typical connections are shown in Annex E. These connections are for illustrative purposes only. In all instances, the design should demonstrate that the connections are able to resist the applied structural actions. Where appropriate, the suitability of the connection to act as a pinned or moment connection is highlighted. The location of the connections for a framed structure shall be as per **6** (Fig. 2, Fig. 3, Fig. 4 and Fig. 5).

8.3.1 *Detailing of Reinforcement*

The following shall be considered:

a) *Lapping of bars* — Where bars passing through the connection are lapped to provide continuity of reinforcement, the recommendations good practice [C-7(2)] and integrity tie requirements shall be followed;

b) Reinforcement grouted into aperture — An adequate capacity should be provided for grouted reinforcing bars to prevent pull-out. Embedment length shall be as per manufacturer guidelines, however testing shall be carried out as per **12**. If grouted tube is used, the embedment length of dowel shall be at least 300 mm or as required by design, whichever is higher;

c) *Reinforcement loops* — Loop reinforcement shall be provided as per **8.1.2** and Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS;

d) *Couplers* — Reinforcement may be connected by couplers. The concrete cover to the couplers (other than grouted tube or duct) shall be at least 25 mm or as per specifications of the project. A locking device should be used for threaded coupler connections where there is a risk of the threaded connection working loose, for example during vibration of in-situ concrete. Reference should be made to [C-7(9)] and the manufacturers' technical specifications for guidance and acceptance criteria for the usage. Testing of couplers shall be as per **12** and relevant guidelines of [C-7(9)]. When the system is part of lateral force resisting system, the grout strength shall not be less than 90 MPa and testing shall be carried out as per **12** by the supplier of the coupler or as agreed upon by the project team. Modify to mention 12 for LFRS and 7(9) for regular coupler without lateral force; and

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e) *Welding of bars* — When bars are connected by welding, the connection shall ensure full capacity of the bar is achieved and failure of weld is avoided. Testing of the welded samples shall be as per relevant standards. Welding of rebar shall be avoided for normal circumstances, to be used for repair or strengthening of a connection, if required.

8.3.2 *Connections in Compression*

8.3.2.1 The design of connections in tension shall follow the guidelines of 8.1.1.

8.3.2.2 In normal circumstances, the area of concrete to be utilized for calculating the strength of the connection in a wall or column should be the greater of:

a) 75 percent of the area of contact between the wall or column and connection; or

b) The area of the in-situ concrete excluding the part of any intruding slab or beam units. However, those parts of the floor slab or beam units that are solid over the bearing may be included in calculating the strength and such units should be properly bedded on concrete or mortar of adequate quality. This area should not be taken as greater than 90 percent of the wall or column area.

8.3.3 *Connections in Tension*

8.3.3.1 The design of connection in tension shall follow the guidelines of 8.1.2.

8.3.3.2 Stirrups shall be placed along the lap length, for the connection utilizing lapping of bars, grouted duct, or grouted coupler for transfer of tensile forces.

8.3.3.3 Welding or bolting shall be used with appropriate design. The embedded inserts shall be appropriately anchored to the primary member.

8.3.4 *Connection in Shear*

8.3.4.1 The design of connection in shear shall follow the guidelines of 8.1.3.

8.3.4.2 A shear connection may be considered effective, if the connection is grouted with a suitable concrete or mortar mix of strength of at least 5 MPa higher than the connecting members and the criteria of 8.1.3 is satisfied.

8.3.4.3 For joints subjected to minimal shear between wall panels or thin members, looped wire ropes shall be used to connect between the members with a maximum spacing of 450 mm. These connections shall not be used in lateral force resisting system and in transfer of seismic forces.

8.3.4.4 Transfer of forces solely by friction caused due to gravity load shall not be used.

8.3.5 *Bond*, *Anchorage*, *Bearing*, *Laps*, *Joints and Bends in Bars*

8.3.5.1 Provision of bond, anchorage, bearing, laps, joints and bend in bars shall be as follows:

a) *Avoidance of bond failure due to ultimate loads* — At both sides of any cross-section the force in each bar should be developed by an appropriate embedment length or other end anchorage;

b) *Anchorage bond stress* — Anchorage bond stress (τ_b) is assumed to be constant over the effective anchorage length. It may be taken as the force in the bar divided by its effective surface anchorage area, as given below:

$$\tau_b = \frac{F_s}{\pi \varphi_E L}$$

where

τ_b = bond stress;

F_s = force in the bar or group of bars;

φ_E = effective bar size which, for a single bar is equal to the bar size and, for a group of bars; and

L = anchorage length.

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c) *Values for design ultimate anchorage bond stress* — Values for design ultimate anchorage bond stress τ_{bu} may be obtained from the equation. The anchorage bond stress (τ_b) shall not exceed the design ultimate anchorage bond stress:

$$\tau_{bu} = \beta\sqrt{f_{cu}}$$

where

τ_{bu} = design ultimate anchorage bond stress; and

β = a coefficient dependent on the bar type.

The values of β may be taken from Table 11. These values include a partial safety factor (γ_m) of 1.4.

Table 11 Values of Bond Coefficient β

[*Clause* 8.3.5.1(c)]

Sl No.	Bar Type	β	
		Bar in Tension	Bar in Compression
(1)	(2)	(3)	(4)
i)	Plain bar	0.28	0.35
ii)	Deformed bar	0.4	0.63

8.4 Connections Other than Those Involving Continuity of Reinforcement

8.4.1 *Joints with Structural Steel Inserts*

Joints with structural steel inserts generally consist of a steel plate or rolled steel section projecting from the face of a column to support the end of a beam. The reinforcement in the ends of the supported member shall be designed in accordance with the provisions mentioned in Part C/Section 5 of this NBCS. Detailed design provisions shall include the following:

a) The steel sections and any bolted or welded connections should be designed in accordance with the appropriate recommendations;

b) Design ultimate bearing stresses shall be as per **7.11**; and

c) Consideration should be given in the design to the possibility of vertical splitting under the steel section because of shrinkage effects and localized bearing stresses, for example under narrow steel plate.

8.4.2 *Resin Adhesives*

Resin adhesives may be used to form joints subjected to compression but not to resist tension or shear. These adhesives should only be used where they are adequately protected against the effects of fire.

8.4.3 *Other Types of Connections*

Any other type of connection that can be shown to be capable of carrying the ultimate loads acting on it may be used. Amongst those suitable for resisting shear and flexure are those made by prestressing across the joint.

8.5 Structural Integrity and Progressive Collapse

Reinforcement and connections shall be detailed to tie the structure together effectively maintaining adequate structural integrity and to prevent progressive collapse. Integrity ties shall be followed in accordance with the following clauses. Figure 32 shows the typical arrangements of integrity ties in a large panel structure and frame structure. Progressive collapse requirements shall be as per **8.5.10**.

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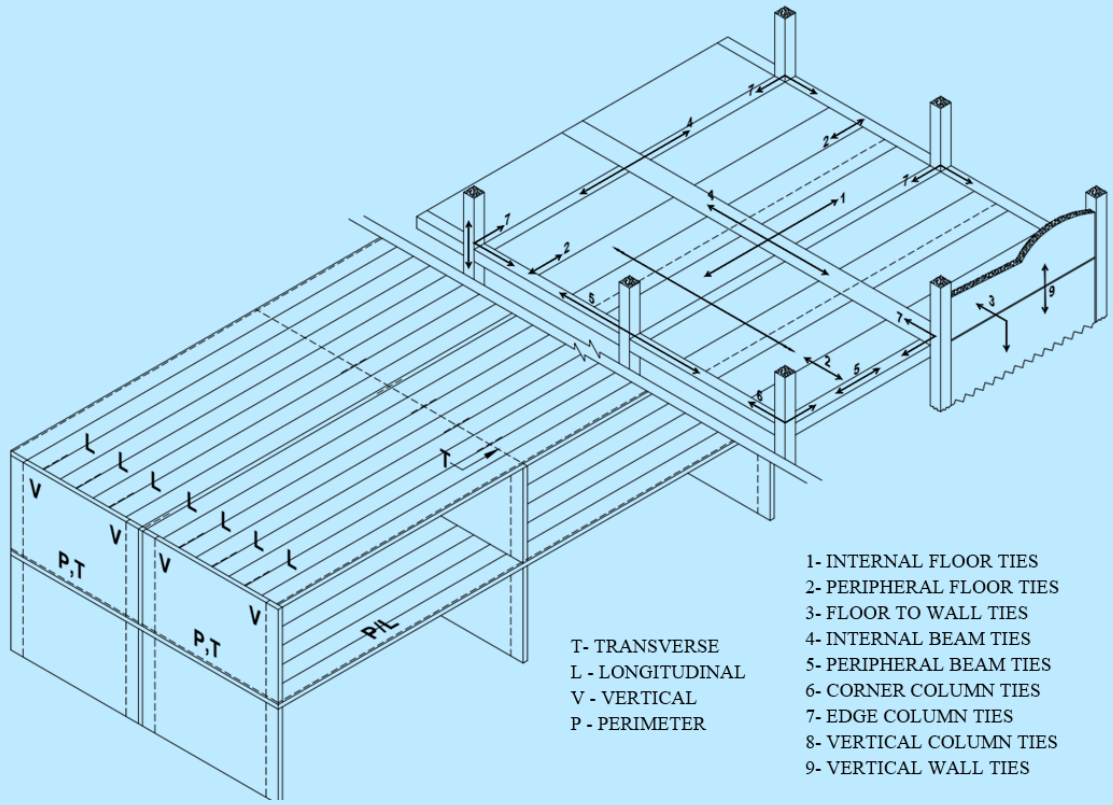

FIG. 32 TYPICAL ARRANGEMENT OF INTEGRITY TIES

8.5.1 *Peripheral Ties*

At each floor and roof level an effectively continuous tie shall be provided within 1.2 m of the edge of the building or within the perimeter wall (*see* Fig. 33). The tie shall be capable to resist a tensile force of F_t equal to 60 kN or (20 + 4N) kN, whichever is less, where N is the number of storeys (including basement).

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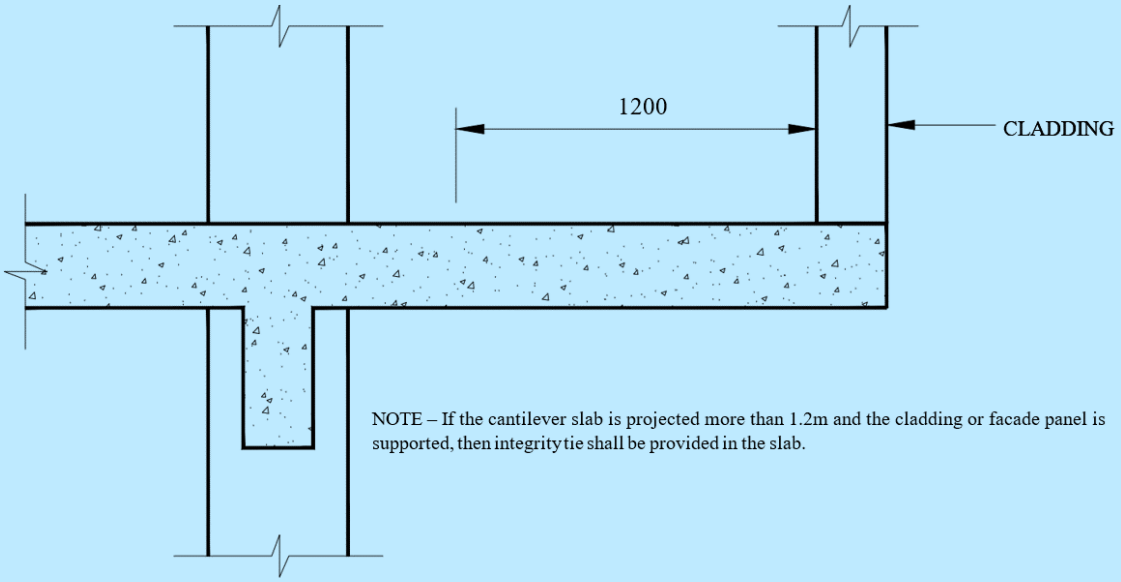
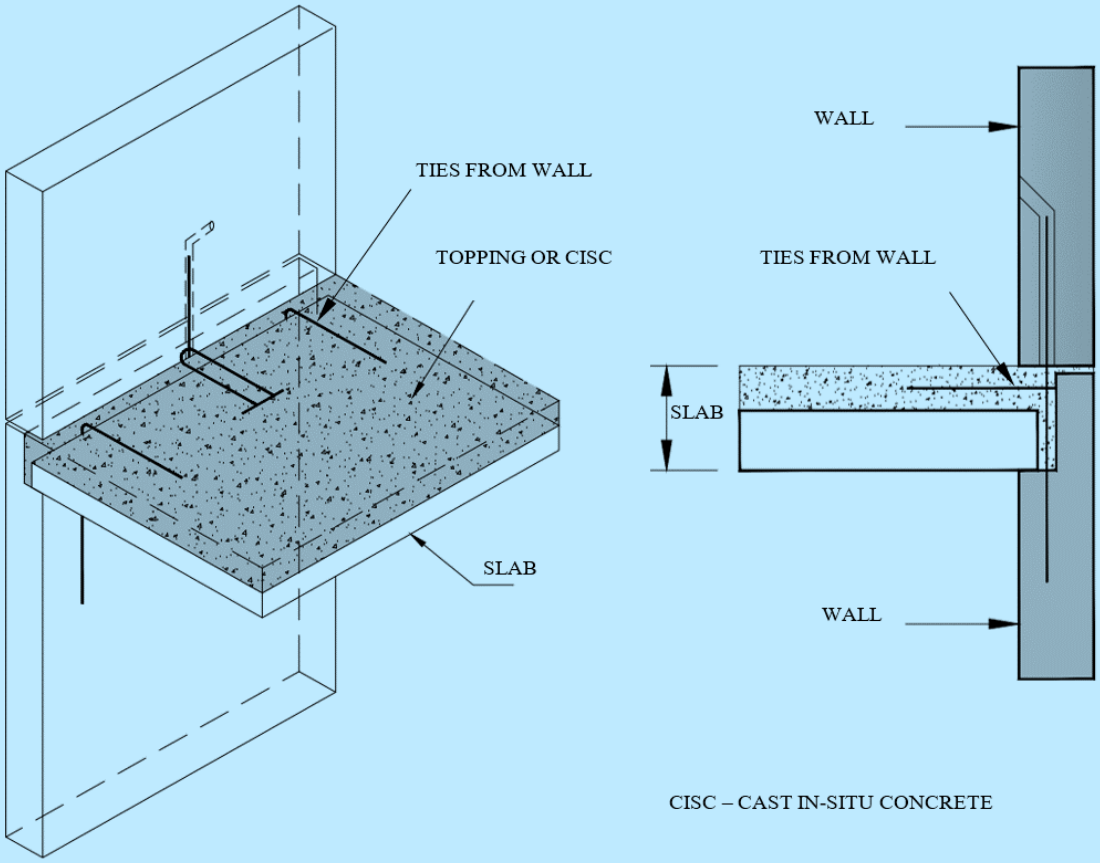


33A) POSITION OF PERIPHERAL TIE



33B) PRECAST WALL TO WALL CONNECTION WITH SHEATHING PIPE

FIG. 33 POSITION FOR PERIPHERAL TIE (CONTINUED)

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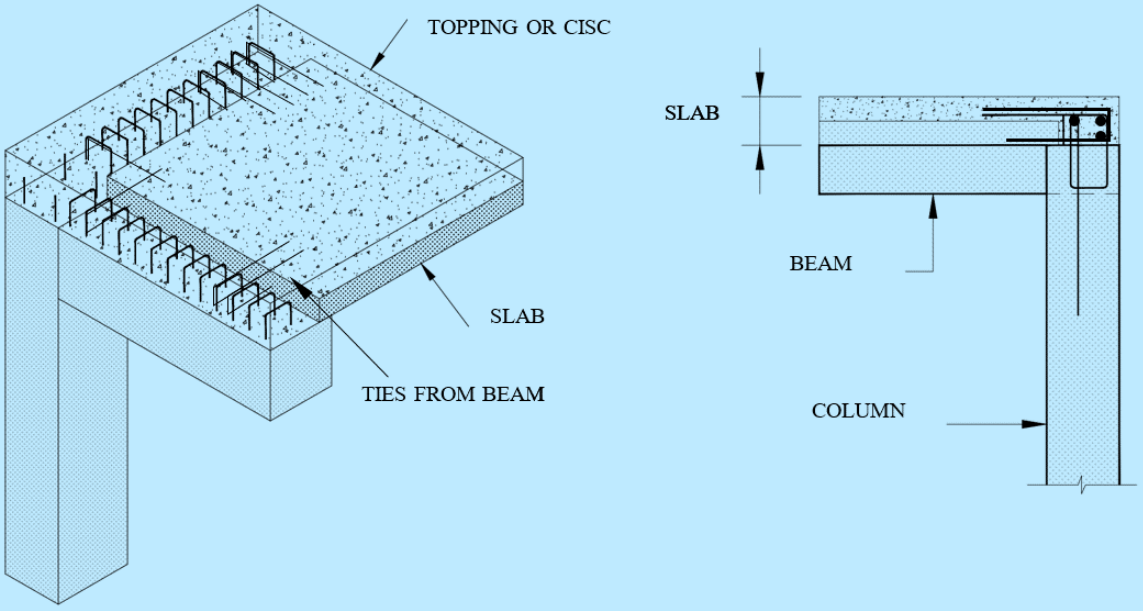
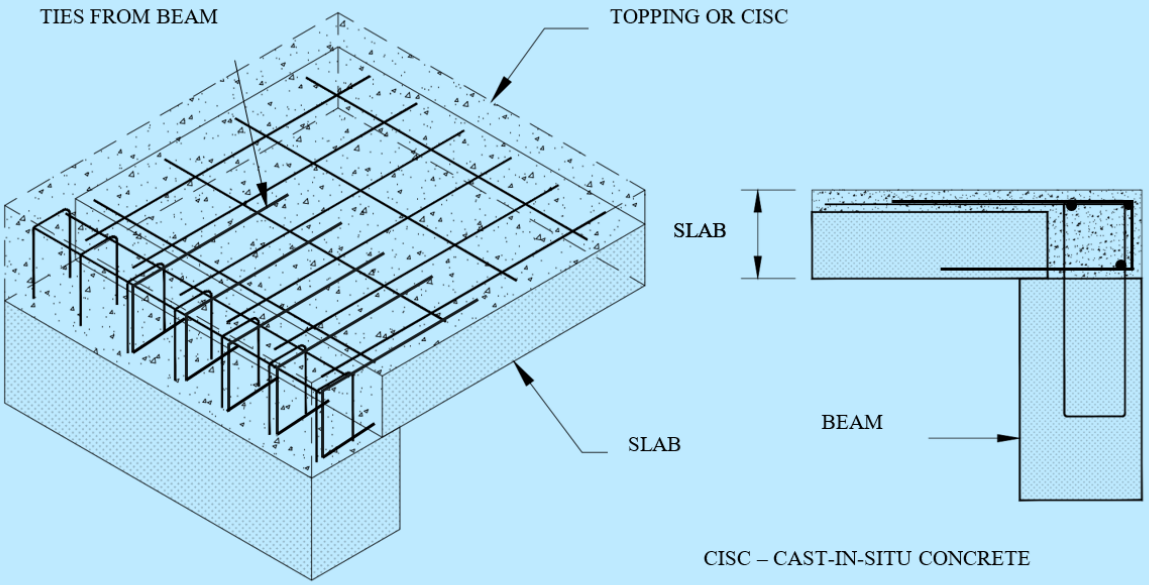


33C) PRECAST COLUMN, BEAM AND SLAB CONNECTION



33D) PRECAST BEAM SLAB CONNECTION

FIG. 33 POSITION FOR PERIPHERAL TIE (CONTINUED)

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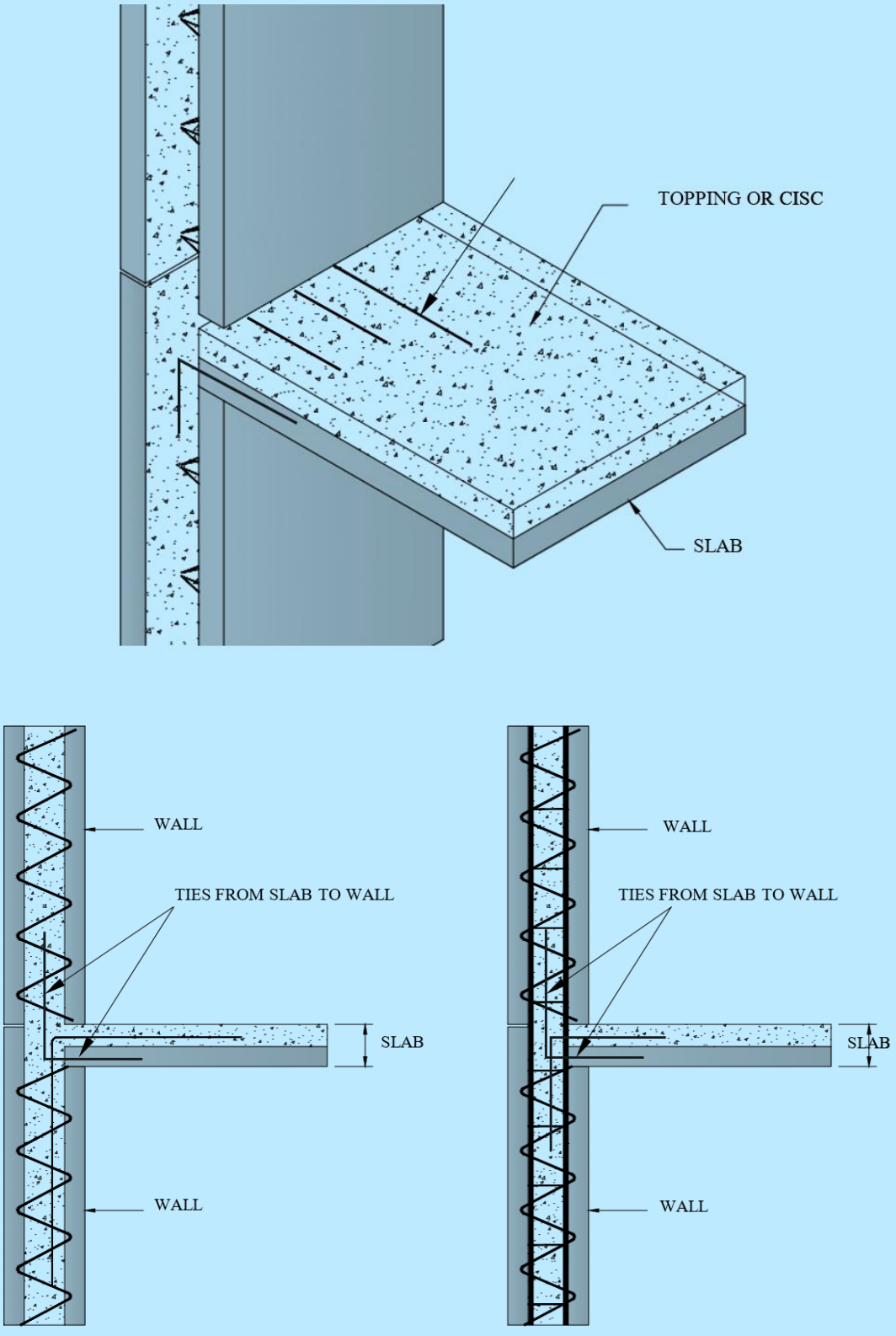

FIG. 33 POSITION FOR PERIPHERAL TIE

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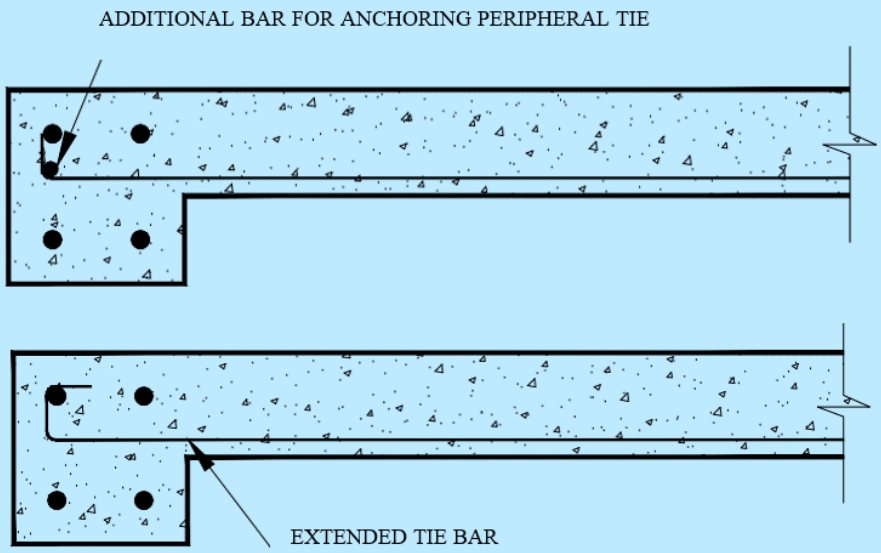
8.5.2 *Internal Ties*

These shall be provided at each floor and roof level in two directions approximately at right angles. Ties shall be effectively continuous throughout their length and be anchored to the peripheral tie at both ends, unless continuing as horizontal ties to columns or walls (*see* Fig. 34). The tensile strength, in kN/m width shall be the greater of:

$$\left(\frac{g_k + q_k}{7.5}\right)\frac{L_r F_t}{5} \text{ and } F_t$$

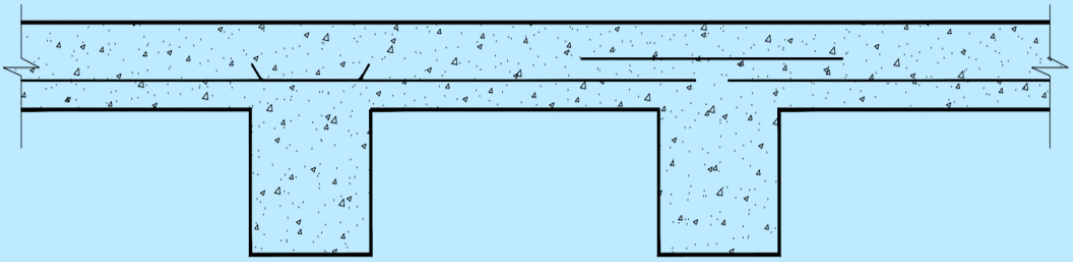
where, $(g_k + q_k)$ is the sum of average characteristic dead and imposed floor loads, in kN/m^2 and L_r is the greater of the distance between the centre of columns, frames or walls supporting any two adjacent floor spans in the direction of the tie under consideration.



NOTE – If the peripheral tie consists of bars in an edge beam, then the bottom bars in the slabs will not be at the same level as the peripheral tie bars. It is suggested that either an additional bar be used for anchoring in the peripheral tie or the internal tie bars be extended and anchored around the top bar in the beam.

FIG. 34 ANCHORING OF TIES IN SLABS

The bars providing these ties shall be distributed evenly in the slabs (*see* Fig. 35) or shall be grouped at or in the beams, walls or other appropriate positions but at spacing generally not greater than 1.5 L_r.



NOTE – For continuity in continuous slabs, bars are distributed evenly in a floor slab by means of lapping some bottom slab reinforcement at supports, either by extending existing bars or by an addition of splice bar.

FIG. 35 CONTINUITY REQUIREMENT FOR SLAB

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8.5.3 *Horizontal Ties to Column and Wall*

All external load-bearing members such as columns and walls shall be anchored or tied horizontally into the structure at each floor and roof level. The design force for the tie shall be greater of:

a) $2F_t$ kN or $Ls \times (F_t/2.5)$ kN, whichever is less for a column or for each metre length, if there is a wall (Ls is the floor to ceiling height in metre); and

b) 3 percent of the total ultimate vertical load in the column or wall at that level.

For corner columns, this tie force should be provided in each of two directions approximately at right angles.

8.5.4 *Vertical Ties*

Each column and each wall carrying vertical load should be tied continuously from the foundation to the roof level.

8.5.4.1 The reinforcement provided is required to resist a nominal tensile force of at least 50 kN per horizontal metre of wall.

8.5.4.2 At least two integrity ties shall be provided in each wall panel.

8.5.4.3 Connection between precast column shall have vertical integrity ties, with a nominal tensile strength of $1.4A_g$, where A_g is the gross area of the column. For column with larger cross section reduced effective area equal to the actual cross section required shall be used. However, the reduced area shall be at least one-half of the gross area of column.

8.5.4.4 In situation where provision of vertical ties cannot be done, the element should be removed, and the surrounding members shall be designed to bridge the gap.

8.5.4.5 All members in the lateral force resisting system shall have one tie continuous from foundation to terrace at each end of the member with Class H coupler as per [C-7(9)] (ties as per design) or direct lapping (ties as per lapping requirements).

8.5.5 *Tie Anchorage*

A tie crossing another tie at right angles may be considered anchored if the bars extend 12 times the bar diameter or an equivalent anchorage beyond the other tie; or an effective anchorage length, calculated from the tension force, beyond the centreline of the other tie. At abrupt changes in construction or at re-entrant corners, it shall be ensured that the ties are sufficiently anchored or to be made effective by other means.

8.5.6 *Continuity of Ties*

A continuous tie shall satisfy **8.3.1**. The minimum thickness of in-situ concrete section where tie bars are provided shall be at least the total dimension of the bar diameter (or two diameters at laps) and twice the maximum aggregate size plus 10 mm.

To provide continuity, the tie shall also satisfy one of the following:

a) A bar in a precast concrete unit lapped with a bar encased in in-situ concrete bounded on two opposite sides by rough faces of the same precast unit (*see* Fig. 36);

b) A bar in a precast concrete unit lapped with a bar encased in the topping of in-situ concrete tied to the precast unit by links or stirrups. The tensile resistance of such links and stirrups should not be less than the designed tension in the tie (*see* Fig. 37); and

c) Bars lapped within in-situ topping concrete with projecting links or stirrups from the supporting precast units, such as beams or slabs (*see* Fig. 38).

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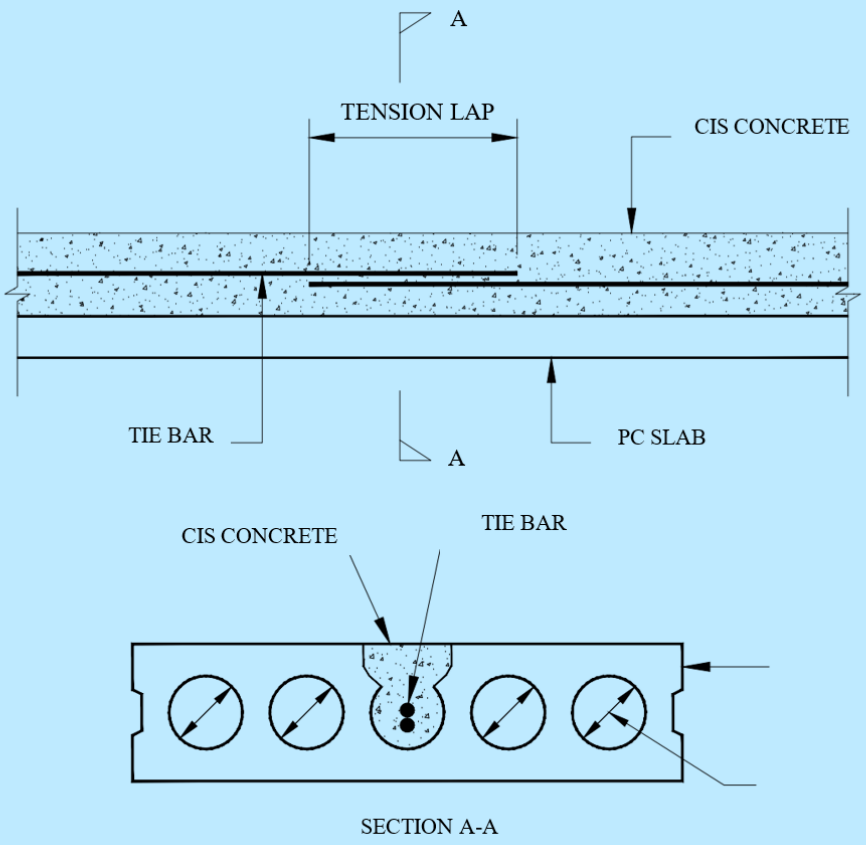

FIG. 36 CONTINUITY OF TIES: BARS IN PRECAST MEMBER LAPPED WITH BAR IN CIS

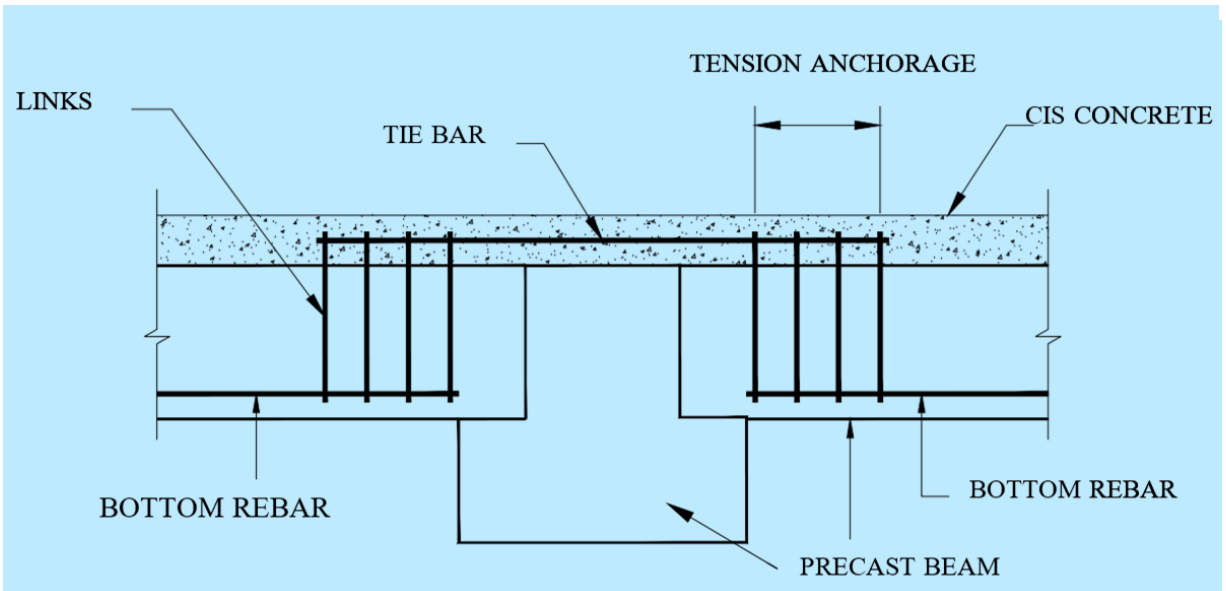

FIG. 37 CONTINUITY OF TIES: ANCHORAGE BY ENCLOSING LINKS

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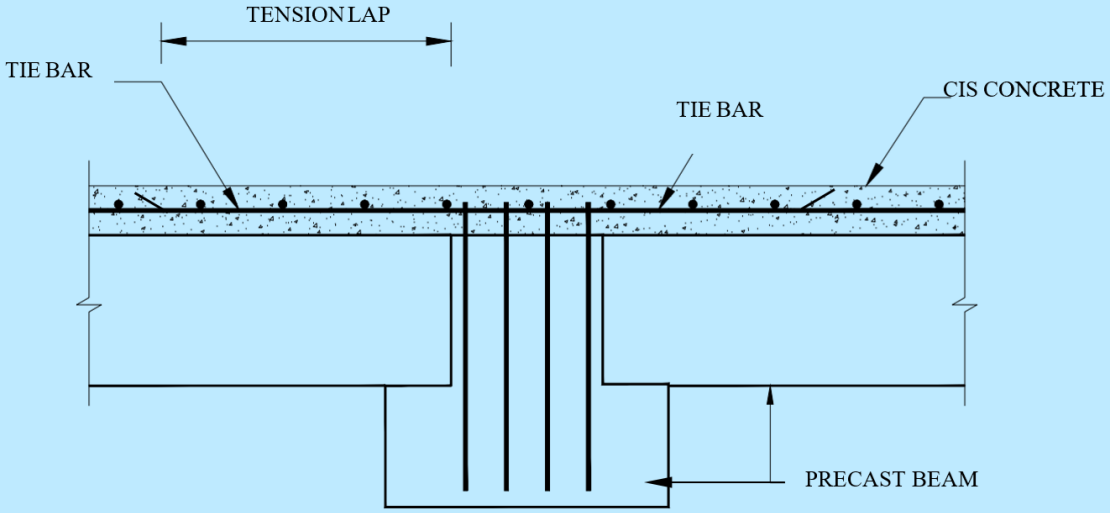

FIG. 38 CONTINUITY OF TIES: BARS LAPPED WITHIN IN-SITU CONCRETE

8.5.7 *Ties between Precast Element and Topping*

In potential plastic hinge regions, the consequences of delamination of the topping from the precast members shall be assessed. If the maximum shear stress at any section exceeds 0.4 MPa, connectors shall be provided between the topping and precast to achieve composite action as per the following guidelines:

a) Ties with an effective area of 40 mm^2 per m^2 of floor area, or equivalent connectors, shall connect the topping to the precast element;

b) Spacing of connectors shall not exceed 1 500 mm and the tributary area of topping reliant on each connector shall not exceed 2.25 m^2;

c) Connectors shall engage horizontal reinforcement, or shall be otherwise effectively anchored into both the topping and the precast element, or into the joints between precast elements; and

d) In HCS appropriate connection shall be placed in slab joints.

8.5.8 *Anchorage in Structures*

All precast floor and precast roof units should be effectively anchored if these units are not utilised to provide the ties. The anchorage shall be capable of supporting the dead weight of the precast unit to that part of the structure which contains the ties.

8.5.9 *Eccentricity*

Ties connecting precast floor or roof units shall be so designed and placed as to minimise eccentricity.

8.5.10 *Anchorage at supports*

In case reinforcing bars are used to provide the structural integrity of slab ends supported on corbels or nibs, great care shall be taken to ensure it is adequately lapped and anchored. Allowance should also be made for constructional inaccuracies.

8.5.11 *Progressive Collapse*

The following recommendations shall be followed for ensuring stability of prefabricated structures:

a) Adequate strengthening of external joints of external wall panels is important since these elements are not fully restrained on both sides by floor panels. Adequate design precautions shall be taken.

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Connections for the external wall panels are likely to be the weakest points of a precast panel building, if not designed properly;

b) Restraint shall be provided to all load bearing elements at the corners of the building. These elements and the external ends of cross-wall units shall be stiffened either by introducing columns as connecting units or by jointing them to non-structural wall units, which in emergency shall act in force transfer;

c) In prefabricated construction, the possibility of gas or other explosions which can damage primary structural elements leading to progressive collapse of the structure shall be considered. It is, therefore, necessary to consider the possibility of progressive collapse in which the failure or displacement of one element of a structure causes the failure or displacement of another element and results in the partial or total collapse of the building;

d) Provision in the design to reduce the probability of progressive collapse is essential in buildings of height more than 15 m and is of relatively higher priority than for buildings of lower height. However, buildings of special importance shall be checked for progressive collapse irrespective of height guidelines. One way of preventing progressive collapse of external wall panels is to design the combined precast wall panels of entire width as a beam of depth equal to the floor height for the impact loading of upper floors and provide reinforcement accordingly at the top and bottom of all precast wall panels at each floor. To make reinforcement continuous across joints, top and bottom bars projected into joints between wall panels shall be joined with lapping bars;

e) It is necessary to ensure that any local damage to a structure does not spread to other parts of the structure remote from the point of mishap and that the overall stability is not impaired, but it may not be necessary to stiffen all parts of the structure against local damage or collapse in the immediate vicinity of a mishap, unless the design briefs specially require this to be done. The requirements as specified at 8.5 shall be followed to prevent progressive collapse of the structure;

f) Additional protection may be required in respect of damage from vehicles. Further, it is necessary to consider the effect of damage to or displacement of a load-bearing member by an uncontrolled vehicle. It is strongly recommended that important structural members are adequately protected by concrete kerbs or similar method. In areas with possibility of passenger vehicular impact, the precast components shall be designed for an un-factored load of 27 kN, in any direction, acting at heights between 450 mm and 700 mm on an area not larger than 300 mm × 300 mm to produce maximum load effect. This load is not required to act concurrently with any handrail or guard rail system loads; and

g) In all aspects of erection that affect structural design, it is essential that the designer shall maintain a close liaison with the builder/contractor regarding the erection procedures to be followed.

8.5.12 *Key Element*

For buildings of five or more storeys, the layout should be checked to identify key elements. A key element is such that its failure would cause the collapse of more than a limited area close to it. The limited area defined above may be taken equal to 70 m^2 or 15 percent of the area of the storey, whichever is less. If key element exists, it is preferable to modify the layout so that the key element is avoided or design the element for a force of 36 kN/m^2.

8.6 Miscellaneous Items for Joints

8.6.1 *Sealants for Joints*

In choosing the appropriate type of sealant for the joints, consideration should be made in respect of the movements between the components to be joined, the bond that is achievable between the components and the sealant and the nature of the sealing material itself. Hybrid, PU (polyurethane, polysulphide), silicone based, or any other sealant shall be used depending on requirements.

8.6.2 *Joint Width*

Joint widths should be sized to accommodate construction tolerances and movements without overstressing the jointing material. Irrespective of movement, a joint width of 10 mm is the minimum practicable for sealant application.

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8.6.2.1 Minimum width of joint between the erected elements or supporting elements shall be as follows:

a) 10 mm, if not specified;

b) 20 mm for wall elements;

c) 30 mm for column elements; and

d) For larger elements as per grouting requirements but shall preferably not exceed 50 mm.

8.6.3 *Joint Preparation*

Satisfactory performance is critically dependent on the satisfactory adhesion of the sealant to the joint surfaces. Primers may be recommended by the manufacturer for some materials. Formwork oils, curing compounds, silicone waterproofing admixtures and surface coating materials may reduce bond and require special precautions.

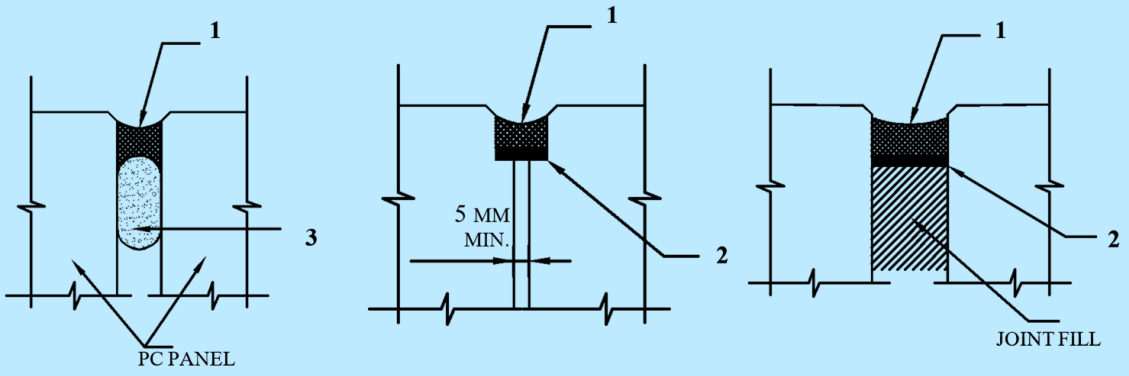
Water jetting, sand blasting, wire brushing or the use of retarder may be required to prepare the concrete joint surfaces in certain instances. Consideration should be given to the use of chamfers to reduce edge damage.

8.6.4 *Back up Material and Bond Breaker*

To ensure good adhesion, a firm backing should be provided for the application of sealants. The required shape and proportions of the sealant are achieved by the correct installation of the back up material.

Sealants in movement joints should not adhere to the backing material to avoid any unnecessary restraint. To ensure that the sealant and back up materials, bond breakers and joints fillers are compatible and appropriate for the proposed end use, advice from the manufacturer should be obtained as necessary.

The sealant backing may be provided by a closed cell foam back up material alone (*see* Fig. 39A), a thin self-adhesive bond breaker tape (*see* Fig. 39B) or a joint filler separated from the sealant by a thin self-adhesive bond breaking tape (*see* Fig. 39C).



39A CLOSED CELL FOAM BACKING 39B) BOND BREAKER TAPE BACKING 39C) JOINT FILLER WITH BOND BREAKER TAPE

1 - SEALANT,
2 - BREAKER TAPE OR BOND BREAKER
3 - BACKER ROD

FIG. 39 BACK-UP MATERIALS AND BOND BREAKERS IN MOVEMENT JOINTS

8.6.4.1 *Minimum joint gap widths*

For elastic sealants, the minimum thickness shall be 5 mm, the ratio of width to thickness should never exceed 1 : 1 and for optimum performance 2 : 1.

8.6.4.2 *Sealant application*

High moisture content may be detrimental to the adhesion of the sealant. Sealing should not be undertaken if there is free water present on the surface of the concrete.

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8.6.4.3 *Performance, choice and failure of sealant*

The magnitude of the movement, and also the mode, frequency and rate of movement affect the performance of a sealed joint.

There are many factors affecting the choice of sealant suitable for different types of movement. However, as a general rule, joints which have to accommodate frequent and rapid movement need an elastic sealant, while joints in massive components, with high thermal inertia resulting in much slower movement, may be satisfactorily sealed with an elastoplastic, plasto-elastic or a plastic sealant.

The different ways in which sealants can fail as a consequence of different factors including climatic conditions, environmental factors, substrate incompatibility, abrasion and traffic loading need to be considered when selecting a sealant.

8.6.5 *Sealing Strips*

Preformed sealing strips of appropriate sizes and sections shall be used as required. There are two basic types:

a) Mastic strips; and

b) Impregnated or coated cellular strips.

8.6.5.1 *Mastic strips*

These should be normally installed during the assembly of components. They require an initial compression to ensure proper adhesion to the components forming the joint. As a degree of compression is also required during in service, mastic strips should not be used for joints which open beyond their assembled size.

8.6.5.2 *Impregnated or coated cellular strips*

Impregnated or coated cellular strips may be supplied in a pre-compressed form, and one face may be adhesive coated. When pre-compressed, they can readily be installed within the joint gaps, but should be of a suitable size to be maintained under the degree of compression specified by the manufacturer throughout the range of joint movements. The degree of compression may be varied according to the level of sealing performance required.

To ensure uniformity of compression and stability of the seal, it is important that the joint faces are parallel. Sufficient joint depth shall be provided to accommodate the seal with these depths to width ratios. For external applications, sealing strips should have adequate exposure resistance for the proposed service conditions.

8.6.6 *Joint Fillers*

Fillers for movement joints should be carefully selected to suit their intended use. To ensure that the sealant and filler are compatible, advice from the manufacturer should be sought. Fillers should be non-absorbent with appropriate elastic properties and fire resistant.

8.6.7 *Overlapping of Precast Facade*

The resistance to water penetration between the upper and lower panel of precast facade is commonly provided by an upstand profile. A minimum overlap of 75 mm between the upper and lower panel is recommended.

9 CONSTRUCTION PLANNING

9.1 Pre-Engineering and Design Coordination

9.1.1 *Importance of Pre-Engineering*

Pre-engineering plays a critical role in the planning and execution of any precast concrete project. To ensure a successful outcome, the planning engineer should begin working from the design stage to ensure proper production, installation, connections, and logistics. Key resources must be identified and prepared for each phase of the project, including:

a) Availability land/approvals/water and electricity for a site-based factory;

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b) Suitable plant and machinery;

c) Steel moulds;

d) Lifting machinery at factory;

e) Trailers;

f) Lifting machinery on site (tower cranes/mobile cranes); and

g) Site access for transportation of large elements as well as route studies and local regulations and hours for operating large trailers.

9.1.2 *Resource Planning and Feasibility Factors*

The resources should be carefully planned based on the types of project elements and their quantities. The engineer is also responsible for reserving these resources throughout the project duration. Key feasibility considerations include:

a) *Building geometry* — Ensure shapes and geometry are as symmetrical and regular as possible;

b) *Grid-lines* — Column and wall locations should, preferably, follow a standardized pattern;

c) *Height clearances* — Consideration of clear height expectations between beams and slabs;

d) *Grid spacing* — Optimize grid spacing to take full advantage of precast elements; and

e) *Standardization* — Standardize elements within one floor and across multiple floors when feasible.

9.1.3 *Pre-Engineering Design Stage Process*

The pre-engineering design stage should include the following steps:

a) Study of building functionality;

b) Load assumptions and calculations;

c) Preparation of the design basis report (DBR);

d) Structural analysis;

e) Preliminary framing plan selection for precast elements;

f) Approval of preliminary framing plans by the architect/client;

g) Review meetings with architects and consultants;

h) Finalization of the structural framing plan after incorporating feedback and value engineering;

j) Release of the good for construction (GFC) and structural framing plan (R0), including:

   1) Geometry of precast elements;

   2) General arrangement (GA) drawings with reinforcement details; and

   3) Connections and inserts.

k) Coordination with mechanical, electrical, plumbing and other service consultants should occur during the early stages.

9.2 Project Planning, Scheduling and Monitoring

9.2.1 *Production Planning*

Careful planning is essential for ensuring that precast concrete systems are produced efficiently. Key factors to address include:

a) Synchronization of manufacturing schedules and resources to meet construction and storage needs;

b) Anticipation of any special transportation or access challenges;

c) Use of detailed production shop drawings to facilitate smooth execution; and

d) Factory curing periods for high strength concrete elements.

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9.2.1.1 *Shop drawings*

Shop drawings should contain:

a) Member shapes (elevations and sections) and dimensions;

b) Details of reinforcement, anchors, inserts, etc;

c) Connection details;

d) Production tolerances;

e) Handling and lifting devices;

f) Finishing and bearing seat details;

g) Transportation and storage methods; and

h) Unique identification of elements (ID, location and orientation).

9.2.1.2 *Factory vs on-site production*

A decision whether precast elements should be produced in an off-site factory or on-site, based on factors such as:

a) Project scale and volume;

b) Availability of factory space near the site;

c) Contractual agreements regarding space provision;

d) Element type, weight, and size; and

e) Cost-benefit analysis.

9.2.2 *Production Schedule*

9.2.2.1 *Cycle time and throughout rate and the key aspects*

a) *Cycle time* — refers to the total time to complete one production cycle of an element, from mould preparation to quality checks and delivery; and

b) *Throughput rate* — refers to the rate of production (units per day, week, or month).

9.2.2.2 *Developing the production schedule*

A production schedule developed based on the following:

a) Splitting the floor plan into various ‘installation parts’, typically aligned with crane locations and capacities;

b) Estimation the number of elements per part and the installation duration for each part based on element type and connection complexity;

c) Using historical data and experience to estimate installation cycle times for each element;

d) Assigning unique element IDs to each installation part and sequence them accordingly;

e) Publishing a sequence chart that includes:

   1) Planned drawing release date;

   2) Planned casting date;

   3) Planned transportation date; and

   4) Planned installation date.

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9.3 Project and Factory Logistics Planning

9.3.1 *Haulage Planning*

Transportation constraints influence the size and shape of precast elements. The following factors should be considered in haulage planning:

a) Trailer lengths (standard lengths of 6 m, 12 m, with special cases up to 30 m);

b) Trailer width (typically between 2.4 m to 2.7 m);

c) Height clearance (3.6 m to 4 m, adjustable with low-bed trailers);

d) Weight capacity (25 MT to 35 MT);

e) Special permits for oversized loads that may be needed;

f) Route surveys and permissions to be obtained if crossing bridges or navigating urban areas; and

g) Large element transportation time slots through day/night as per local regulations.

9.3.2 *Factory Stockyard and Logistics Coordination*

Precast elements in the factory stockyard should be stored using first in, first out (FIFO) or last in, first out (LIFO) methods, depending on site requirements. Proper stock registers and inventory management software should be used for efficient tracking and organization. Storage size of the yards should be defined by the minimum concrete strength and curing periods.

9.3.3 *Crane Planning*

A comprehensive crane layout should be developed in collaboration with the installation planning engineers considering the following:

a) Types of cranes (tower cranes, mobile cranes);

b) Site constraints (space, access);

c) Ground strata for crane footing;

d) Lifting capacities and feeding locations;

e) Crane removal and repositioning areas; and

f) Double handling constraints due to storage at site or crane reach.

9.4 Installation Sequence

Once the casting sequence is set, installation dates to each element should be assigned based on factors such as:

a) Crane type and operational speed;

b) Reduced crane efficiency where multiple cranes are deployed with overlapping zones;

c) Element type and installation cycle time;

d) Connection complexity;

e) Grouting and curing durations;

f) The number of shifts and crane operation hours; and

g) Adjustment of installation dates and resources as needed based on available machinery and logistics.

9.5 On Site Work Planning

9.5.1 *Surveying*

The first activity on-site is surveying and line-out. It should be ensured that key points are marked, including:

a) Building corners and faces;

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b) Compound wall boundaries;

c) Grid lines and column centres;

d) Footing corners;

e) Shear wall grid lines; and

f) Site constraints such as high-tension lines or existing canals or water bodies or filled up areas or marsh lands.

9.5.1.1 Drone surveys are recommended for more accurate and automated site measurements, which also help in tracking progress.

9.5.2 *Cast-In Situ Works*

Cast in-situ (CIS) work is crucial in precast projects. The interface between precast and CIS activities typically occurs during installation and connection phases. The interfaces of the joints shall be roughened to ensure proper bonding with the CIS concrete or grout. Minimizing inter-trade dependencies improves project success in terms of both time and cost. This interface also needs careful attention, as the dowels projecting from precast elements may have higher variation that affects precast installation. Detailed as-built surveys must be completed well in advance to avoid last minute repairs and long delays.

9.6 Digital Tracking and Monitoring Tools

To enhance precision and efficiency, building information modelling (BIM) may be implemented for both production and assembly of precast elements. This technology supports:

a) Access to real-time drawing by all engineers, reducing errors;

b) Seamless use across production, installation, and progress monitoring stages; and

c) Required infrastructure such as Wi-Fi network and system access between various agencies are be in place.

10 CONSTRUCTION, MANUFACTURE, STORAGE, TRANSPORT AND ERECTION OF PRECAST ELEMENTS

10.1 Manufacture of Precast Concrete Elements

10.1.1 A precast facility shall be located based on proximity to raw materials, logistics infrastructure, storage capacity, and crane access to ensure economic production and streamlined erection.

10.1.2 *Manufacture*

Precast elements may be manufactured at a permanent factory or temporary site facility, selected based on project scale, logistics, and environmental goals. For reduction in site pollution and wastage to achieve sustainability in precast construction a reference may be made to G-4.

10.1.2.1 *Factory prefabrication*

Factory prefabrication involves the commercial production of standardized components in a factory setting, typically for long-term manufacturing. It is a capital-intensive process where work is carried out year-round, ideally under a covered shed to minimize the impact of seasonal variations. This type of facility allows for high levels of mechanization, with production organized in a factory-like manner, supported by a consistent team of workers.

10.1.2.2 *Site prefabrication*

Site prefabrication refers to the production of precast components at or near the construction site and shall be adopted for large-scale or long-duration projects where it offers measurable advantages in transportation cost and

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productivity. Although temporary, the site factory should be comparable to a permanent facility with strict quality control. There are the following two types of site prefabrication:

a) *Semi-mechanized* — Carried out in open areas using primarily local labour, with minimal equipment; moulds may be stationary or mobile, and the level of skill shall be appropriate to the component complexity; and

b) *Fully-mechanized* — Conducted within covered sheds using skilled labour and factory-grade machinery, designed for high-volume and high-quality precast production.

10.1.2.3 *Precast factory quality audits*

All precast production facilities shall undergo quality audits at least annually. Audits shall be conducted by a competent team jointly approved by the contractor and client. Audit scope shall include production quality, safety and documentation.

10.1.3 The manufacturing of precast elements shall involve three categories of processes: core, auxiliary and subsidiary. Each category shall be planned and coordinated to ensure continuous, high-quality production.

10.1.3.1 *Core process*

The main processes that include the following steps:

a) Providing and assembling moulds, positioning the reinforcement cage for reinforced concrete work and stressing the wires for prestressed elements;

b) Installing concealed service conduits/pipes;

c) Fixing inserts and tubes as necessary (for handling purposes);

d) Pouring concrete into the moulds;

e) Vibrating and finishing the concrete;

f) Curing the concrete; and

g) Demoulding the forms and stacking the precast products.

10.1.3.2 *Auxiliary process*

These processes are necessary for the successful completion of the core processes and include the following:

a) Mixing and producing fresh concrete (carried out in a mixing station or batching plant);

b) Prefabricating the reinforcement cage (performed in a steel yard or workshop);

c) Manufacturing inserts and other finishing components to be incorporated into the main precast products; and

d) Finishing the precast products.

10.1.3.3 *Subsidiary process*

Subsidiary processes shall include material handling, maintenance and utility services essential for uninterrupted production. These shall include:

a) Proper storage of raw materials;

b) Internal transport systems for fresh concrete and cages;

c) Safe movement and stacking of precast elements;

d) Periodic servicing of equipment and moulds; and

e) Maintenance of curing and utility facilities (for example, steam).

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10.1.4 The manufacturing of precast elements requires systematic planning of all the processes to achieve the following objectives:

a) A cyclic working method to enhance speed and cost-efficiency in production;

b) Mechanization of processes to boost productivity and improve quality;

c) Optimal production levels that meet quality control standards while maintaining the desired construction speed;

d) Improved working conditions for the workforce; and

e) Minimization of weather-related disruptions to the manufacturing schedule.

10.1.5 The various stages of precasting, as outlined in Table 12, are classified based on the equipment required for each stage. This classification facilitates the mechanization and rationalization of work throughout the process. Stages 6 and 7, as shown in Table 12, represent the core processes in the manufacturing of precast concrete elements. For these stages, various technological processes are employed, tailored to the specific concrete product being produced. These methods have been proven to be rational, economical and time-saving. Figure 40 provides a diagrammatic representation of the different stages involved in the plant process.

10.1.6 The accepted methods of manufacturing precast units can be broadly classified into two categories:

a) The stationary method, where the moulds remain in a fixed position and the various processes are carried out in a cyclic order at the same location; and

b) The circulation method, where the precast unit moves through different stages of production, with the processes being carried out in an assembly-line fashion.

The various accepted pre-casting methods are listed in Table 13 along with the details regarding the elements that can be manufactured by these methods.

Table 12 Stages of Precasting of Concrete Products

(*Clause* 10.1.5)

Sl No.	Precasting Stage No.	Name of Process	Operations Involved
(1)	(2)	(3)	(4)
i)	1	Procurement and storage of construction materials	Unloading and transport of cement, coarse and fine aggregates and steel, and storing them in bins, silos or storage sheds
ii)	2	Testing of materials	Testing of all materials including steel
iii)	3	Design of concrete mix	Testing of raw materials, plotting of grading curves and trial of mixes in laboratory
iv)	4	Making of reinforcement cages	Unloading reinforcement bars from wagons or lorries and stacking them in the steel yard, cutting, bending, tying or welding the reinforcements and making in the form of a cage, which can be directly introduced into the mould
v)	5	Applying form release agent and laying of moulds in position	Moulds are cleaned, applied with form release agent, and assembled and placed at the right place
vi)	6	Placing of reinforcement cages, inserts and fixtures	The reinforcement cages are placed in the moulds with spacers, etc, as per data sheet prepared for the specific prefabricate units. Pre-tensioning of wires in the case of pre-stressing of concrete element

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Table 12 (*Concluded*)

Sl No.	Precasting Stage No.	Name of Process	Operations Involved
(1)	(2)	(3)	(4)
vii)	7	Preparation of green concrete	Taking out aggregates and cement from bins, silos, etc, batching and mixing
viii)	8	Transport of green concrete	Transport of green concrete from the mixer to the moulds. In the case of precast method involving direct transfer of concrete from mixer to the mould or a concrete hopper attached to the mould this prefabrication stage is not necessary
ix)	9	Pouring and consolidation of concrete	Concrete is poured and vibrated to a good finish
x)	10	Curing of concrete and demoulding	Either natural curing with water or an accelerated curing using steam and other techniques. In the case of steam curing using trenches or autoclaves, this stage involves transport of moulds with the green concrete into the trench or autoclave and taking them out after the curing and demoulding elements. Cutting of protruding wires also falls in this stage. In certain cases, the moulds have to be partly removed and inserts have to be removed after the initial set. Final demoulding is done after concrete has gained designed strength, enough to resist handling stress, and the components are then allowed to be cured further. The minimum curing duration is as per Part C/Section 5 of this NBCS except in cases where the design strength of the precast element is achieved earlier. In that case, curing only up to achieving the design strength is sufficient, subject to appropriate approvals
xi)	11	Stacking of precast elements	Lifting of precast elements from the mould and transporting to the stacking yard for further transport by trailer or rail is part of this stage
xii)	12	Testing of finished components	Tests are carried out on the components individually and in combination to ensure the adequacy of their strength
xiii)	13	Miscellaneous	a) Generation of steam involving storing of coal or oil necessary for generation of steam and providing insulated steam pipe connection up to the various lines b) Repair of machines used in the production

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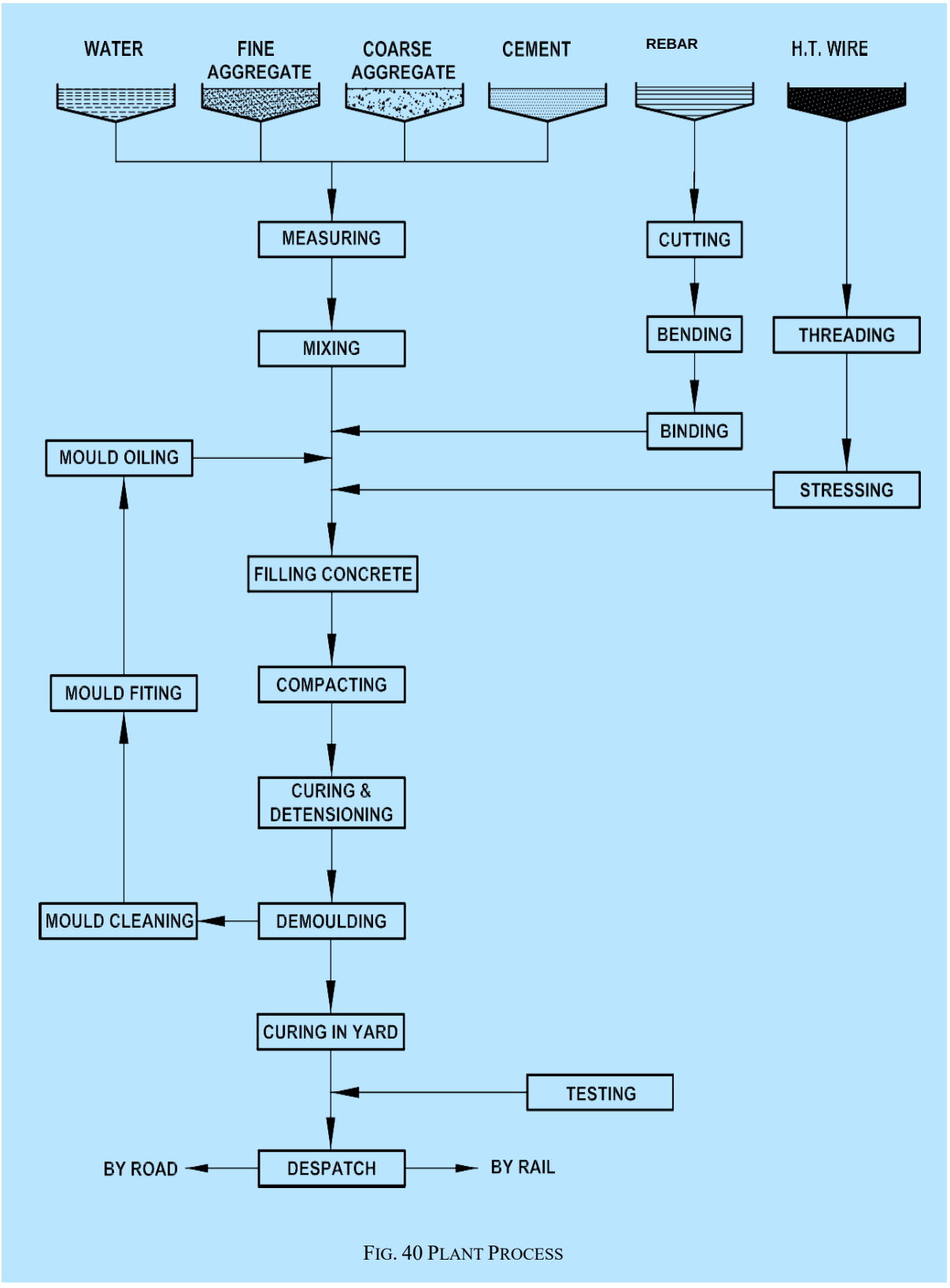

FIG. 40 PLANT PROCESS

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Table 13 Precasting Methods

(*Clauses* 10.1.6 *and* 10.6.1)

Sl No.	Precasting Method	Where Used	Recommended Dimensions and Weights	Advantages and Remarks
(1)	(2)	(3)	(4)	(5)
i)	*Individual mould method* — Precasting method using mould which may be easily assembled out of bottom and sides, transportable, if necessary. This may be either in timber or in steel using needle or mould vibrators and capable of taking pre-stressing forces	Ribbed slabs, beams, girders, window panels, box type units and special elements, prestressed railway sleepers, parts of pre-stressed girders, etc	No limit in size and weight. It depends on the equipment used for demoulding, transporting and placing	Strengthening of the cross-section possible Openings are possible in two planes
ii)	*Battery mould method* — The shuttering panels may be adjusted into the form of a battery at the required distances equal to the thickness of the concrete member	Interior wall panels, shell elements, reinforced concrete battens, rafters, purlins, roof and floor slabs	Length ≤ 9 m Height ≤ 3 m Mass ≤ 5 t	Especially suitable for mass production of wall panels where shuttering cost largely reduced. Vapour or steam curing may be adopted by taking the steam pipes through the shuttering panels
iii)	*Stack method*	Floor and roof slab panels	Length: Any desired length Breadth: 1 to 4 m Mass ≤ 5 t	For casting identical reinforced or pre-stressed panels one over the other with separating media interposed in between
iv)	*Tilting mould method* — This method is capable of being skipped vertically using hydraulic jacks	Exterior wall panels where special finishes are required on one face or for sandwich panel	Length ≤ 6 m Breadth ≤ 4 m Mass ≤ 5 t	Suitable for manufacturing the external wall panels
v)	*Long line prestressing bed method*	Double tees, ribbed slabs, purlins, piles and beams	Length ≤ Any desired length Breadth ≤ 2 m Height ≤ 2 m Mass : Up to 10 t	Ideally suited for pretension members
vi)	*Extrusion method* — Long concrete mould with constant cross-section where concreting and vibration are done automatically just as in hollow cored slab casting	Roof slabs, foam concrete wall panels and beams cross-section	Length : Any desired length Breadth : < 2 m Height : < 3 m	May be used with advantage in the case of un-reinforced blocks, foam concrete panels

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10.1.7 *Preparation and Storage of Materials*

Proper storage of materials is crucial in the precast industry, as errors in planning this aspect can significantly impact production economics. In a precast factory, where quality is paramount, the proper storage and preservation of building materials especially cement, coarse aggregates and fine aggregates are essential. Materials should be stored in accordance with accepted standard [C-7(21)]. Additionally, precast elements should follow a 'first-in, first-out' (FIFO) system to minimize or avoid issues related to shrinkage, creep and other material-related concerns.

10.2 Moulds

10.2.1 Moulds used for the manufacture of precast elements may be made from steel, timber, concrete, plastic, or a combination of these materials. When designing moulds for various elements, special attention should be given to facilitating easy demoulding and the assembly of parts. Additionally, the rigidity, strength, and watertightness of the mould are critical, especially considering the forces involved during the pouring of green concrete and the vibrating process. The deflection of any supporting member of the mould shall not exceed $L/350$ or 3 mm, whichever is less, where L represents the span of the member in millimetres).

10.2.2 Tolerances

The moulds shall be designed to accommodate the tolerances as per the casting tolerances given in Table 14.

Table 14 Casting Tolerances for Precast Components

(*Clause* 10.2.2)

Sl No.	Product (*See* Key No.)	Product Tolerances
(1)	(2)	(3)
i)	*Length*:	
	1, 10, 12, 15, 16, 17	± 5 mm
	9, 11	± 10 mm
	2, 3, 4, 6, 7, 8	± 10 mm
	5	± 2 mm for length below and up to 500 mm ± 5 mm for length over 500 mm
	13, 14, 18, 19	± 15 mm
ii)	*Thickness/cross sectional dimensions*:	
	1	± 3 mm
	2, 8	± 3 mm or 0.1 percent, whichever is greater
	4, 5, 7, 10, 12, 15	± 3 mm, for thickness up to 300 mm ± 5 mm, for thickness greater than 300 mm
	3, 6, 9, 11, 16, 17, 18, 19	± 5 mm
	13, 14	+ 5 mm, - 3 mm
iii)	*Straightness/bow*:	
	2, 4, 5, 7, 8, 10, 15, 16, 18	1/360 of length, maximum of ± 10 mm
	1, 5	± 3 mm
iv)	*Squareness* — When considering the squareness of the corner, the longer of two adjacent sides being checked shall be taken as the base line:	
	a) *Skew across width of an end:*	
	15	± 3 mm per 300 mm length, with maximum of ± 5 mm
	7, 9	± 3 mm per 300 mm length, with maximum of ± 10 mm
	1, 2, 4, 5, 8, 10, 11	± 3 mm per 300 mm length, with maximum of ± 15 mm

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Table 14 (*Concluded*)

Sl No.	Product (*See* Key No.)	Product Tolerances
(1)	(2)	(3)
	b) *Skew across thickness or depth or height of an end*:	
	12, 16	± 3 mm
	4, 7, 8, 16	± 3 mm per 300 mm length
	13, 14	Depth ≤ 600 mm, ± 5 mm Depth ≥ 600 mm ± 1.5 mm for 300 mm length with maximum of ± 15 mm
v)	*Twist* — Any corner shall not be more than the tolerance given below from the plane containing the other three corners:	
	2, 8	5 mm, Up to 600 mm in width and up to 6 m in length 10 mm, Over 600 mm in width and for any length
	4	± 1/1 500 of dimension or ± 5 mm, whichever is less
	1	± 3 mm
	7	± 1 mm
vi)	*Flatness* — The maximum deviation from 1.5 m straight edge placed in any position on a nominal plane surface shall not exceed:	
	2, 8	± 5 mm
	4	± 3 mm
	1	± 2 mm
	5	± 5 mm or maximum of 0.1 percent length
vii)	*Warping* — Deviation from plane in which the corners of the panel do not fall within the same plane:	
	2, 4, 5, 7, 8, 16, 18	± 1.5 mm per 300 mm length
	10, 15	± 5 mm

Key No.

1	Channel unit	7	Precast planks	14	Double tee
2	Ribbed slab unit/hollow slab	8	Ribbed/plain wall panel	15	Stadium riser
3	Waffle unit	9	Column	16	Cladding panel/facade
4	Large panel prefabricatio	10	Step unit	17	H-Frame
5	Cellular floor/roof slabs	11	Beam and spandrel	18	Foundation units
6	Prefabricated brick panel	12	Modular unit	19	Drain
		13	Single tee		

10.2.3 *Mould Wall Slopes*

To facilitate the easy removal of precast elements from the mould, particularly when the mould has fixed sides and the elements need to be lifted out, the side slopes should ideally be 1 in 6 but shall not be less than 1 in 12. For slopes less than 1 in 6, the mould should have provisions of appropriate hydraulic jacks at the base of the mould to assist demoulding. A gentle slope helps prevent the elements from becoming stuck in the mould during demoulding.

10.3 Accelerated Curing

In precasting factories, using accelerated or artificial curing methods is economical, as it allows elements to be demoulded much earlier, enabling the early re-use of moulds. Any of the following methods shall be adopted:

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10.3.1 *Steam Curing*

10.3.1.1 The steam curing of concrete products shall take place under tarpaulin in tents, under hoods, under chambers, in tunnels or in special autoclaves. The steam shall have a uniform flow throughout the length of the member. The precast elements shall be so stacked with sufficient clearance between each other and the bounding enclosure, to allow proper circulation of steam.

10.3.1.2 Before the concrete products are subjected to any accelerated method of curing, the cement to be used shall be tested for soundness, setting time and suitability for steam curing.

10.3.1.3 In the case of elements manufactured by accelerated curing methods, concrete admixtures to reduce the water content may be allowed to be used. The normal aeration agents used to increase the workability of concrete should not be allowed to be used. Use of calcium chloride-based admixtures should not be used for reinforced concrete elements so as to meet the chloride limits prescribed in Part C 'Structural design, Section 5 Structural Concrete' of this NBCS.

10.3.1.4 The inside face of the steam curing chamber, tunnel or hood shall have a damp-proof layer to maintain the humidity of steam. Moreover, a proper slope shall be given to the floor and the roof to allow the condensed water to be easily drained away. When steam is first let into the curing chambers, the air inside shall be allowed to go out through openings provided in the hoods or side walls which shall be closed soon after moist steam is seen jetting out.

10.3.1.5 It is preferable to let in steam from the top of the chamber through perforated pipelines to allow uniform entry of steam throughout the chamber.

10.3.1.6 The fresh concrete in the moulds should be allowed to get the initial set before allowing the concrete to come into contact with steam. The regular heating up of fresh concrete products from about 20 °C to 35 °C should start only after a waiting period ranging from 2 h to 5 h depending on the setting time of cement used. It may be further noted that steam can be let in earlier than this waiting period, provided the temperature of the concrete product does not rise beyond 35 °C within this waiting period.

10.3.1.7 The second stage in steam curing process is to heat up the concrete elements, moulds and the surroundings in the chamber in the following manners:

a) In the low-pressure steam curing, the airspace around the member is heated up to a temperature of 75 °C to 80 °C at a gradual rate, usually not faster than 30 °C/h (this process takes around 1 h to 1 h 30 min depending upon outside temperature); and

b) In the case of curing under high pressure steam in autoclaves, the temperature and pressure are gradually built up during a period of about 4 h.

10.3.1.8 The third stage of steam curing is to maintain the uniform temperature and pressure for a duration depending upon thickness of the section. This may vary from 3 h to 5½ h in the case of low-pressure steam curing and 4 h to 7 h in the case of high-pressure steam curing.

10.3.1.9 The fourth stage of steam curing is the gradual cooling down of concrete products and surroundings in the chamber and normalization of the pressure to bring it at par with outside air. The maximum cooling rate, which is dependent on the thickness of the member, should normally not exceed 30 °C/h.

10.3.1.10 In all these cases, the difference between the temperature of the concrete product and the outside temperature should not be more than 60 °C for concrete up to M30 and 75 °C for concretes of M45 or greater. In the case of light weight concrete, the difference in temperature should not be more than 60 °C for concretes less than M25. For concretes greater than M50, the temperature differences can go up to 75 °C.

10.3.2 *Vapour Curing*

Vapour curing shall consist of maintaining a high relative humidity around the concrete by means of application of moisture in the form of vapour or steam. The system and requirements shall be similar to those specified under **10.3.1**. The temperature range to be maintained shall be as recommended by the manufacturer of the equipment, depending upon the product requirements.

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10.3.3 *Steam Injection during Mixing of Concrete*

Low pressure saturated steam shall be injected into the mixer during the mixing of aggregates. This process shall raise the temperature of the concrete to about 60 °C. Concrete so produced, when placed in the moulds, shall develop high early strength.

10.3.4 *Heated Air Method*

The concrete elements are kept in contact with hot air with a relative humidity not less than 80 percent. This method is particularly suitable for light weight concrete products using porous coarse aggregates.

10.3.5 *Hot Water Method*

The concrete elements shall be immersed in a bath of hot water maintained at a temperature between 50 °C and 80 °C. The principles of this method shall generally be similar to those of steam curing.

10.3.6 CO_2 *Curing*

Carbon dioxide shall be injected into the concrete during mixing. This promotes faster hydration and is particularly effective in accelerating the hardening of precast concrete elements, thereby improving both efficiency and quality.

10.3.7 Concrete elements subjected to any of the accelerated curing methods specified in 10.3.2 to 10.3.6 shall be subjected thereafter to normal curing methods, so as to enable the development of the final strength.

10.3.8 *Other Methods of Accelerated Hardening*

Accelerated hardening of concrete may also be achieved by one or more of the following techniques:

a) *Use of Construction Chemicals* — Suitable construction chemicals may be used for promoting early hardening;

b) *Consolidation by Spinning* — This method shall generally be employed in centrifugal moulding of pipes and similar units. The spinning action removes excess water, effects consolidation, and permits earlier demoulding;

c) *Pressed Concrete* — This method shall be suitable for the fabrication of small or large products at high rates of production. Presses in the range of 100 t to 200 t capacity shall be used to compress the wet concrete in rigid moulds, thereby expelling excess water. This results in early handling and produces dense, wear-resistant concrete;

d) *Vacuum Treatment* — In this method, surplus air and water shall be removed from the freshly placed concrete (for example, slabs and similar elements). A suction of up to 70 percent of an atmosphere shall be applied for a period of 20 min to 30 min per cm thickness of the units; and

e) *Consolidation by Shock* — This method shall be suitable for small concrete units placed in strong moulds and subjected to repeated dropping from a height. The number of shocks required to expel excess water and air may vary from 6 to 20 and the height of drop may be up to half the depth of the mould.

10.3.9 *Stacking During Transport and Storage*

Every precaution shall be taken against overstress or damage, by the provision of suitable packing at agreed points of support. In general, the final condition of the elements in the structure should be studied to finalize on the staking support. Even number of supports is preferred over the odd number of supports. Particular attention is directed to the inherent dangers of breakage and damage caused by supporting other than at two positions, and by the careless placing of packing (for example, not vertically one above the other). Ribs, corners and intricate projections from solid section should be adequately protected. Packing pieces shall not stain, disfigure or otherwise cause permanent mark on units or members. Stacking shall be arranged, or the precast units protected, to prevent the accumulation of trapped water or rubbish and, if necessary, to reduce the risk of efflorescence.

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The following points shall be kept in view during stacking:

a) Care should be taken to ensure that the flat elements are stacked as per their final position at the structure. For identification, top surfaces should be clearly marked;

b) Stacking should be done on hard and suitable ground to avoid any sinking of support when elements are stacked;

c) In case of horizontal stacking, packing materials shall be at specified locations and shall be exactly one over the other to avoid cantilever stress in panels;

d) Components should be packed in a uniform way to avoid any undue projection of elements in the stack which normally is a source of accident;

e) In general, vertical stacking should be limited to 6 components to 10 components. Minimum ground capacity shall be 5 t/m^2 for stacking of panels. Where the ground is only compacted, the maximum ground load at support should be restricted to 15 t/m^2. Where the ground is of PCC layer, the maximum ground load at support should be restricted to 30 t/m^2 subjected to capacity of the ground. In both cases, the ground should be levelled, and the support should be continuous along the width of the panels; and

f) All wall panels shall be stacked, preferably symmetrically on both sides, against steel A-frames. The slope of the sides of the A-frame shall be between 75° and 80° to the horizontal and the bottom of the frame shall be provided with concrete footings to resist component of self- weight and horizontal wind pressure on the wall panel. A similar type of A-frame should be used for transporting wall panels.

10.4 Handling Arrangements

10.4.1 Lifting and handling positions shall be clearly defined particularly where these sections are critical. Where necessary special facilities, such as bolt holes or projecting loops, shall be provided in the units and full instructions supplied for handling. For bolts/hooks, bond strength shall be the criteria for embedded bolts and bearing strength for through bolts. For bond strength, pull out test of concrete shall be carried out. *See* **10.8** for additional design considerations.

10.4.2 For precast pre-stressed concrete members, the residual pre-stress at the age of operation of handling and erection shall be considered in conjunction with any stresses caused by the handling or erection of member. The compressive stress thus computed shall not exceed 50 percent of the cube strength of the concrete at the time of handling and erection. Tensile stresses up to a limit of 50 percent above those specified in Part C/Section 5 of this NBCS shall be permissible.

10.5 Identification and Marking

All precast units shall bear an indelible identification, location and orientation marks as and where necessary. The date of manufacture shall also be marked on the units.

The identification markings on the drawings shall be the same as that indicated in the manufacturer's literature and shall be shown in a tabular format on the setting schedule together with the length, type, size of the unit, and the sizes and arrangement of all reinforcement. Implementation of QR Code or similar stickers pasted on precast elements with above mentioned details are highly recommended.

10.6 Transport

Transport of precast elements inside the factory and to the site of erection is of considerable importance not only from the point of view of economics but also from the point of view of design and efficient management. Transport of precast elements shall be carried out with extreme care to avoid any impact and distress in elements and handled as far as possible in the same orientation as it is to be placed in the final position. *See* **10.8** for additional design considerations.

10.6.1 *Transport Inside the Factory*

Transport of precast elements moulded inside the factory depends on the method of production selected for the manufacture as given in Table 13.

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10.6.2 *Transport from Stacking Yard Inside the Factory to the Site of Erection*

10.6.2.1 Transport of precast concrete elements from the factory to the site of erection should be planned in such a way to be in conformity with the traffic rules and regulations as stipulated by the Authorities. The size of the elements is often restricted by the availability of suitable transport equipment, such as tractor-cum-trailers, to suit the load and dimensions of the member in addition to the opening dimensions under the bridge and load carrying capacity while transporting the elements over the bridge.

10.6.2.2 While transporting elements in various systems, that is, wagons, trailers, trucks, etc, care should be taken to avoid excessive cantilever actions and desired support are maintained. Transportation of prefabricated elements should be carried out with care. Sufficient care should be taken to also protect the steel inserts and reinforced bars of the precast elements during transportation. Precast elements should be secured with safety ties to ensure that vibrations to the elements in transit are minimum. Special care should be taken at location of sharp bends and on uneven or slushy roads to avoid undesirable stresses in elements.

10.6.2.3 Before loading the elements, care should be taken to ensure that the base packing for supporting the elements is located at specified positions only. Subsequent packing shall be kept strictly one over the other.

10.7 Erection

10.7.1 Tower cranes or other mobile cranes have varying lifting capacities at different radii, with the maximum weight at a radius closer to the centre and the minimum weight at a radius farther from the centre. To minimize the number of precast elements, the selected crane should be capable of handling the heaviest precast elements at its maximum radius. The floor plan should indicate the weight of panels at different crane radii, ensuring they align with the crane's lifting capacity at each radius.

In the erection of precast elements, all the following items of work are meant to be included:

a) Slinging of the precast element;

b) Tying up of erection ropes connecting to the erection hooks;

c) Cleaning of the elements and the site of erection;

d) Cleaning of the steel inserts before incorporation in the joints, lifting of the elements, setting them down into the correct envisaged position;

e) Adjustment to get the stipulated level, line and plumb by means of levelling bolts and adjustable props;

f) Welding of cleats;

g) Changing of the erection tackles;

h) Putting up and removing the necessary scaffolding or supports;

j) Welding of the inserts, laying of reinforcements in joints and grouting the joints; and

k) Finishing the joints to bring the whole work to be like finished product.

See 10.8 for additional design considerations.

10.7.2 Since the erection work in various construction jobs using prefabricated concrete elements differs from place to place depending on the site conditions, safety precautions in the work are of utmost importance. Hence, only those skilled foremen, trained workers and fitters who have been properly instructed about the safety precautions to be taken should be employed on the job.

10.7.3 Transport of people, workers or visitors, by using cranes shall be strictly prohibited on an erection site.

10.7.4 *For Travelling Cranes*

a) The travel tracks shall be perfectly levelled both longitudinally and transversally;

b) The gauge shall be constant and the rail tracks perfectly straight and with the same shape along the entire path;

c) The rail tracks shall be placed on a solid base;

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d) Travelling stop buffers shall be placed at the ends of the rail tracks;

e) For rail shape tolerances, the OEM's installation manual shall be referred; and

f) The track of the crane should be checked daily to see that all fish plates and bolts connecting them to the sleepers are in place and in good condition.

10.7.5 The operation of all equipment used for handling and erection shall follow the operations manual provided by the manufacturer. All safety precautions shall be taken in the operations of handling and erection.

10.7.6 *Erection Tolerances*

As per Table 15.

Table 15 Erection Tolerance for Precast Components

(*Clause* 10.7.6)

Sl No.	Product	Tolerances mm
(1)	(2)	(3)
i)	Level differences between support lines of floor or wall panels	± 5
ii)	Plumb lines of wall panels	± 5
iii)	Bearing for precast floor panels	± 5
iv)	Joint dimensions	± 5
v)	Maximum accumulated deviation	
	a) Till 10 storey building	1/1 250 of height or length or ± 20
	b) Greater than 20 storey building	1/1 250 of height or length or ± 50 (whichever is lesser)
vi)	Variation in plan location (any column or beam, any location)	± 10 for column ± 15 for beams
vii)	Variation in plan parallel to specified building lines	+ 2 mm per meter for any beam less than 6 m long or adjacent to columns paced 6 m apart or less 15 mm maximum for adjacent columns spaced more than 6 m

10.8 Design Considerations

Based on the design intent, production environment and aesthetic requirements of the prefabricated panels, the resultant stresses should be controlled to minimize cracking in precast and/or prestressed panels. Precast (non-prestressed) components are generally designed as 'cracked' unless cracking is an aesthetic concern. However, demoulding of precast elements shall not be allowed until the concrete achieves a strength of at least 1.25 times the (extreme fibre) stress which the concrete may be subjected at the time of demoulding. The same shall be applicable while transporting and erection. Reference may be made to Table 7 for load multiplier to ascertain handling stress. The strength referred to shall be that of concrete using the same cement and aggregates and

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admixture, if any, with the same proportions and cured under conditions of temperature and moisture similar to those existing on the work.

10.8.1 Structural design requirements of minimum spacing, skin reinforcement and crack width criteria as specified in Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS should be applicable.

10.8.2 Lifting hooks should be located such that while lifting the panel, minimum bending moments are produced.

10.8.3 Static load multipliers should be used to account for impact, and demoulding forces as per Table 7.

10.8.4 The load multipliers are only needed for the flexural design and are not applicable for the lifting devices. Lifting hooks shall be designed as per **7.14.6**.

10.8.4.1 For wall panels cast vertically on tilting moulds, battery moulds, and other vertically cast elements, the required concrete strength is not always dictated by flexural stresses. Instead, the minimum concrete strength during the initial lift may be governed by the capacity requirements of the lifting inserts. The capacity of the lifting inserts may be lower than their rated value due to factors such as inadequate embedment depth, insufficient edge distances, or low concrete strength at the time of lifting. Therefore, higher concrete strengths than those recommended in Table 9 may be necessary to fully develop the capacity of certain lifting inserts and ensure the safe handling of the panels.

10.8.4.2 For prestressed elements, special care must be taken to ensure that lifting devices are anchored in compression zones, unless otherwise specified by the design.

10.8.4.3 Initial lifting should be done cautiously and gradually to overcome suction and friction, avoiding the application of sudden impact forces.

10.9 Equipment

10.9.1 The strategic equipment used in the precast concrete industry/construction may be classified into the following categories:

a) *Production equipment* — Batching plant, concrete shuttle conveyors concrete vibrators, circulation tables, EOT, gantry, shear lines, stirrup making machines, weld mesh machines, robots;

b) *Erection equipment* — Tower cranes, Mobile cranes, derricks, hoists, chain pulley blocks, etc;

c) *Transport machinery* — Tractor-*cum*-trailers, dumpers, lorries, locomotives and rarely even helicopters;

d) *Workshop machinery* — For making and repairing steel and timber moulds;

e) Minor tools and tackles, such as lifting equipment, electric vehicle-dumpers, concrete buckets, etc; and

f) Steam generation plant for accelerated curing.

Each of the above groups may further be classified into various categories of machines and further to various other types depending on the source of power and capacity. For use of renewable energy sources to achieve sustainability in precast construction **G-3** shall be referred.

10.9.2 *Mechanization of the Construction and Erection Processes*

The processes can be mechanized/automated, similar to other industries, to achieve the benefits of mass production of standardized components. This approach will enhance productivity, lower production costs over time and ensure consistent product quality.

10.10 Systems for Mixed and Composite Construction

The system of mixed and composite construction depends on the extent of the use of prefabricated components, their materials, sizes and the technique adopted for their manufacture and use in building.

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10.10.1 *Combinations of System Components for Mixed and Composite Construction*

The following combinations may be used in mixed and composite construction:

a) Structural steel work and timber roofs on precast frames;

b) Precast half slab, solid slabs, hollow core slabs (floors) onto steel and concrete beams and masonry walls;

c) Profiled metal decking on precast beams;

d) Cast in situ slab on precast wall;

e) Precast slab on cast in situ wall/3D printed wall;

f) Precast frames onto cast in-situ foundations, retaining walls, etc;

g) Precast frames stabilized by masonry walls, steel bracing, etc;

h) Precast cladding in steel or cast in-situ frames and vice-versa;

j) Glass curtain walling, stone cladding or metal sheeting onto precast concrete frames, etc; and

k) Reinforced concrete and structural steel as composite columns and beams.

10.10.2 Precast concrete may be combined with cast in-situ concrete, often termed hybrid construction. Cast in-situ is mostly used to form homogenous connections between precast elements and provide a structural topping for horizontal diaphragm action. In other cases, it is used to form the foundations and sub-structure to the building.

10.10.3 Structural steel work is largely used in long span prestressed concrete floors (hollow core) supported on rolled and prefabricated steel beams and also as steel roof trusses supported on concrete columns.

10.10.4 Timber may be used as long span glue-laminated beams and rafters, with precast concrete. Precast floors may be used in timber frame construction. Similarly, timber frames with precast elements shall be used as a building system.

10.10.5 Brick and block masonry may be combined with precast concrete structures and floors. The most common combination is to use prestressed floors on load bearing walls.

NOTE — Any prefabricated floor system covered under other Indian Standards, for example, reinforced concrete plank system and speed floor system may be combined with load bearing masonry walls/insulating concrete form (ICF) walls/precast large reinforced concrete wall panels/precast or cast in-situ reinforced concrete column-beam frames provided overall structural design safety, stability and integrity for composite structure is ensured in accordance with this section and other safety standards, such as good practice [C-7(1)], **5** of Part C/Section 1 of this NBCS including fire safety and safety against progressive collapse as per provisions of respective good practices, such as [C-7(19)], Part C/Section 5 and Part C/Section 6A of this NBCS.

10.10.6 *Design Considerations*

The mixed and composite structures shall be analysed appropriately and the joints in them designed to take the forces of an equivalent discrete system. Resistance to horizontal loading shall be provided by placing beams, walls and/or bracings in two directions at right angles or otherwise. The individual components shall be designed, taking into consideration appropriate end conditions and loads at various stages of construction. The components of the structure shall be designed for loads in accordance with good practices [C-7(1)] and[C-7(20)] and **5** of Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS. In addition, members shall be designed for handling, erection and impact loads that may be expected during handling and erection.

For mixed and composite construction, the following shall be considered:

a) *Positions of stability cores*, *walls*, *bracing*, *etc* — In high rise buildings, the most popular method is a central core constructed several storeys ahead of the framework. The core could be built using either precast concrete panels, or cast in-situ walls, or steel cross bracing or precast concrete diagonal bracing;

b) *Maturity of connections* — Cast-in-situ grouted joints need temporary propping until desired design strength is achieved unless combined mechanical connections are also used. The overall construction sequence should be planned ahead of time carefully as per the design requirements. The design of temporary bracing system using key components should be looked at critically;

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c) *Availability and/or positioning of equipment to transport and erect components* — The size and weight of the various components shall be organized to make optimum use of crane capacity, for example, the lightest units farthest from the operating zone;

d) *Erection safety and speed of construction, with attention to cast in-situ concreting sequences* — This is particularly important where fixing gangs are unaccustomed to working with different materials; and

e) *Tolerances for construction* — This is particularly important where different manufacturers are producing components in different materials.

Other design considerations and safety requirements against progressive collapse shall be in accordance with this NBCS.

10.11 Prefabricated Structural Units

For the design and construction of composite structures made up of prefabricated structural units and cast in-situ concrete, reference may be made to good practice [C-7(22)].

For design and construction of precast reinforced and prestressed concrete triangulated trusses, reference may be made to good practice [C-7(23)].

For design and construction of floors and roofs using various precast units, reference may be made to good practice [C-7(24)].

For construction of floors and roofs with joists and filler blocks, reference may be made to good practice [C-7(25)].

11 QUALITY ASSURANCE AND QUALITY CONTROL

The provisions for quality assurance as per good practice Part C/Section 5 of this NBCS shall apply. A documented quality assurance program covering various stages from precast unit/element production to erection shall be implemented and maintained. Common defects and remedies in precast concrete elements are presented in Annex F. For achieving sustainability in precast concrete construction a reference may be made to Annex G. Additionally, the flow chart given in Fig. 41 shall be considered.

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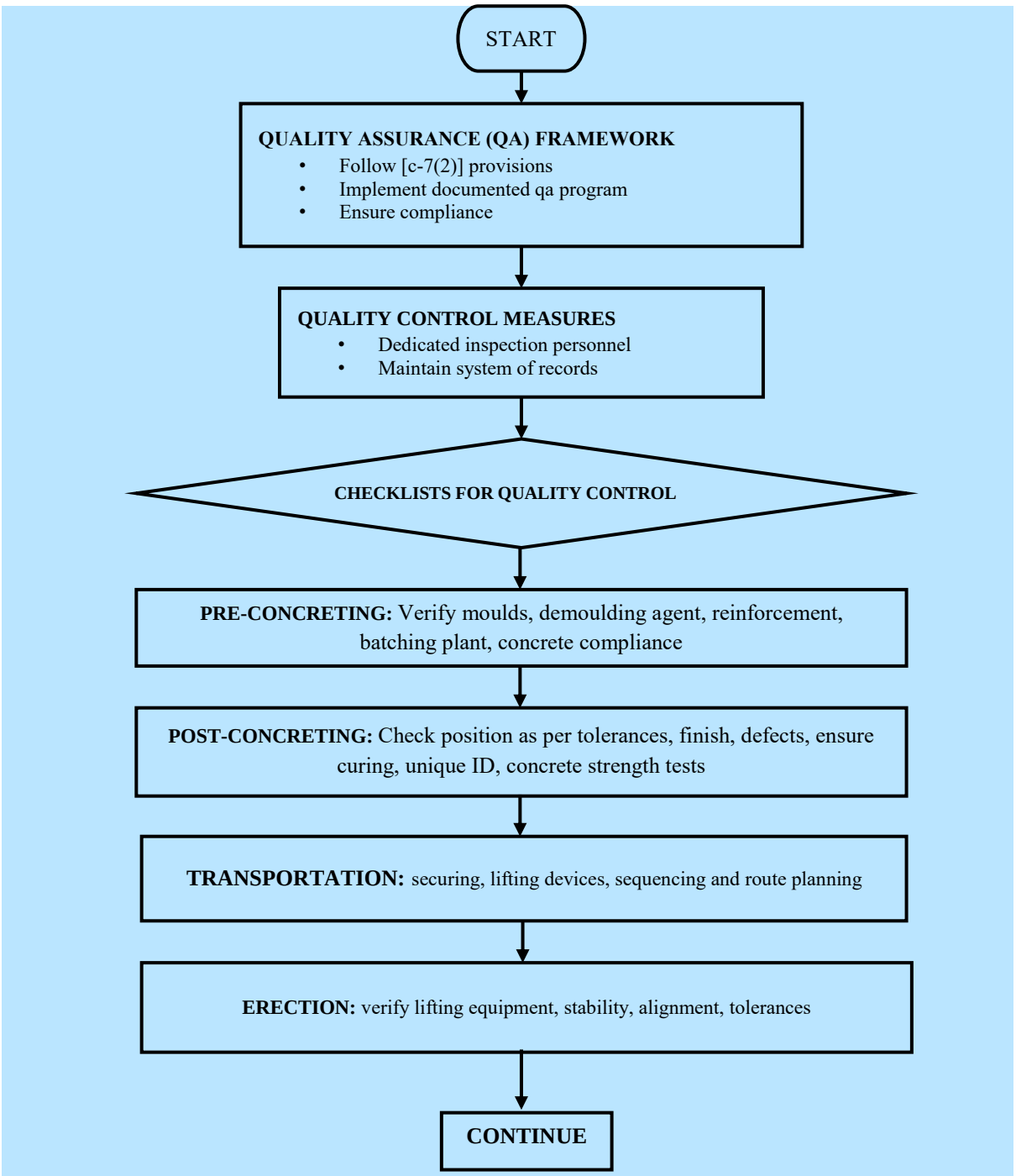

FIG. 41 FLOWCHART FOR QUALITY ASSURANCE AND CONTROL (CONTINUED)

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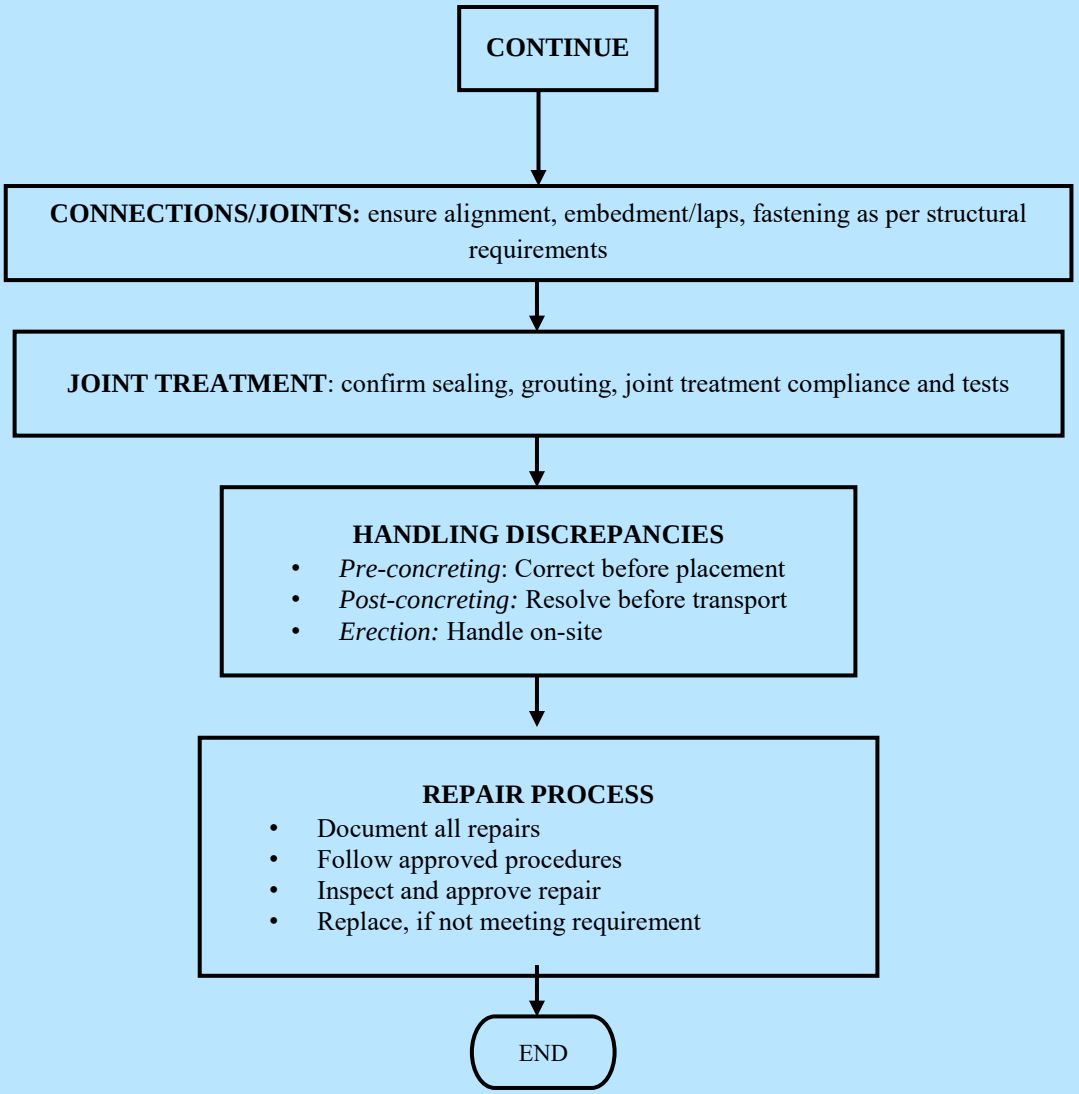

FIG. 41 FLOWCHART FOR QUALITY ASSURANCE AND CONTROL

12 TESTING

12.1 Sampling procedures

12.1.1 *Lot*

12.1.1.1 All the precast units of same size, manufactured from same material under similar conditions of production shall be grouped together to constitute a lot.

12.1.1.2 The number of units to be selected from each lot for dimensional requirements shall depend upon the size of the lot and shall be in accordance with col (3) and col (4) of Table 16.

12.1.1.3 The units shall be selected from the lot at random. In order to ensure the randomness of the selection, reference may be made to accepted standard [C-7(26)].

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Table 16 Sample Size and Rejection Number

(*Clauses* 12.1.1.2 *and* 12.1.2.1)

Sl No.	Lot Size	First Sample Size	Second Sample Size	First Rejection Number	Second Rejection Number
(1)	(2)	(3)	(4)	(5)	(6)
i)	Up to 100	5	5	2	2
ii)	101 to 300	8	8	2	2
iii)	301 to 500	13	13	2	2
iv)	501 and above	20	20	3	4

12.1.2 *Number of Tests and Criteria for Conformity*

12.1.2.1 All the units selected at random in accordance with Table 16 shall be subjected to testing of dimensional tolerance requirements. A unit failing to satisfy any of the dimensional requirements shall be termed defective. The following shall be considered:

a) If no defect is found in the sample, the lot shall be considered as conforming else shall be rejected, if the number of defectives is greater than or equal to the first rejection number;

b) If the number of defectives is less than the first rejection number, the second lot shall be subjected to testing for dimensional tolerance requirements; and

c) If the combined number of defectives is less than the second rejection number, the lot shall be considered as conforming to dimensional tolerance requirements; otherwise not.

12.1.2.2 The lot which has been found satisfactory with respect to dimensional tolerance requirements shall then be considered for load testing as per **12.4**. One unit out of every 300 units or part thereof shall be selected for testing. The lot shall be considered as conforming to the strength requirement, if all units meet the requirement; otherwise not.

12.2 Testing on Individual Precast Units

12.2.1 The flexural components shall be load tested individually or along with the structural topping, depending on the requirements of the project or if any structural cracks (as per Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS) or damages are observed. The testing shall be carried out on the selected sample along with one undamaged unit or part for reference. If individual element is tested, then the loads to be placed should be prorated such that element experiences similar demand over capacity levels when loaded. In either case, the element should be loaded for 24 h at its full span with a total load of 1.25 times the sum of dead (including SIDL, etc) and imposed loads used in the design. At the end of this time, it should not show obvious signs of deterioration or excessive deflection. For large span elements, larger than 10 m span, with design stresses near cracking under the ultimate loads, it is not uncommon to see hairline cracks during the load test at maximum loads in sustained conditions. Except for the design criteria of un-cracked elements, if applicable, if the test elements in such cases perform at a satisfactory level with recovery as noted in **12.2.2** and **12.2.3** as applicable, the test should be considered passed. If such cracks are substantial, the test should be further evaluated by the engineer-in-charge.

12.2.2 The recovery of maximum deflection observed during testing after 24 h should not be less than 75 percent.

12.2.3 If prestressed, it should not show any visible cracks up to the working load, and the recovery of maximum deflection observed during testing after 3 h should not be less than 85 percent.

12.2.4 For inspection and testing of the compression members, provision of Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS shall apply.

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12.3 Testing of Connecting Devices and Inserts

12.3.1 Grouted coupler or mechanical coupler shall be tested according to accepted standard [C-7(9)]. Where the grouted couplers are used for connections of lateral force resisting system (LFRS) transferring seismic and wind lateral loads, the grouted couplers should also be tested for the reverse cyclic load as per Table 17, in an NABL accredited laboratory or institute of repute under the supervision of appropriate authority.

Table 17 Cyclic Tension and Compression Test for Grouted Coupler

(*Clause* 12.3.1)

Sl No.	Stage	Tension	Compression	Cycles
(1)	(2)	(3)	(4)	(5)
i)	I	0.95 f_y	0.5 f_y	20
ii)	II	2ε_y	0.5 f_y	4
iii)	III	5ε_y	0.5 f_y	4
iv)	IV	Load in tension till failure		

where

f_y = specified yield strength of the reinforcing bar; and

ε_y = strain of reinforcing bar at actual yield stress.

12.3.2 Any new connection assemblies such as, but not limited to, column shoe or shear connectors, that are used in connection of LFRS as noted in 12.3.1, shall be tested for monotonic as well as reverse cyclic load as per 12.3.1 in an NABL accredited laboratory or institutes of repute under the supervision of appropriate authority. The failure should only occur in the reinforcement with evidence of ductility without any damage or deterioration in the brittle components of the connection assembly.

12.3.3 Lifting inserts and devices shall be tested for pull-out and should not show any damage till design load (factor of safety times the working load). Factor of safety shall be according to 7.14.

12.4 Floor Load Testing of Structure or Part of Structure

12.4.1 Floor load testing on a part or entire floor shall be carried out before occupancy, depending on the requirements of the project or if any structural cracks or damages are observed or to ensure adequacy of strength or serviceability.

12.4.2 The structure should be loaded for 24 h with the full dead load and an imposed load equal to 1.25 times the specified imposed load used in design. The imposed load shall be removed, and the deflection shall be monitored. During the test, vertical struts should be placed appropriately to ensure safety.

NOTE — Dead load includes the self-weight of structural members plus weight of finishes and walls or partitions, if any, as considered in the design.

12.4.3 The recovery of maximum deflection shall be at least 75 percent otherwise the test shall be repeated after 72 h. If the recovery after second test is less than 80 percent the structure shall be deemed to be unacceptable. If unacceptable, the strengthening of this structure shall be carried out and 80 percent recovery of the structure shall be verified. The strengthening of the structure shall be verified by relevant authority.

12.4.4 If prestressed, the recovery of maximum deflection should be at least 85 percent, otherwise test shall be repeated after 72 h. If the recovery after second test is less than 85 percent the structure shall be deemed to be unacceptable.

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12.4.5 If the maximum deflection in mm, shown during 24 h under load is less than 40 L^2/D, where *L* is the effective span in m; and D is the overall depth of the section in mm, it is not necessary for the recovery to be measured and the recovery provisions of 12.4.3 and 12.4.4 shall not apply.

12.4.6 *Testing of Connection/Joint and System*

12.4.6.1 Testing of connection or system shall be performed for proprietary or unique connection or system.

12.4.6.2 Monotonic test shall be carried out for beam-column, walls-slab, wall-wall joints for quantifying the static behaviour.

12.4.6.3 Reverse cyclical load simulation test under intended loads of beam-column, wall-slab, wall-wall joints, shall be carried out for quantifying the energy dissipation and hysteresis behaviour.

12.4.6.4 System level testing shall be carried, if required by the contract or specification of the project.

12.4.6.5 Testing shall be carried out in reputed institutes with appropriate requirements.

12.4.6.6 Acceptance criteria for such test should be mutually established by the authority and the concerned institute where testing is being conducted.

12.5 Testing for Water Tightness

12.5.1 *External Precast Wall/Facade Panels*

Any of the testing procedures given in 12.5.1.1 and 12.5.1.2 may be followed for testing of external wall and facade panel joints for water tightness as per the engineer-in-charge.

12.5.1.1 *Testing procedure* 1

12.5.1.1.1 *General*

If it is deemed necessary to demonstrate that the precast facade joints are adequate to resist water penetration, a water penetration test on a full-scale panel including joint mock-ups may be undertaken.

12.5.1.1.2 *Test procedures*

Mock-up panels to be tested for water tightness should be subjected to both static and cyclic pressure in accordance with standard accepted procedures. Available international testing procedures are to be modified for local conditions as follows:

a) Differential test pressure shall be 20 percent of the maximum inward design wind load but not less than 0.77 kPa; and

b) The test shall be performed with a water flow rate of 3.4 litre/min/m² for 15 min.

12.5.1.1.3 *Sampling for a water penetration test*

Water tightness testing of vertical and horizontal joints in precast facade panels shall be carried out at the rate of 0.5 percent for each type of joint and combination of panels or at least one for each joint type and precast unit, whichever is greater.

12.5.1.1.4 *Failure criterion*

The facade units or joints are deemed to have failed, if signs of water seepage through the joints or through the facade unit, including signs of damp patches, are observed during the test or within the subsequent 2 h after the test.

12.5.1.1.5 *Remedial action on failure*

Upon failure of a water penetration test on a precast facade system the cause of failure should be identified and the system revised and re-tested until a satisfactory test result is achieved. Modifications shall be realistic in terms of job conditions and shall maintain standards of quality and durability.

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12.5.1.2 *Testing procedure* 2

12.5.1.2.1 For external walls, a continuous jet of water shall be sprayed on the joint corresponding to 600 litre/h from a water hose having a nozzle velocity of 2 m/s and a cone scatter of approximately 60° held at a distance of 1.2 m from the wall surface under test. The duration of the test shall not be less than 2 h. The hose shall be placed centred on the horizontal or vertical joint width. For external walls, the following tests shall be conducted for each building block as per Table 18.

Table 18 Frequency of Tests for Vertical and Horizontal Joints

(*Clause* 12.5.1.2.1)

Sl No.	Building Height	Frequency of Tests
(1)	(2)	(3)
i)	Up to 10 storeys	5
ii)	Up to 20 storeys	10
iii)	Up to 30 storeys	15
iv)	Up to 40 storeys	20
v)	Up to 50 storeys	25

12.5.1.2.2 Twenty percent of these tests shall be carried out at the intersection of the vertical and horizontal joint.

12.5.1.2.3 These tests shall be carried out at any location as instructed by the engineer-in-charge.

12.5.1.2.4 *Acceptance criterion*

Water shall not seep into any part of the building during the test.

12.5.1.2.5 *Remedial action on failure*

For each test that fails, the tested joint shall be repaired as per the approved repair procedure as directed by the engineer-in-charge and the joint shall be retested for performance as per acceptance criterion. Two additional joints should be tested for each failed joint unless directed otherwise by the engineer-in-charge.

12.5.2 *Floor or Roof Slabs, Toilet Units and Kitchen Units*

12.5.2.1 A total of 5 percent of all horizontal floor slabs and the entire roof slab area, including bathroom/toilet floors, and kitchen floors shall be tested for water tightness by ponding of water for period of 4 h. This test shall be carried out before the application of floor finish or tiling. The depth of ponded water shall be taken as the maximum of the following:

a) At least 25 mm; and

b) 10 mm more than thickness of joint.

12.5.2.2 *Acceptance criteria*

Water shall not seep into any part of the building during the test and after 24 h. The rooms below the tested room shall not show any water leakage. Minor dampness or seepage without dripping water in limited area of less than approximately 50 cm^2 is considered acceptable at a few areas and should be repaired suitably to avoid seepage. In any other cases, the test should be repeated per **12.5.2.3**.

12.5.2.3 *Remedial action on failure*

For each test which fails, the tested joint shall be repaired as per the approved repair procedure as directed by engineer-in-charge and the joint shall be retested for water tightness. Two additional joints/areas should be tested for each failed joint unless directed otherwise by the engineer-in-charge.

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ANNEX A

(*Clause* 4)

SYMBOLS AND NOTATIONS

A_n	Area of tension reinforcement
A_{vf}	Area of shear friction reinforcement
A_1	Loaded area (bearing constant area)
A_2	Area of the lower base of pyramid, cone or tapered wedge
b_t	Bearing width
b_w	Average width of the stem for the full depth of the section
d_e	Distance from centre of load to the end of the ledge
f_{ck}	Characteristic strength of concrete
f_y	Yield strength of tension reinforcement
f_{ys}	Yield strength of shear reinforcement
h_l	Height of beam ledge
L	Anchorage length
l_p	Length of ledge projection
N_u	Factored axial force
s	Minimum spacing of concentrated loads
V_u	Factored shear force
V_{uR}	Design shear resistance
β	coefficient dependent on the bar type
τ_b	Bond stress
τ_{bu}	Design ultimate anchorage bond stress
φ_E	Effective bar size which, for a single bar is equal to the bar size and, for a group of bars

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ANNEX B

(*Foreword*, *Clauses* 6.5.1 *and* G-6.2.2)

GUIDELINE FOR 3D PRINTED CONCRETE STRUCTURES

B-1 GENERAL

This annex provides an overview of the materials, methods and tests associated with concrete construction using 3D printing for various applications. The provisions contained herein are based on the current state of knowledge and practice in 3D printed concrete construction. In addition to these guidelines, reference shall be made to the applicable existing Indian standards and relevant research publications. Ongoing research at the national and international levels is expected to generate design-related data, which may subsequently be utilized for the purpose of standardization.

B-2 METHODOLOGY AND SCOPE

The following points are considered while defining the application and performance guidelines:

a) *Printing technique* — This document considers only extrusion-based 3D printing technology, where layers are deposited one over the other to construct a structure;

b) *Design freedom* — 3D printing allows for flexibility in the geometry and form. The shape, size and topology of any 3D printed concrete element for any application shall not be restricted. However, the performance requirements for 3D printing need to be satisfied;

c) *Material* — The material shall be printable and buildable. As per the current state of knowledge of concrete for 3D printing, the limits on the usage of cementitious materials as given in good practice [C-7(6)] and Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS would not be applicable for 3D printed concrete. The assessment of 3D printability of any concrete mixture needs to be carried out as per the available specialist literature on the subject. The mechanical, durability and long-term properties of the printable material shall be determined, as applicable. In the absence of a well-established procedure for preparation of test specimens for such evaluations, reference shall be made to recognized specialized literature. The testing shall be conducted at accredited laboratories and approved by competent authority;

d) *Performance criteria* — The performance criteria of any 3D printed components shall conform to the relevant Indian Standards for equivalent components. In the absence of any standards, an experiment-based approach shall be followed to satisfy the requirements for the specific element or component; and

e) *Special applications* — In the absence of any relevant standard, the engineer-in-charge, authorised to define the qualification requirements in accordance with the applicable Indian Standards, shall take decisions based on sound engineering judgement and established practice. The testing shall be conducted at accredited laboratories and approved by competent authority.

B-3 APPLICATION — NON-LOAD BEARING

B-3.1 Falsework/Formwork

3D-printed concrete may be used as stay-in-place (permanent) or temporary formwork for new concrete construction. The structural integrity of the formwork shall be either certified by a practicing structural engineer before the application or the formwork shall satisfy deformation limits and structural requirement mentioned in good practice [C-7(26)]. The formwork shall be designed by testing. The stay in place formwork shall not be considered as a part of the structural member.

B-3.2 3D Printed Concrete Building Blocks

The blocks can be solid as well as hollow, following the definition in accepted standard [C-7(27)]. The blocks shall have any external dimension greater than the corresponding dimension of a brick as specified in accepted standard [C-7(28)]. The size and the mass shall be such that it can be handled by as single person. In case of mechanized placement of the blocks, larger units can be used. The height of the block shall not exceed either its

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length or six times its width whichever is less. The faces of the block shall be manufactured based on the mutual agreement of the manufacturer and supplier/purchaser. Accepted standard [C-7(27)] shall be followed for considering the tests required to satisfy the performance criteria of the blocks.

B-3.3 Autoclaved 3D Printed Blocks

The autoclaved 3D printed solid blocks shall be tested based on the requirements mentioned in accepted standard [C-7(29)]. The blocks shall be of such dimensions and shape that they can be conveniently handled by a single person. The face of the block need not be plain, and the design shall be decided mutually by the manufacturer and supplier/purchaser.

B-3.4 Non-Load-Bearing 3D Printed Walls

3D printed concrete structures can be used as non-load bearing walls. The non-load-bearing walls can be divided into three types as mentioned as per Part C 'Structural Design, Section 4 Masonry' of this NBCS.

a) *Panel walls* — An exterior non-load-bearing wall in framed construction, wholly supported at each story but subjected to lateral loads;

b) *Curtain walls* — A non-load-bearing wall subject to lateral loads. It may be laterally supported by vertical or horizontal structural members, where necessary; and

c) *Partition walls* — An interior non-load bearing wall, one storey or part storey in height.

Such walls can be assessed for the performance requirements in the given application, and if necessary, testing of individual elements with necessary connection details can be done to ascertain lateral load resistance and intended performance.

B-3.5 Precast 3D Printed Panels

Precast 3D printed panels can be used for external or internal non-load-bearing walls. The panels shall be connected considering the guidelines provided in **8**. The printing and erection tolerance shall follow the permissible limits mentioned in **10.7**. The designing of the wall panels (including reinforcement) shall be provided by a certified structural engineer using this NBCS or following the concept of design by testing (in the absence of standardized design methods).

B-3.6 External Facing and Veneers

3D printed components can be used for external facing following the guidelines mentioned in good practice [C-7(30)]. Precast-facing concrete blocks shall generally conform to the requirements of accepted standard [C-7(27)] and, in addition, shall have special treatment regarding durability, colour and surface textures for the exposed facing. The face finish of slabs shall also be adequately waterproofed. The joint details, fixing technique and method of attachment shall be provided by a certified structural engineer following the concept of design by testing, in the absence of standardized design methods.

B-3.7 Acoustic

The acoustic treatment of non-industrial buildings, industrial establishments, and assembly spaces such as cinema halls or conference rooms is achieved through the design flexibility offered by 3D printing. The incorporation of air cavities between the inner and outer shells of walls contributes to the reduction of sound transmission between rooms. Panel and partition walls nay be printed with a suitable spacing between face shells as suggested in good practice [C-7(31)]. The semi-discontinuous and discontinuous construction mentioned in the good practice [C-7(31)] can be followed for non-industrial buildings. For industrial establishments and cinema halls or conference rooms, the reverberation time governs the acoustic experience. Good practices [C-7(32)] shall be followed to determine the reverberation time required for each application. The face of the walls and ceilings can be designed to govern the reverberation time while satisfying the requirements in the standards mentioned.

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B-4 APPLICATION — LOAD BEARING

B-4.1 Design Philosophy

3D printed concrete structure shall satisfy the strength as well as serviceability requirements. The scope of this NBCS is limited to the load-bearing members in a building. Other structural applications are not considered for the purpose of this NBCS. Additionally, the durability and long-term behaviour of the printed structure shall be considered during design.

The load-bearing walls consist of a 3D printed shell and cast in-situ reinforced concrete core. In these cases, the 3D printed shell is considered to act integrally with the core. Depending on the application and the loading, the requirement of the reinforced concrete core may be done away with, in which case, the shell itself is capable of resisting both in-plane and lateral loads.

The design of load bearing walls shall be dependent on the load resistance mechanism. For the following two applications, the design using available information, and based on established principles of analysis and deriving load resistance, as commonly used in established codes of practice may be followed:

a) If the load resistance is derived from cast in-situ reinforced concrete core, (*see* Fig. 42); and

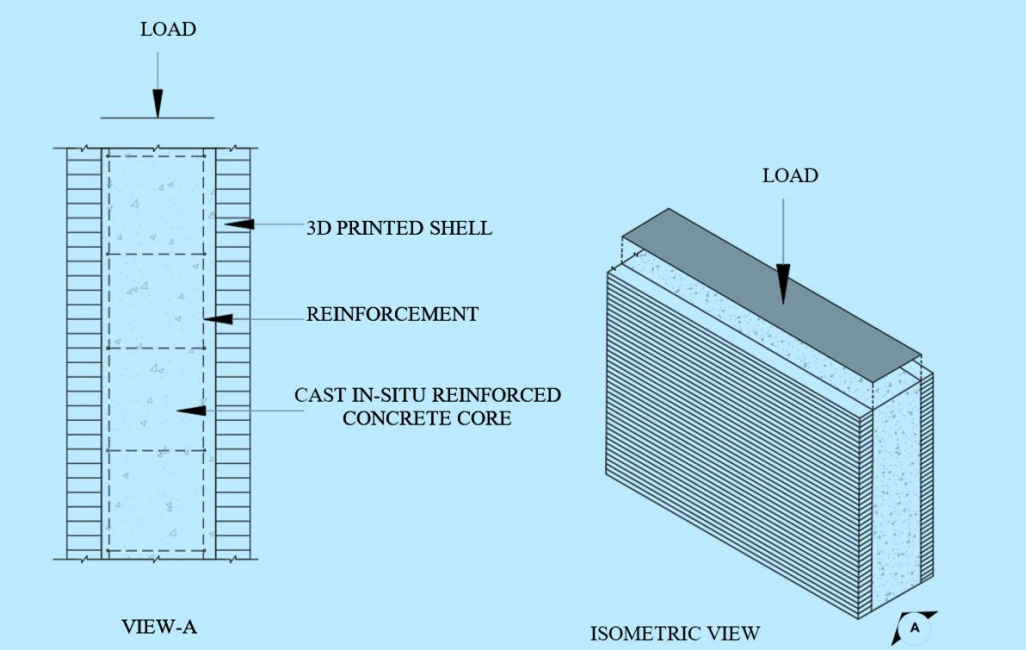

FIG. 42 LOAD RESISTANCE IS DERIVED FROM CAST IN-SITU REINFORCED CONCRETE CORE

b) If both 3D printed shell and cast in situ reinforced core are made using the same concrete mix, resulting in monolithic behaviour (*see* Fig. 43).

Where load resistance is derived using composite action between printed shell and cast in-situ core, design by testing the individual components may be followed.

All attempts to make the printed object behave in a ductile manner should be made to the extent possible, irrespective of the nature of usage.

NOTE — Specialized literature may be referred to while designing the test methods for the evaluation of structural walls. Alternatively, load-bearing 3D printed structures can be constructed with the approval of a certified external authority.

The use of load bearing walls is limited to gravity load. The use for load resistance subjected to earthquake loading as per good practice [C-7(20)] and **5** of Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS and the requirements of ductile detailing as per good practice [C-7(1)] are outside the scope of this annex. Design by testing approach may be followed for any specific application. Additionally, the foundation shall comply all the requirements of good practice [C-7(33)] and to any other relevant Indian standards.

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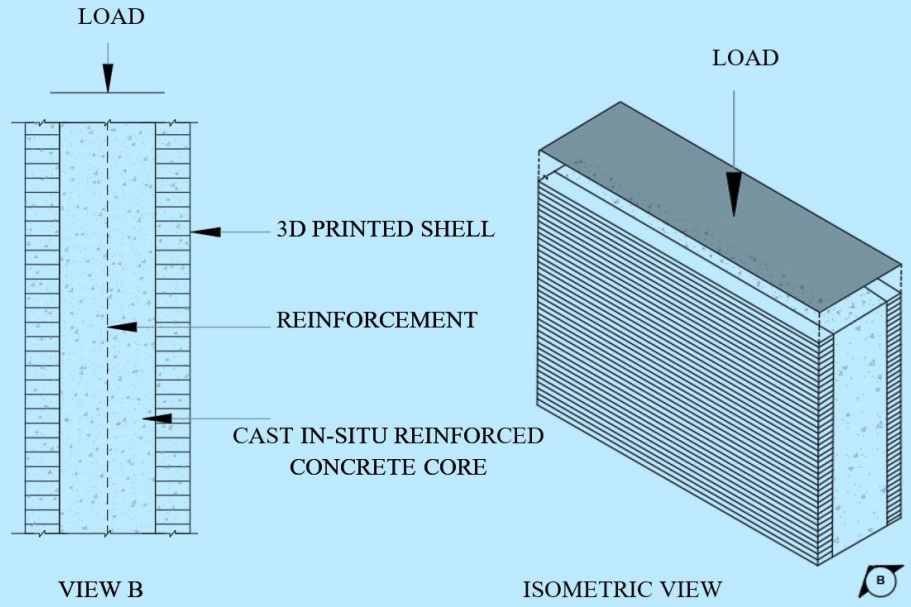

FIG. 43 LOAD RESISTANCE IS DERIVED FROM 3D PRINTED SHELL AND CAST IN-SITU REINFORCED CONCRETE CORE

B-4.2 Loads

The load criteria shall be as per Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS. The lateral and earthquake load shall be taken special care of while designing the load bearing structures. The components of the structures shall be tested at full scale individual element level with necessary boundary conditions.

B-4.3 On-Site and Off-Site Construction

In case of an on-site construction, the printer is expected to be stationed at the location and construction to be carried out in-situ. For off-site construction, panels/sub-parts of the structure are printed in a factory and assembled at the site. In such a case, particular attention is to be paid to the design of the connections, as per the prevailing standards and codes.

B-4.4 Load Bearing Wall

3D printed load bearing walls can be used for buildings up to a height where the effect of lateral load is not severe, which may be restricted to building of height of 12 m. For buildings beyond 12 m height, the structural design and construction methodology shall be approved by the competent authority. Special care shall be taken while designing the connection between wall to slab, wall to beam and wall to footing. The minimum effective thickness of the load bearing wall shall be 100 mm in accordance with the guidelines of Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS. The 3D printed load bearing wall can be considered as:

a) 3D printed perimeter (with or without printed internal rib filaments) and cast in situ concrete core. Depending on the application and extent of loading, these walls can be fully solid (that is with the core completely filled with cast in situ reinforced concrete), hollow (that is without any internal filling) or partially filled (with reinforced concrete in some of the cavities); or

b) Completely printed wall with no hollow space.

The cover depth of 3D printed wall shall consider exposure condition and fire resistance as per Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS. The cover depth should include the thickness of the 3D printed perimeter filament and the cover (if any) provided by the core concrete. The minimum reinforcement criteria laid down in Part C 'Structural Design, Section 5 Structural Concrete' 5 of this NBCS shall be followed. Horizontal reinforcement in the form of high yield strength deformed (HYSD) rebar or welded wire mesh or textile mesh shall be provided to connect the outer leaves to the inner core, when the design assumes a composite action of the 3D printed perimeter and the inner core. In other cases, the printed leaves shall be considered as stay-in-place formwork and shall not be taken as a part of the load-bearing wall. However, even in such cases, efforts

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should be made to ensure that spalling or collapse of the outer periphery does not occur in extreme events such as earthquakes.

The walls shall satisfy the strength and serviceability criteria of the load bearing walls as per Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS. If required, the design shall be on basis of testing of individual elements with necessary boundary conditions or connection details. For testing guidelines, specialist literature and publication may be referred if necessary.

NOTE — Specialist literature/publication may include those published by International Code Council, etc.

B-4.5 Load Bearing Column

Column shall be designed using as per Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS. If required, test results obtained by full scale testing of individual elements with necessary boundary conditions or connection details shall be used. Buckling load criteria shall be satisfied considering the end-restraint conditions by testing. A certified structural engineer shall design the column as per logical reasoning based on experimental evidence.

B-4.6 Shear Walls

The shear walls shall be solid walls designed as per the requirements of good practice [C-7(2)]. The design guidelines similar to that of load-bearing 3D printed walls shall apply. The performance of 3D printed shear walls under lateral and earthquake load shall be tested. The design shall be on basis of experimental outcomes by a certified structural engineer.

B-4.7 Load-Bearing Beams and Slabs

The 3D printed beam and slab shall satisfy the load and serviceability requirements mentioned in Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS. The flexural capacity of the printed beam shall be tested using 3-point and 4-point bending test at full-length-scale element level. The characteristic strength of the printable mix shall not be used to design the flexure member. The results from full-length-scale element level tests shall be used. A certified structural engineer shall design the flexural members as per the reports from the testing.

B-4.8 Foundation

A 3D printed foundation shall be designed following the design by testing method. The foundation can be printed in-situ or assembled at site like precast foundation after printing at the factory. The foundation shall be designed by a certified structural engineer while considering inputs from geotechnical engineers about the soil bearing characteristics. Durability requirements, as mentioned in good practice [C-7(2)], for the foundation shall be strictly satisfied.

If the outer perimeter of the foundation is only printed and the core is cast as conventional foundation, then the printed part shall not be taken into the load calculation and shall be treated as a formwork.

B-4.9 Retaining Wall and Basement

Retaining wall and basement shall be solid for load-bearing applications. Reinforcement strategy shall follow the guidelines mentioned for 3D printed load bearing walls.

B-4.10 Concrete-Reinforcement Bond Strength and Development Length

These are to be determined for each case and the structural engineer shall take into account the same while designing the reinforcement. The effect of layer time gap and moisture loss shall also be considered while designing the reinforcement. The bond between the rebar and the printed concrete shall be measured before designing the reinforcement.

B-4.11 Connections

Connections between the elements, including, but not limited to between two printed elements; printed non-structural element and cast in-situ structural reinforced concrete element; printed non-structural element and

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precast structural element; printed structural element and precast structural element; follow the requirements given in Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS. If required, the connections shall be designed by testing. However, the connection between cast in-situ reinforced core and other structural elements shall follow standard design practice.

B-5 OTHER APPLICATIONS

B-5.1 Boundary Walls

The 3D-printed boundary wall shall not be a retaining wall or any load-bearing structure.

B-5.2 Design Elements for Outdoor Spaces

These elements shall comprise, but are not limited to, the following:

a) Pavement and other pedestrian movement spaces such as footpaths, plaza assembly spaces, kerb to footpath, steps and ramps;

b) Parking and vehicular movement corridors such as road dividers, road marking, etc;

c) Traffic management units such as gate/access control, barriers, traffic separators, etc;

d) Outdoor public conveniences such as seating, drinking fountains, toilet/washrooms, etc;

e) Shelter and kiosks such as bus shelters, police and telephone booth, food stall, information desk, etc;

f) Outdoor illumination such as street light pedestals, etc;

g) Tree guards, tree grates and planters;

h) Litter bins, and spittoons; and

j) Service utilities relating to water supply network, storm water network, sewerage system, irrigation network, etc.

These structures shall be designed by a certified structural engineer following the reports from experimental evaluation.

B-6 CURING

The printed structures shall be cured following the provisions mentioned in Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS. However, curing shall be initiated at the early age to avoid plastic shrinkage cracks. Water spraying in the form of a fine mist must be started even before the initial set. At any given instance, the surface shall not be dry. The curing shall be done until the designed strength is reached for the 3D printed walls and slabs. Curing compounds can be used to reduce the number of days of water curing. The engineer-in-charge shall take the decision regarding the curing requirements based on experimental evidence.

B-7 QUALITY ASSURANCE AND QUALITY CONTROL

The following properties shall be assessed for determining the quality of the 3D printable concrete mix. The sampling shall be done as per Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS:

a) The mechanical properties including cube compressive strength, elastic modulus, split tensile strength, and flexural strength of the concrete shall be tested following the method in accepted standard [C-7(34)]. The properties of water permeability, drying shrinkage, and rapid chloride permeability shall also be tested following the relevant Indian standards. The tests shall be performed on the specimens sawn/cored from printed samples. The anisotropy of the structure shall be tested along X, Y and Z coordinates. The test shall be conducted on the cast specimen also with the concrete used for printing the structure; and

b) Additionally, the bond strength of the layer shall be tested by extracting beam specimens of 40 mm × 40 mm × 160 mm from the printed structure, or a special test element. 3-point and 4-point bending test shall be performed on the extracted specimens with a minimum number of 5 specimens for each sample.

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ANNEX C

(*Foreword* and Clause 6.5.2)

PREFABRICATED PREFINISHED VOLUMETRIC CONSTRUCTION (PPVC)

C-1 GENERAL

This annex provides a comprehensive guideline on prefabricated prefinished volumetric construction (PPVC) methods, materials, design standards and performance criteria for use by building professionals and other stakeholders involved in modular construction. This annex aims to facilitate the understanding and practical application of PPVC across various types of construction projects, including residential, commercial and industrial buildings. By outlining the key requirements and standards, this annex supports efficient and effective PPVC project planning, production, and assembly of PPVC to ensure high-quality, durable and sustainable modular structures.

This annex provides insights into essential aspects of prefabricated prefinished volumetric construction (PPVC), including design principles, manufacturing and installation practices, quality assurance, transportation and site assembly. It also highlights critical considerations for structural integrity, fire safety, environmental sustainability and inspection protocols, thereby promoting the adoption of best practices and supporting ongoing efforts to advance safe and innovative construction methods.

C-2 METHODOLOGY

The following points are considered while defining the application and performance guidelines.

C-2.1 Design

Design of PPVC modules considers both architectural and structural planning, taking into account modularization, alignment, and standardization of dimensions. It ensures that the modules comply with specifications for efficient on-site assembly, structural integrity and adherence to applicable building standards and design requirements.

C-2.2 Off-Site Manufacturing

The modules should be constructed in a controlled factory environment. This allows precise manufacturing with minimal errors and faster production times. Components, including concrete and steel modules, are cast, cured and finished with necessary fixtures, plumbing, and electrical setups before transportation to the construction site.

C-2.3 Transportation

Once manufactured, modules are carefully transported to the construction site. The transportation plan ensures that modules remain damaged and conform to transportation regulations, accounting for dimensions, weight and route restrictions.

C-2.4 On-Site Assembly

This stage involves the installation and connection of PPVC modules at the construction site. Modules are hoisted and secured in position, followed by finishing operations, including joint sealing and final inspections, to ensure that the assembled structure complies with all applicable quality and safety standards.

C-2.5 Modular Dimensions

PPVC modules should designed to fit within standard transportation limits to ensure easy delivery to construction sites. Common dimensions range from 3 m to 12 m in length, 2.5 m to 3.5 m in width, and 2.5 m to 4.5 m in height. Specific adaptations shall account for the maximum width (2.5 m), height (4.5 m including truck height), and length (up to 18 m) based on transportation guideline.

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C-2.6 Transportability and Site Constraints

The size and weight of PPVC modules are influenced by on-site access, route clearance and crane requirements. The structure shall be modularized to ensure efficient on-site alignment and joining of modules, while minimizing the need for additional handling at the construction site.

C-2.7 Materials

a) *Concrete and steel modules* — PPVC modules are typically constructed from high-performance materials like reinforced concrete (RC) or steel. Reinforced concrete offers durability and structural integrity, whereas steel provides a lighter alternative, especially useful for high-rise buildings; and

b) *Enhanced material specifications* — Materials are chosen for performance under factory conditions and transportation stresses. For instance, high-strength/performance concrete (HPC) and steel conforming to relevant Indian Standard (such as Fe 500 or higher for reinforcement) are preferred. The use of sustainable materials, such as fly ash and other admixtures, is also encouraged to improve durability and reduce environmental impact.

C-2.8 Connections

Connections should be strong enough to secure modules under various stresses, including lifting, transportation, and environmental forces. Common PPVC connection methods include grouted couplers, wire loop boxes, dowel systems, and mechanical fasteners. These ensure that modules are securely integrated without compromising flexibility needed for seismic or wind forces.

Vertical joints (for stacking modules) and horizontal connections (for lateral stability) are essential for PPVC structures. These connections often involve interlocking systems or bolted end plates that provide stability while allowing for some movement to accommodate load and environmental stresses.

Connections should be designed to follow the Indian Standards for seismic and wind loads, such as good practice [C-7(35)] and **5** of Part C/Section 1 of this NBCS, to ensure safety and resilience under local conditions. Proper connection systems are necessary to transfer loads between modules and secure them as a unified structure.

C-2.9 Alignment

The alignment of prefabricated prefinished volumetric construction (PPVC) modules is critical to ensure that the final building is structurally sound and aesthetically pleasing. There are several key considerations to keep in mind when aligning PPVC modules:

a) The foundation on which the PPVC modules are placed should be level and square. If the foundation is not level, it can cause the modules to shift and potentially compromise the structural integrity of the building. Unless it is designed as a paper joint, a minimum 20 mm joint thickness shall be detailed between the top of the foundation and the PPVC module to account for site tolerances. The joint shall be later grouted with high strength grout as per design specification for structural integrity as well as continuity for functional performance;

b) Each PPVC module should be precisely aligned with the adjacent modules to ensure that the building is straight and true. This is typically achieved using alignment pins or other methods to ensure that the modules are aligned both vertically and horizontally. The erection tolerances shall be followed as per **10.7.6**;

c) Ensuring that the PPVC modules are level vertically is also critical to the structural integrity of the building. Laser levels and other tools can be used to ensure that the modules are plumb and aligned with the adjacent modules; and

d) The facade of the building, which includes windows, doors, and exterior finishes, should also be precisely aligned with the PPVC modules to ensure that the final building looks aesthetically pleasing.

C-2.10 Structural Design and Load Compliance

PPVC modules should comply with load standards as outlined in Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS, for addressing dead loads, live loads, wind loads and other special loads requirements.

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Modules should also meet seismic design requirements under **5** of Part C/Section 1 of this NBCS to ensure stability during earthquakes.

Structural elements for concrete and steel structures shall follow Part C 'Structural Design, Section 5 Structural Concrete' and Part C 'Structural Design, Section 6A Steel' of this NBCS, ensuring that all modules meet load-bearing and durability criteria. The Limit State Method is typically applied for PPVC modules, which includes safety factors to manage risks associated with live loads, dead loads and extreme environmental forces.

C-2.11 Material Standards

a) *Concrete and reinforcement compliance* — For concrete quality, PPVC construction shall follow Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS, including high-performance concrete or self-compacting concrete for durability. Reinforcements use deformed steel bars as per accepted standard [C-7(36)] for high-strength requirements, ensuring modules can withstand handling, transportation and on-site stresses; and

b) *Admixtures and aggregates* — Chemical and mineral admixtures should comply with accepted standards [C-7(5)] and Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS, which specify performance standards for durability and workability. Aggregates are selected as per accepted standard [C-7(4)], ensuring strength and proper gradation.

C-2.12 Fire Safety and Compartmentation

For fire resistance, PPVC structures shall comply with the requirements specified in Part F 'Fire and Life Safety' of this NBCS, including provisions for compartmentation. Each PPVC unit shall be provided with fire-rated walls, floors and doors, as required, to restrict the spread of fire and smoke and to ensure adequate means of egress and safety of occupants.

All materials used shall meet the relevant requirements of Part F 'Fire and Life Safety' of this NBCS for fire safety. Structural elements shall be designed to withstand high temperatures and achieve the required fire resistance in accordance with the relevant provisions of Part C/Section 5 of this NBCS.

C-2.13 MEP (Mechanical, Electrical, Plumbing) Standards

a) Electrical installations in PPVC buildings should meet the accepted standard [C-7(30)] and Part D 'Building Services/Section 2 Electrical and Allied Installations' of this NBCS, ensuring safe and reliable connections between modules. Additionally, accepted standard [C-7(38)] and Part D/Section 2 of this NBCS should be used for lightning protection systems to prevent electrical hazards; and

b) Plumbing systems follow good practice [C-7(39)] and Part E 'Plumbing Services, Section 1 Water Supply and Section 2 Drainage and Sanitation' for water supply and drainage, ensuring leak-proof, pressure-resistant pipes that meet durability and sanitation requirements in modular installations.

CI-2.14 Construction Tolerances

Construction tolerance limits in prefabricated prefinished volumetric construction (PPVC) refer to the allowable deviation from the intended design and specified dimensions. These limits are important to ensure that the final building is structurally sound and aesthetically pleasing. The construction tolerance limit in PPVC can vary depending on the specific materials and construction techniques used, as well as the local building standards and regulations. Permissible tolerances from the specified dimension of a PPVC module should conform to casting tolerances provided in **10.7.5**.

C-2.15 Performance Criteria

The performance criteria of any PPVC components should conform to the relevant Indian Standard for equivalent components. In the absence of any Indian Standard, the safety and durability of the specific element or component should be demonstrated through testing.

C-2.16 Special Applications

In the absence of any standards, the engineer-in-charge should make decisions based on logical reasoning and practical evidence. The test program should be conducted at accredited/approved laboratories and approved by statutory bodies.

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C-3 APPLICATIONS

C-3.1 Residential and Commercial Buildings

PPVC modules for residential and commercial buildings shall be permitted for use in multi-storey structures and may be designed as either load-bearing or non-load-bearing units. Load-bearing modules shall be designed to resist vertical loads and shall satisfy requirements of stability and safety for multi-level configurations. Non-load-bearing modules shall be used as partitions or infill components. The provisions of Part F 'Fire and Life Safety' of this NBCS shall apply for fire safety requirements, while Part C 'Structural Design, Section 4 Masonry' of this NBCS shall govern structural requirements, sound insulation and seismic load-bearing capacity. The design shall ensure adequate connections and overall structural integrity of the system.

C-3.2 Educational and Healthcare Facilities

Modules are pre-finished to meet spatial needs, such as classrooms and patient areas, integrating critical systems like mechanical, electrical and plumbing (MEP) as per good practice [C-7(40)] and Part E 'Plumbing Services, Section 1 Water Supply and Section 2 Drainage and Sanitation' for water supply and drainage.

C-3.3 Industrial Buildings

PPVC in industrial buildings such as warehouses and factories requires modules engineered to meet the load-bearing and lateral resistance standards outlined in Part C 'Structural Design, Section 6A Steel' of this NBCS for structural steel and Part C 'Structural Design, Section 5 Structural Concrete' of this NBCS for reinforced concrete. The modular units allow for open, high-capacity spaces that can bear substantial loads, with compliance to Part C 'Structural Design, Section 1 Load and Load Effects' of this NBCS for design loads, wind, snow loads and for seismic considerations. Structural configurations shall also align with fire safety standards as per good practice [C-7(19)] to facilitate efficient integration of services. This approach enables high-speed assembly, structural integrity and durability for industrial purposes while meeting all requirements.

C-3.4 Design Philosophy

PPVC structures should be designed to meet strength and serviceability requirements, ensuring structural integrity under the anticipated loads. This focuses on load-bearing applications within PPVC, excluding non-structural or purely aesthetic uses. Additionally, the design should account for durability and long-term performance, particularly as modules are prefabricated off-site and subjected to transportation and on-site assembly stresses.

Load-bearing PPVC modules typically consist of a reinforced concrete or steel core surrounded by a prefinished volumetric structure. The core provides essential stability, especially under vertical and lateral loads. In certain applications, the volumetric shell itself may be designed to carry both in-plane and lateral loads, with the core providing additional structural redundancy.

The design approach for load-bearing PPVC modules depends on the load resistance mechanism employed, as outlined below:

a) *Core-based load resistance* — Where the primary load resistance is provided by a reinforced concrete or steel core, which may be cast in-situ or prefabricated; and

b) *Monolithic module design* — Where the PPVC module's core and shell use a compatible concrete mix or reinforcement design to behave as a single monolithic unit under load.

For cases where the load resistance is derived from the combined action of the volumetric shell and the core, individual component testing is recommended to validate the composite action. Efforts to promote ductile behaviour are essential, ensuring that load-bearing PPVC elements respond safely under stress.

NOTE — Testing and certification can be carried out as per specialised literature (reference may be made to applicable standards/Codes of the International Code Council (ICC) for structural evaluation methods), or structures may be certified through recognized government bodies in India. The scope of this section is limited to gravity loads. For resistance to earthquake-induced forces and ductile detailing as per Part C/Section 1 and Part C/Section 5 of this NBCS, additional testing and validation may be required.

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C-3.5 Loads

Load criteria for PPVC modules should conform to good practices, with particular attention to lateral and seismic loads. Structural components should be validated through full-scale testing of individual elements, ensuring that boundary conditions simulate actual applications. Testing should confirm that PPVC modules meet all load-bearing and serviceability requirements, as specified accepted standards, for safe deployment in permanent structures.

C-3.6 Foundations

A PPVC foundation should be designed following a design-by-testing approach to confirm compliance with all load-bearing and durability standards. The foundation may be cast in-situ or prefabricated off-site as a precast unit for on-site assembly. Design shall be overseen by a certified structural engineer, with essential geotechnical input to evaluate soil bearing capacity, ensuring that the foundation is appropriate for the specific site conditions. Durability requirements per 'Structural Design, Section 5 Structural Concrete' of this NBCS should be strictly observed to maintain long-term performance and stability under applied loads.

To ensure a secure connection between the PPVC modules and the foundation, appropriate dowel bars or reinforcement should be incorporated into the foundation design. These reinforcement provisions should align with the load transfer requirements and provide structural continuity between the foundation and the modular components. Proper anchorage and placement of dowels will prevent differential movement and provide stability for the assembled structure.

C-4 CURING

As PPVC modules are manufactured in an off-site, controlled environment, advanced curing methods may be implemented to expedite the process and ensure consistent quality. In addition to following 'Structural Design, Section 5 Structural Concrete' of this NBCS, rapid curing techniques, such as steam curing, warm water curing, or the use of curing compounds, can be employed to accelerate strength development. These methods enable the modules to achieve the desired performance levels within a shorter time frame, which is critical for modular construction timelines.

The curing approach should ensure that modules reach their designed strength before transportation to the construction site. The decision on curing methods and duration should be based on testing and performance requirements, as determined by the supervising engineer, to guarantee durability and structural integrity.

C-5 QUALITY ASSURANCE AND QUALITY CONTROL

Compliance with good practices includes ongoing quality checks at both the manufacturing and construction stages. Regular inspections, as per relevant provisions this NBCS and relevant Indian Standards, to verify structural integrity, alignment and safety of each module, especially focusing on connections and material durability over time.

C-6 MAINTENANCE AND INSPECTION

Regular inspection and maintenance protocols should be set to identify wear or structural concerns in connections and external finishes, ensuring longevity and safety as outlined in relevant provision of this NBCS.

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ANNEX D

(*Foreword* and *Clause* 6.6)

GUIDELINES FOR DESIGN FOR MANUFACTURING AND ASSEMBLY (DFMA)

D-1 The concept of design for manufacturing and assembly (DFMA) emphasizes the use of precast and prefabricated building components. The underlying principles of DFMA call for more efficient building design and construction detailing, especially in the area of precast concrete connection designs, so as to minimize the amount of wet works on-site.

To develop an advanced pre-cast concrete system (APCS), it is important for the various building disciplines to first understand the functional considerations, building aesthetics, and design approaches. This understanding will enable the conceptualization and development of the APCS design, ensuring it meets the project specifications. It is essential that the APCS design is coordinated with the manufacturing specification for components, construction planning and site work to ensure seamless integration and efficient construction processes.

D-2 DESIGN BASIS FOR ADVANCED PRECAST CONCRETE

The following are key principles of efficient construction using APCS systems:

a) *Modular architecture layouts* — Designing buildings in modules to allow for greater use of precast and prefabricated components, reducing on-site construction time and costs;

b) *Precast repetition* — Using standard precast elements repeatedly throughout the building, simplifying the construction process and reducing the need for customization;

c) *Efficient structural design* detailing (standardize design and detailing) — Designing precast connections that can be easily assembled on-site, reducing the need for wet works and temporary supports;

d) *Quick assembly detailing* — Designing connections and components that can be quickly and easily assembled on-site, reducing construction time and costs; and

e) *Limited use of temporary supports* — Designing precast components that can be assembled without the need for temporary supports, further reducing construction time and costs.

D-3 FACADE STANDARDIZATION

Architectural design is a process to create a built environment with functionality for human comfort and sustainability through art and engineering design. In an effective precast concrete construction, the external facade design is generally achieved through optimal standardization in the following:

a) *Shapes* — FRP/rubber-lined moulds can be used to create various shapes for precast concrete components;

b) Textures such as sandblasted or acid-etched finishes can also be added to enhance the visual appearance of the precast components;

c) Different colours can be achieved using pigments or concrete colours, allowing for a wide range of design options;

d) *Repetition* — Standard precast elements can be repeated throughout a building's facade, making construction faster and more efficient;

e) *Modular design* — It allows for the precast components to be easily assembled on-site, reducing the need for wet works and temporary supports;

f) *Small complex facade panels* — Glass reinforced concrete (GRC) can be used instead of concrete for small complex facade panels;

g) *Complex facade as a separate add-on* — Complex facade elements can be designed and manufactured separately from the building structure, allowing for easier handling and installation on-site;

h) *Layouts rotated over mirrored* — Rotating layouts over mirrored elements can make use of the same assembly for construction; and

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j) *Checklists* — Checklists can be used to ensure good precast arch design, covering aspects such as structural integrity, aesthetic appeal and ease of construction.

D-4 STRUCTURAL DESIGN

Structural design has a direct impact on the ease of precast concrete manufacturing and site erection works. The key consideration for structural system design solutions are:

a) *Simple design and detailing* — Precast concrete components should be designed with simple connections that can be easily assembled on-site, reducing construction time and costs;

b) *Commercial - long spans* — Precast concrete components can be used for long-span commercial buildings, such as warehouses or shopping centres;

c) *High rise residential* — In high-rise residential buildings, precast concrete wall panels can be used in conjunction with precast concrete floor slabs;

d) *Low/mid rise residential* — In low to mid-rise residential buildings, precast concrete wall panels can be used with precast concrete floor slabs and beam-column combinations; and

e) *Design to cater for temporary loads during construction* — The structural design should cater for temporary loads during the construction phase, such as the weight of equipment and construction workers.

D-5 TYPES OF STRUCTURAL SYSTEMS

All semi or full precast concrete structural systems need clear consideration for stability. It may take one of the following forms:

a) *Skeletal frames* — Precast concrete components can be used to create skeletal frames, with columns and beams connected with precast framed connections;

b) *Load bearing walls* — Precast concrete wall panels can be used as load-bearing walls, supporting the weight of the floors and roof;

c) *Load bearing facade* — In a load-bearing facade system, the precast facade concrete panels bear the weight of the building and transfer the loads to the foundation independently to the core of the structure;

d) 3-*D boxes* — Precast concrete components can be used to create 3-D boxes, with precast walls, floors, and roofs forming the 3-D building unit;

e) *Mixed systems* — A combination of precast concrete components can be used to create mixed systems, such as precast concrete walls with cast-in-place concrete floors; and

f) *Checklists for good precast structural design* — Checklists can be used to ensure good precast structural design, covering aspects such as strength and durability, fire resistance, seismic design and ease of construction.

D-6 CONNECTIONS

Connection in precast concrete construction must be designed to take the forces derived in the structural modelling. To achieve work efficiency, the design approach should consider the following aspects:

a) *Standardized connections* — Standardized precast concrete connections are designed to be easily designed, manufactured and assembled on-site;

b) *Ease of erection* — Precast concrete connections should be designed with ease of erection in mind, to ensure that they can be quickly and safely assembled on-site;

c) *Simple and minimalistic* — Simple and minimalistic connections are preferred in precast concrete construction, as they are more efficient and cost-effective;

d) *Checklists for good connection systems* — Checklists can be used to ensure good precast concrete connection systems, covering aspects such as durability, strength, and ease of assembly; and

e) *Innovative connections* — Innovative precast concrete connections can be used to create unique and complex structures, such as hybrid steel and precast systems or lotus root precast systems. These connections can offer advantages such as improved structural performance, and reduced construction time.

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D-7 DESIGN OF CONNECTIONS

Design methods for joints in precast construction have been covered by various international technical journals and published academic research papers. They briefly cover the connections listed below:

a) *Corbel-beam end joint* — A hidden mechanical connector can be used for the corbel-beam end joint, similar to hanger connections;

b) *Column/wall base mechanical connections* — Column shoes and wall shoes with anchor bolts can be used for mechanical connections at the base of columns and walls;

c) *Wall to wall connections* — Wire loops can be used for wall to wall connections;

d) *Beam to beam* — Steel hangers can be used for connecting precast concrete beams; and

e) *Rigid beam-column* — Top steel couplers and bottom steel welded connections can be used for rigid beam-column connections, ensuring that the connection is strong and can resist bending and shear forces.

D-8 CONNECTION DESIGN CONCEPT

This can cover the following:

a) Load paths — Diaphragm, slab to wall/column paths, slab-beam shear connection;

b) Frame behaviours;

c) Accidental loads, progressive collapse;

d) Tie design forces;

e) Tie continuity — Laps, tie with enclosed links; and

f) Tie anchorages with other ties.

D-9 DESIGN EXAMPLES

a) Beam half joint (dap connection);

b) Beam half joint (hanger/beam shoe connection);

c) Corbel (conventional);

d) Peikko corbel;

e) Column base — Splice sleeve/grouted couplers;

f) Column base — Shoe connection;

g) Wall base — Splice sleeve/grouted couplers;

h) Wall base — Shoe connection;

j) Wall —Wall joint – Shear keys only;

k) Wall — Wall joint – With loop reinforcement;

m) Beam steel hangers — Generic;

n) Beam steel hangers — Peikko;

p) Proprietary mechanical connections;

q) Column to base and column to column;

r) Beam to column — Moment connection;

s) Beam to column — Simply supported;

t) Beam to beam;

u) Wall to wall — Vertical and horizontal connections; and

v) Landing slabs — TSS.

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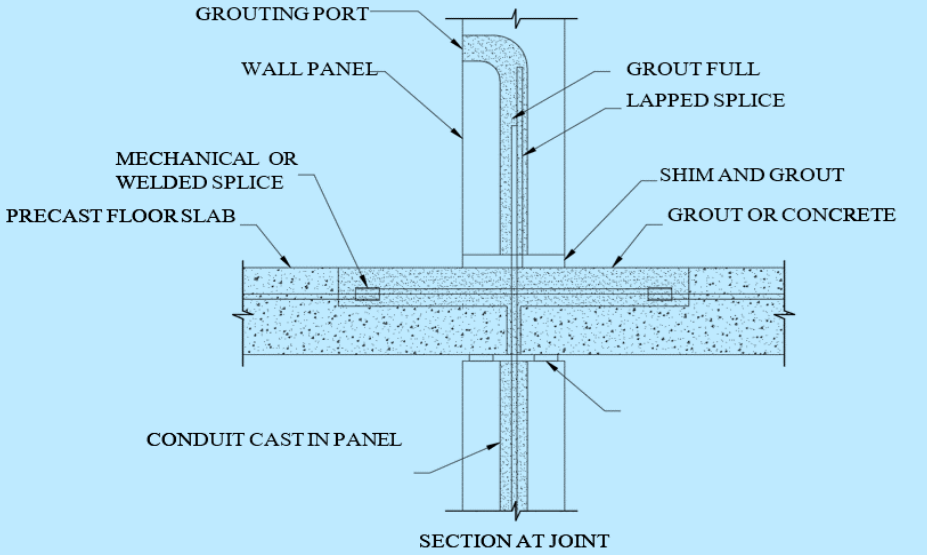
ANNEX E

(*Foreword* and *Clause* 8.3)

COMMON PRECAST CONNECTIONS

E-1 Typical figures of some common precast joint connections are given in Fig. 44 to Fig. 56.



NOTE: Overlapping bars in grout-filled conduit are extended full-height through the structural element

FIG. 44 LAPPED SPLICES IN LARGE CONDUIT

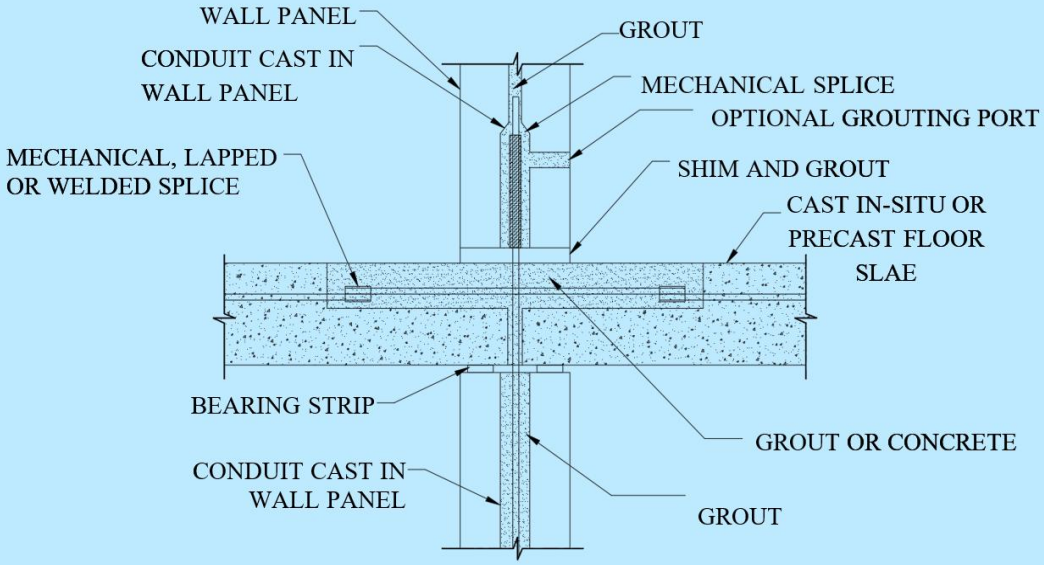


FIG. 45 MECHANICAL CONNECTION IN CONDUIT

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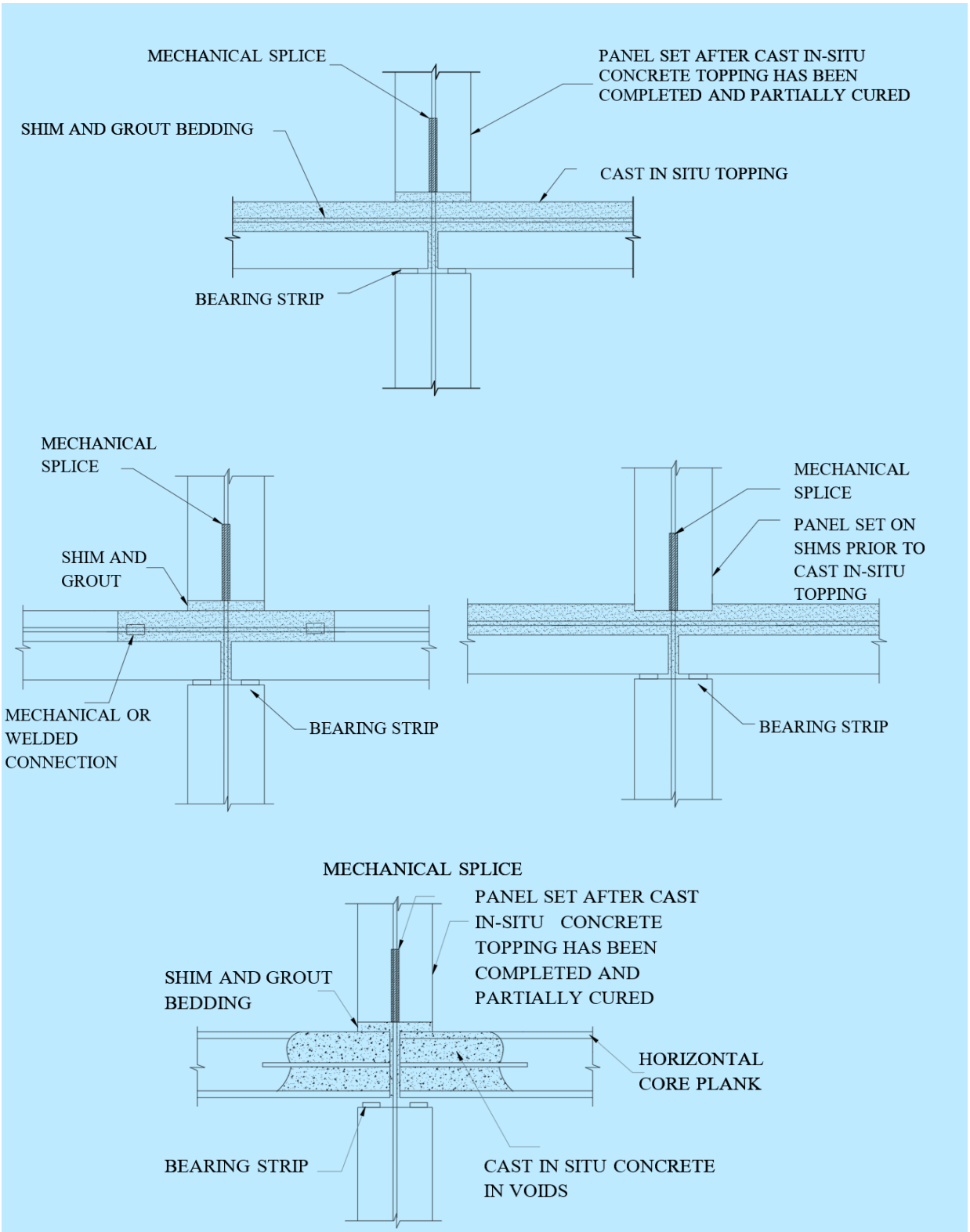

FIG. 46 DIFFERENT TYPES OF MECHANICAL SPLICES FOR CONNECTION OF VARIOUS CONFIGURATIONS OF PRECAST WALL AND FLOORS

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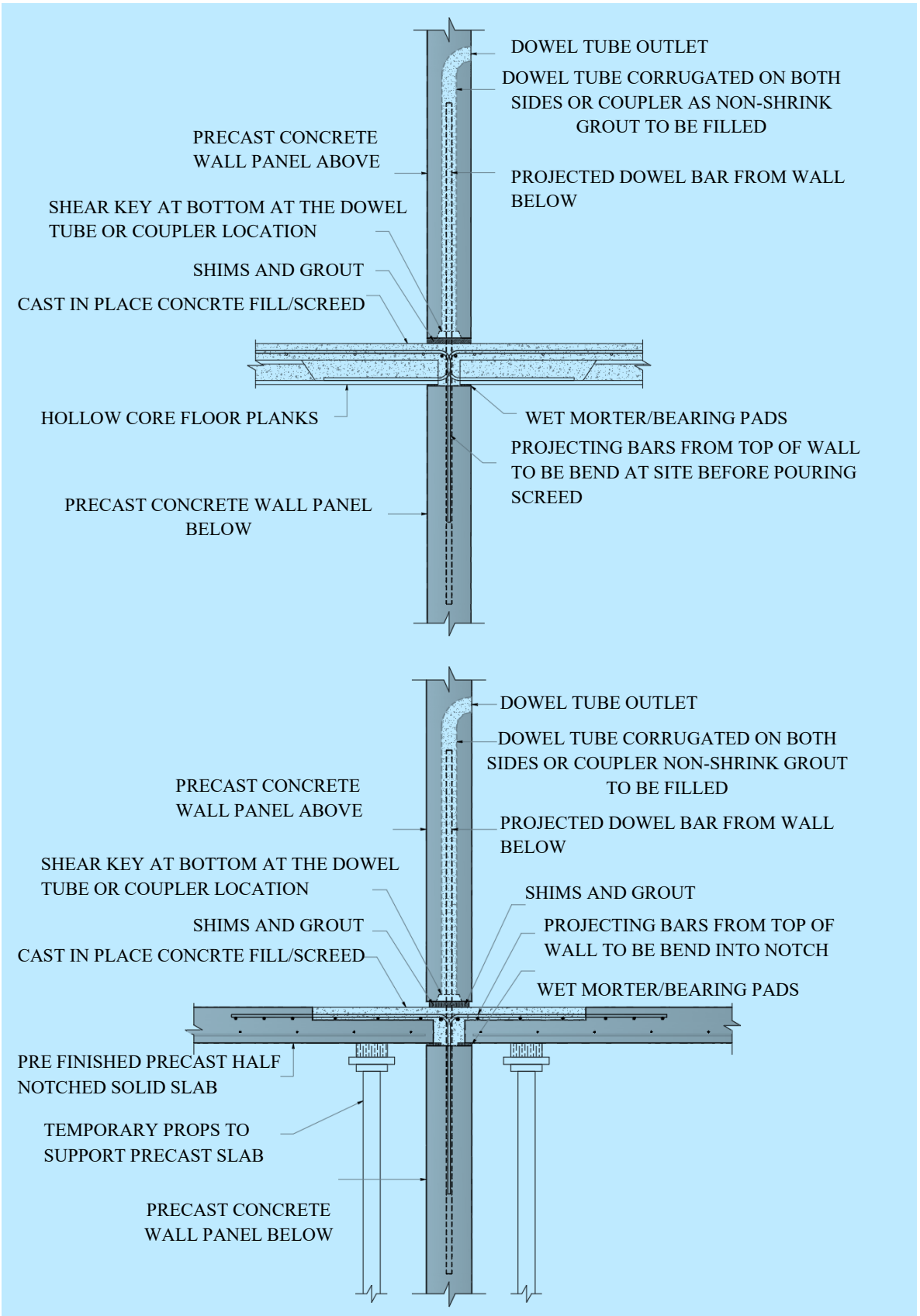

FIG. 47 MECHANICAL SPLICES USING HIGH STRENGTH NON-SHRINK NON-METALLIC GROUT (CONTINUED)

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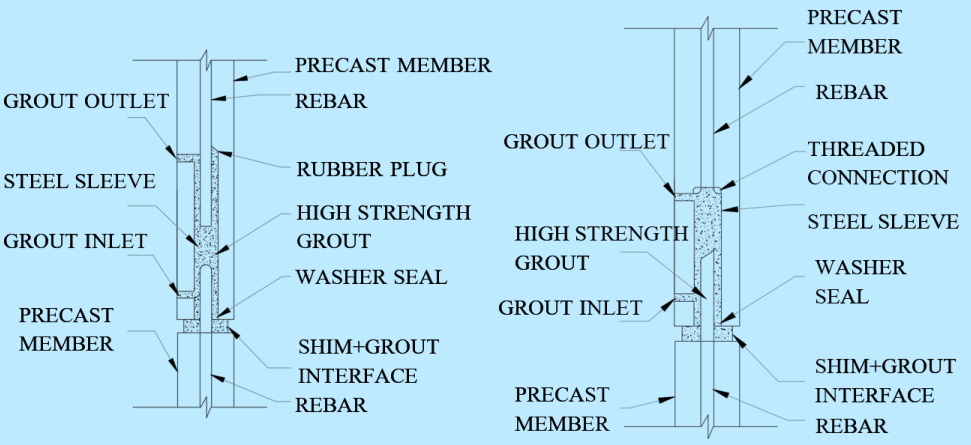

FIG. 47 MECHANICAL SPLICES USING HIGH STRENGTH NON-SHRINK NON-METALLIC GROUT

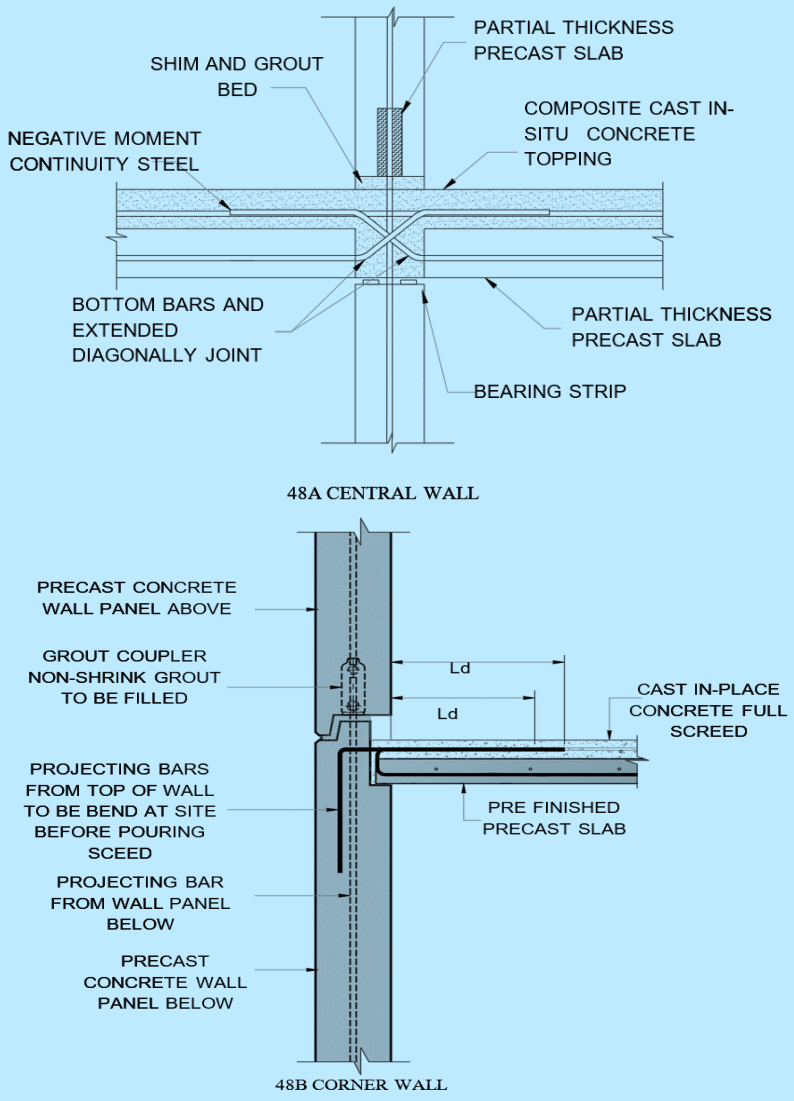

FIG. 48 FLOOR SLAB TO WALL DETAILS

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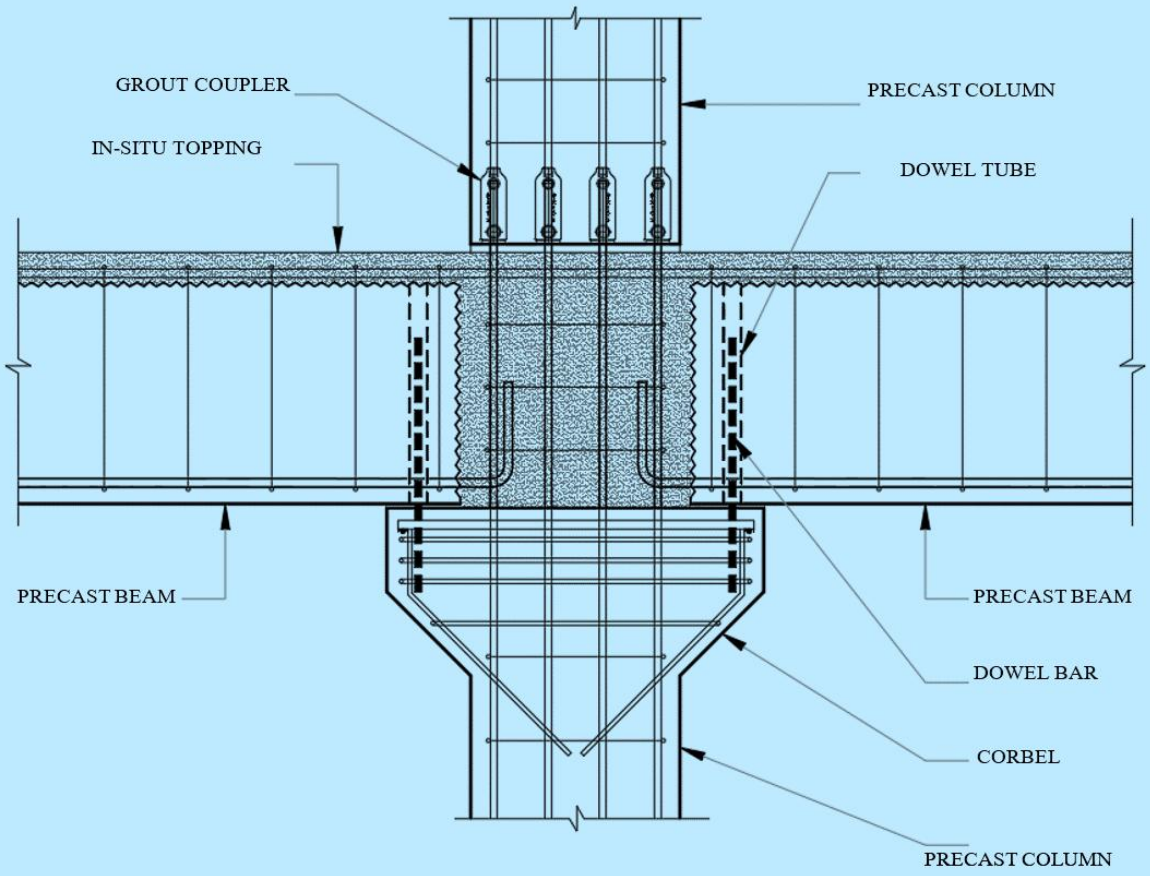
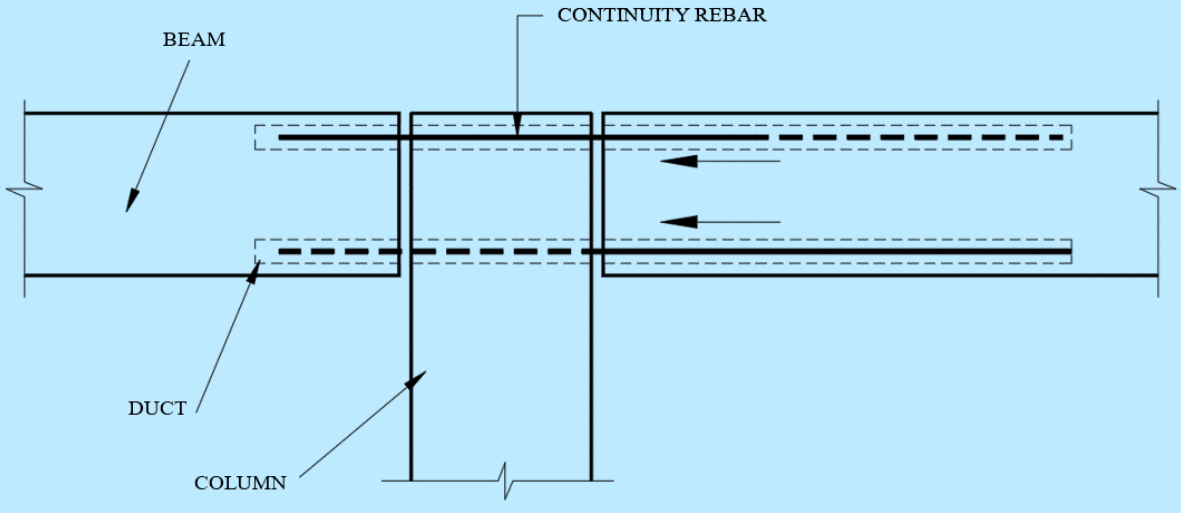


49A WITH CIS JOINT



49B WITH GROUTED JOINT

FIG. 49 BEAM AND COLUMN

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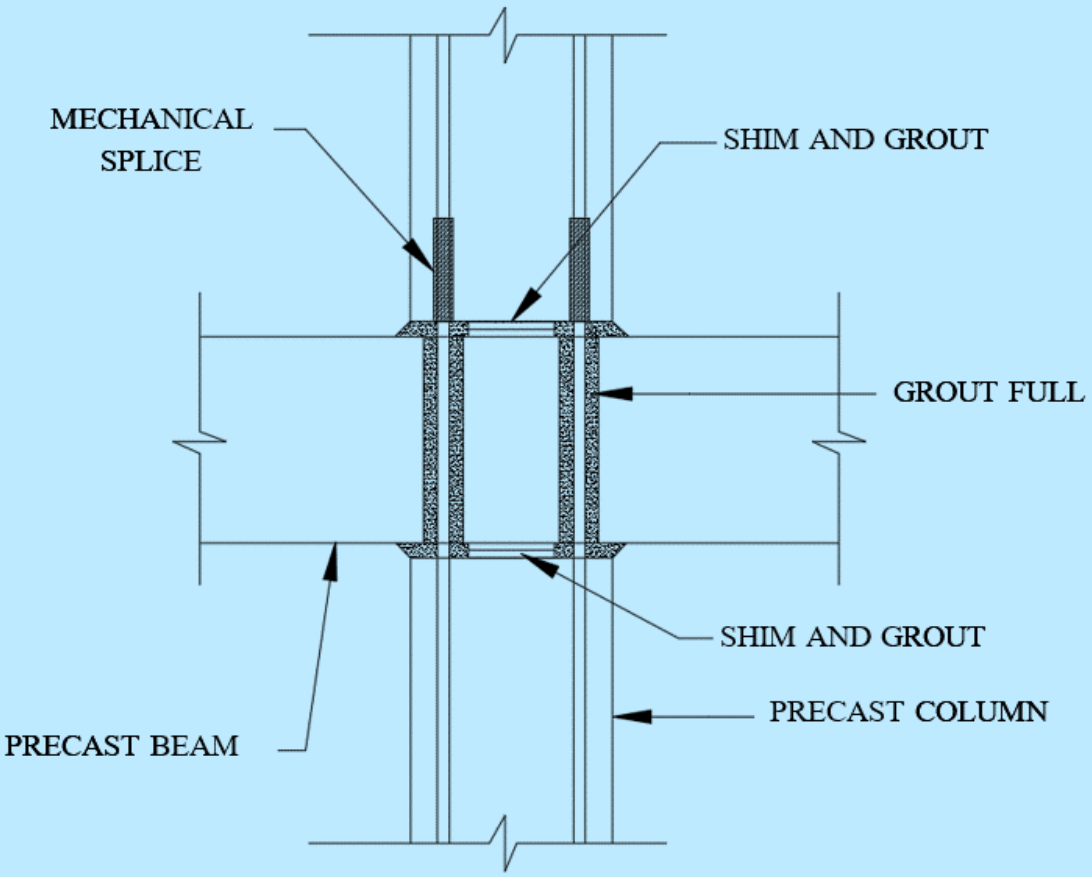

NOTE: Conduit diameter should be two to four times the bar diameter for tolerance in field erection

FIG. 50 COLUMN TO COLUMN THROUGH CONDUIT IN BEAM

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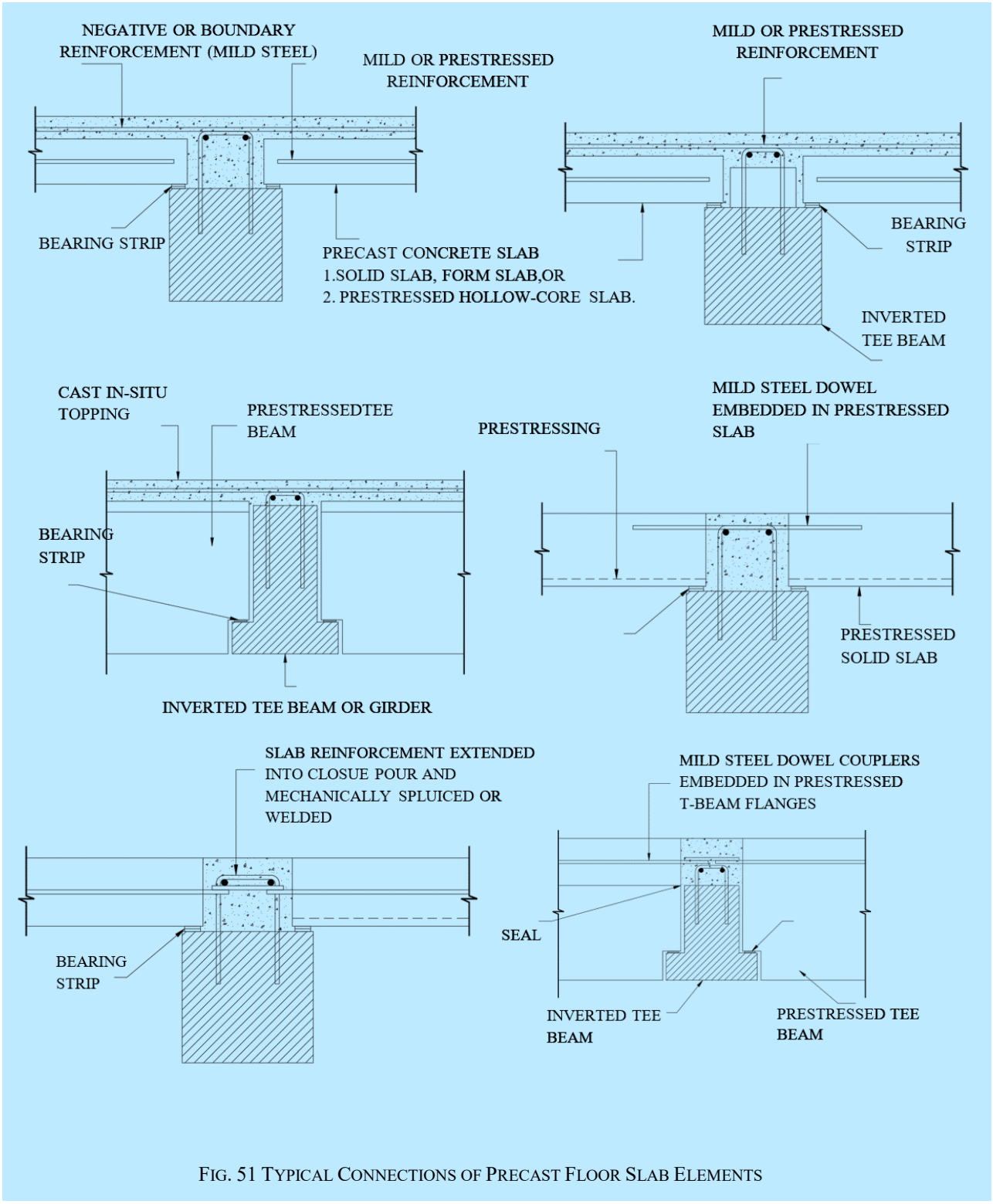

FIG. 51 TYPICAL CONNECTIONS OF PRECAST FLOOR SLAB ELEMENTS

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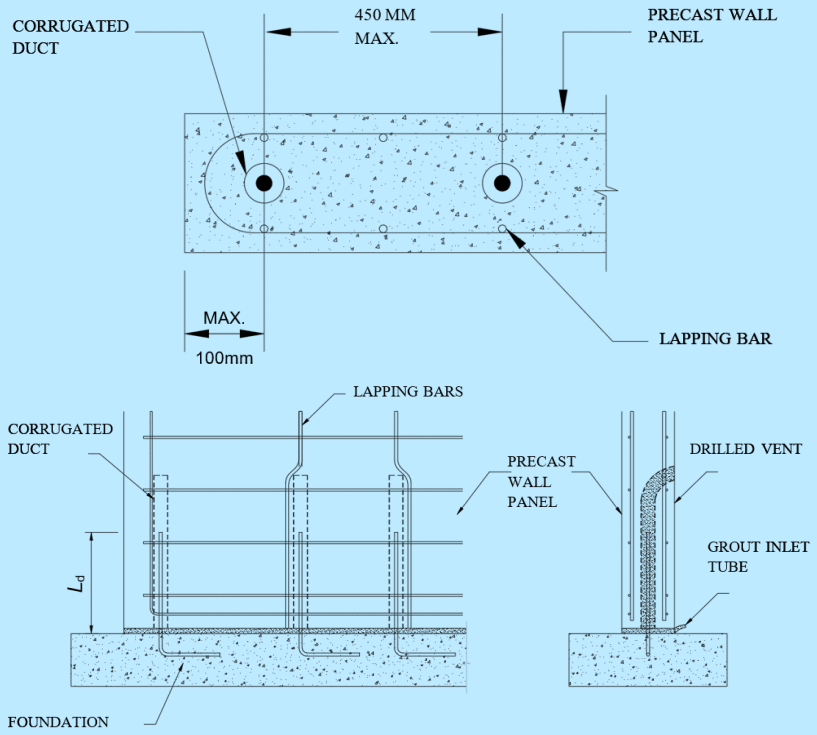

FIG. 52 WALL TO FOUNDATION CONNECTION THROUGH GROUTED DUCT

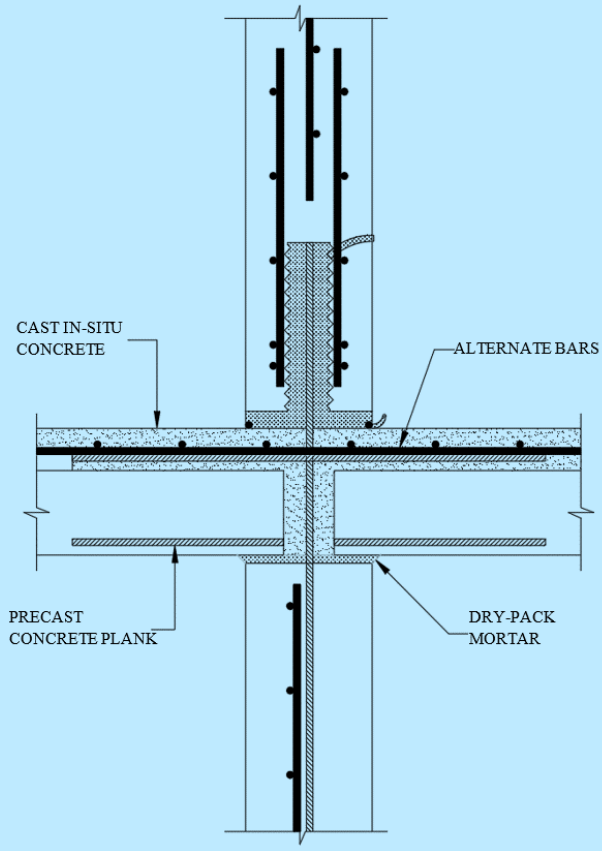

FIG. 53 WALL TO FLOOR SLAB CONNECTION

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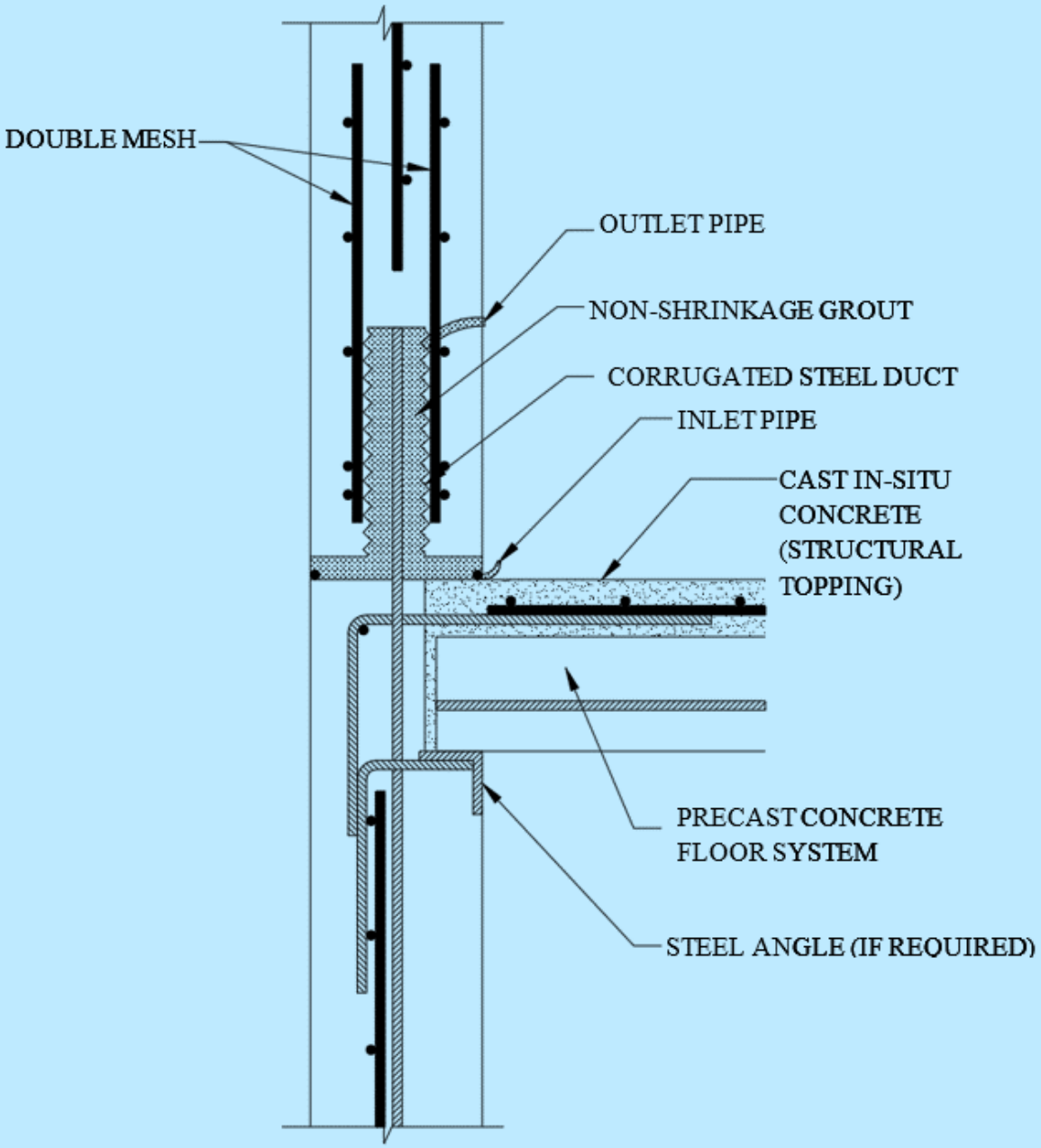

FIG. 54 EXTERIOR WALL TO FLOOR SLAB CONNECTION

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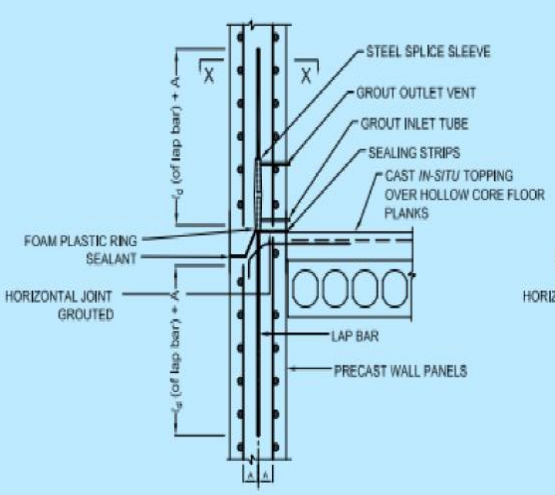
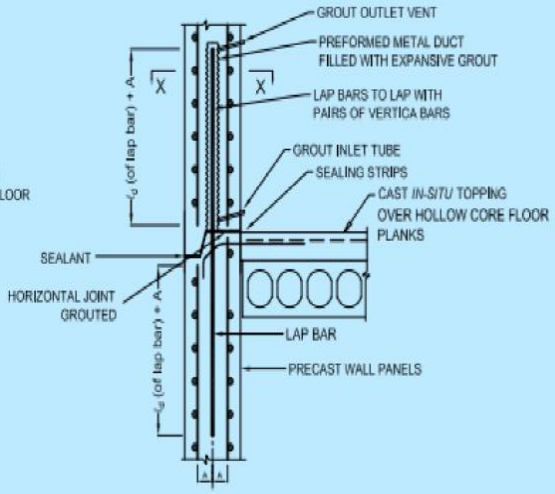
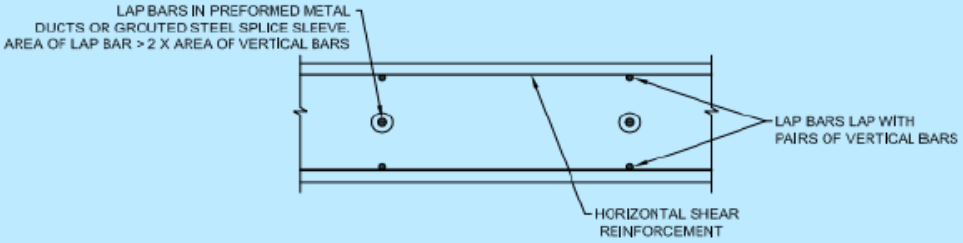


55A HORIZONTAL JOINT – TYPE A

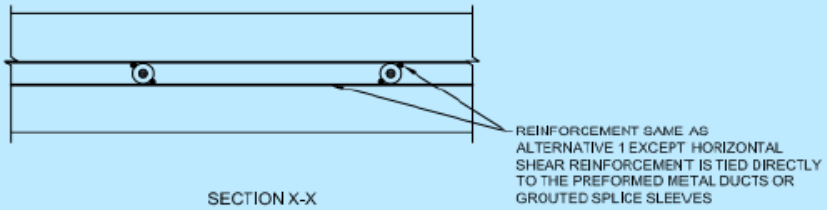


55B HORIZONTAL JOINT – TYPE B



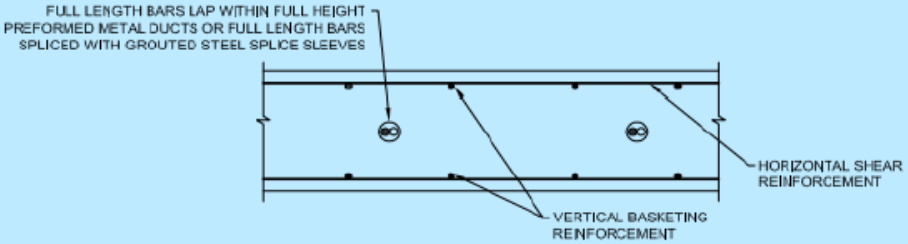
SECTION X-X

ALTERNATIVE 1



SECTION X-X

ALTERNATIVE 2



SECTION X-X

ALTERNATIVE 3

FIG. 55 MONOLITHIC PRECAST WALL CONSTRUCTION

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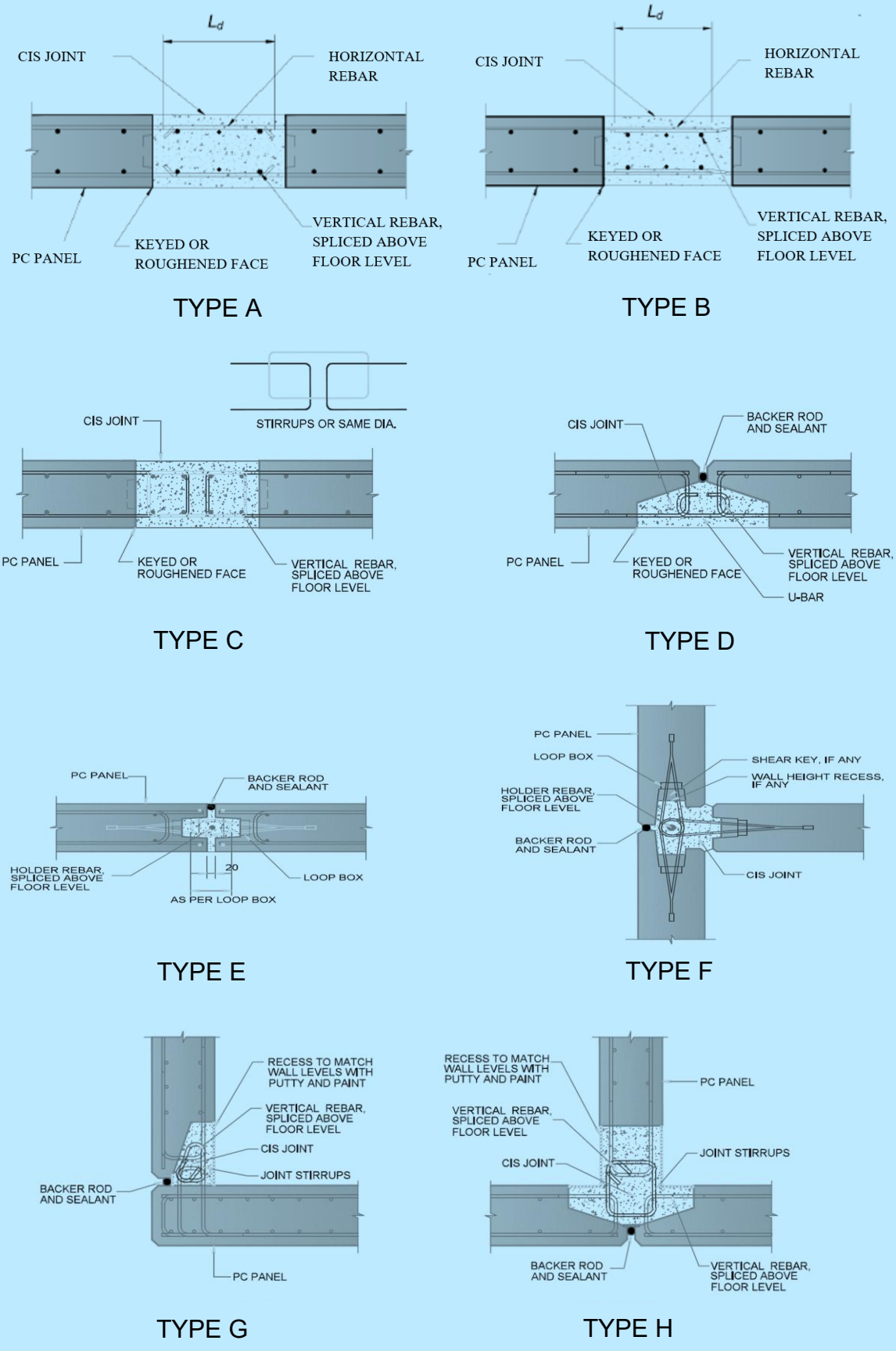

FIG. 56 VERTICAL JOINT BETWEEN WALL TO WALL OR WALL TO COLUMN (CONTINUED)

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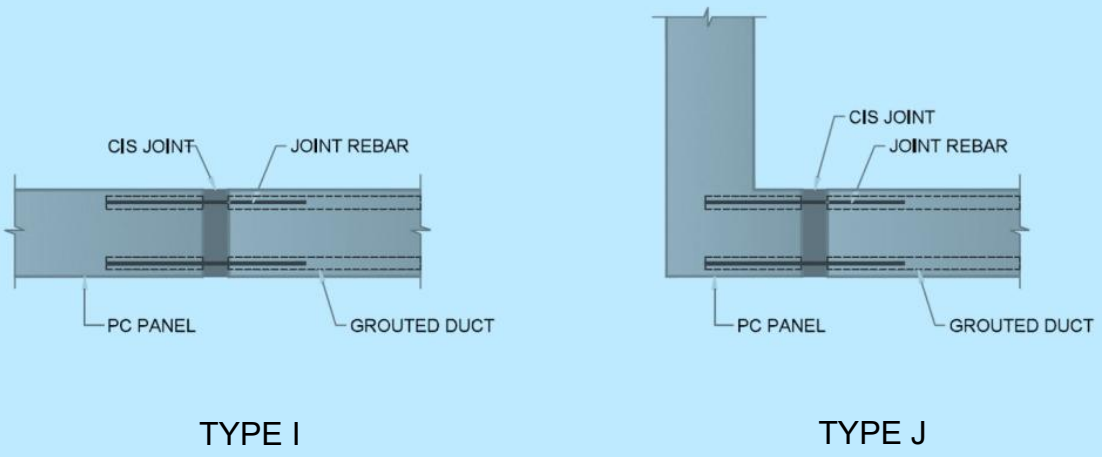

FIG. 56 VERTICAL JOINT BETWEEN WALL TO WALL OR WALL TO COLUMN

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ANNEX F

(*Clause* 11)

COMMON DEFECTS AND REMEDIES

F-1 As defects in precast concrete elements result in direct and indirect cost in terms of rectification and construction time, it is worthwhile to ensure that they are produced and handled in a way to avoid/reduce such incidences. Table 19 illustrates some of these common defects, their causes and preventive and remedial measures.

Table 19 List of Common Defects and Recommended Remedial Measures

(*Clause* F-1)

Sl No.	Common Defects	Possible Causes	Recommendations	Remedial Measures
(1)	(2)	(3)	(4)	(5)
i)	*Dimensional deviation*: a) Variation in the dimension of precast elements would affect the joint alignment between these elements when erected; and b) Precast slab element may warp due to insufficient concrete strength at lifting or improper storage condition.	a) Mould forms may not be sufficiently rigid to maintain specified tolerances during concrete placement; b) Precast elements may not have gained sufficient concrete strength when demoulded; c) Top surface finish of precast elements may not be properly levelled and trowelled during production which result in differential thickness; and d) Precast elements (especially slender wall or slab panels) may be subjected to undue stress and deformation when they are not properly supported during storage.	a) Regular check on the dimensions and rigidity of mould forms before casting operations. As a general guide the recommended thickness for steel mould are: 1) 4.5 mm — Up to 50 castings; 2) 6 mm — Up to 100 castings; and 3) 9 mm — Up to 200 castings. Mould forms conditions will deteriorate with time and usage. They should be repaired, stiffened or replaced when needed. b) Cube tests should be conducted to ascertain the concrete strength of elements before demoulding;	a) For minor deviation, corrective measures such as surface grinding, trimming/ hacking and skim coat application can be appropriately used to remedy the situation; and b) Precast elements that are not within acceptable tolerance limits and have significant effects on the structural integrity or architectural performance should not be used.

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Table 19 (*Continued*)

Sl No.	Common Defects	Possible Causes	Recommendations	Remedial Measures
(1)	(2)	(3)	(4)	(5)
			c) Spreading and levelling of concrete placement using appropriate tools such as structural screeder; and d) Precast elements should be properly stored and stacked at designated points using suitable support spacers and frame rack system.	
ii)	Cracks	a) Precast elements may not have gained sufficient concrete strength before demoulding; b) Cracks may have occurred during initial lifting due to friction between the elements and the casting mould forms; c) Thickness of the precast elements may be too thin (70 mm or less) for safe demoulding and handling; and d) Cracks may have occurred during erection due to lack of planning and provisions for the precast panel geometry, crane rigging configuration and location of openings.	a) Proper curing method, curing time and temperature should be maintained; b) Cube tests should be conducted to ascertain the concrete strength of elements before demoulding; c) Appropriate form release agents should be used and uniformly applied onto the mould surface to minimize friction; d) Sectional thickness of precast elements should be increased to accommodate demoulding and handling stresses; e) Proper handling techniques should be adopted; f) Sufficient lifting points should be used to minimize over-stressing on certain areas;	a) All cracks should be examined by a qualified engineer to determine if they present a structural problem; b) Depending on the locations and seriousness of the cracks, different repair methods can be used to make good the affected precast elements; and c) Hairline cracks (not more than design crack width) can be repaired by cutting V-groove of specified minimum depth along the crack-lines, followed by patching subject to location and length of the crack in relation to length of units and not having adverse effect on the structural strength of the unit. For surface cracks (more than design crack width) or through cracks, epoxy injection method should be used

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Table 19 (*Continued*)

Sl No.	Common Defects	Possible Causes	Recommendations	Remedial Measures
(1)	(2)	(3)	(4)	(5)
			g) Additional reinforcing bars should be placed around the opening and odd corners; and h) Temporary stiffeners for openings should be provided during erection.	to ensure that the cracks are completely bound and filled with epoxy.
iii)	Chip-off and damages	a) Chip-off at panel edges are usually caused by the hard bearing at support or excessive force exerted on the elements when handling; and 4) Improper storage method.	a) Precautions should be taken to avoid damaging the elements during placement on vehicle, travel to site and during the unloading operation. Bearing pads should be used to cushion the contact areas from damage; b) The storage area should be relatively flat and dry; c) Precast elements should be properly stored and stacked at designated points using suitable support spacers and frame rack system; d) The casting slab or mould form should be thoroughly cleansed, levelled to achieve a smooth surface; e) The coverage of form release agents should be adequate and applied uniformly onto the mould surface;	a) Remove all loose concrete and wash off any dust or dirt in the affected area; b) Apply bonding agent to affected concrete surface; c) Welded wire mesh can be included to provide support for the patch mix concrete or grout; d) Patch mix composition or grout should be consistent with the strength requirements of the adjacent concrete; e) Formwork should be placed whenever it is necessary to contain the patch mix or grout; and f) The affected area should be protected from any disturbance during the curing period.

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Table 19 (*Continued*)

Sl No.	Common Defects	Possible Causes	Recommendations	Remedial Measures
(1)	(2)	(3)	(4)	(5)
			f) Spacers of the correct sizes should be used and well secured to maintain the concrete cover required during casting; g) Lifting inserts and concrete strength of elements should be of adequate capacity for the intended lift. Proprietary products for safe and efficient handling may be used for this purpose; and h) Lifting inserts or hooks should be inspected for proper location. They should be fastened to specified depth prior to concrete placement.	
iv)	Honeycomb and excessive pinholes	a) Poor concrete compaction due to ineffective vibration or rebars congestion; annd b) Grout leakage along the perimeter of side forms due to: 1) Loose or missing bolts, fixing pins; 2) Damaged rubber gasket seal; 3) Mould part, if defective.	a) Proper compaction methods should be adopted and carried out; b) Concrete mix design and workability should bereviewed and adjusted when needed; c) Appropriate concrete vibrator such as clamp-on form vibrator may be used to attain better compaction; d) Rebar congestion can be alleviated by having larger size (that is, lesser number) rebars or by increasing the sectional	a) Remove all loose concrete and wash off any dust or dirt in the affected area; b) Apply bonding agent to affected concrete surface; c) Welded wire mesh may be included to provide support for the patch mix concrete or grout; d) Patch mix composition or grout should be consistent with the strength requirements of the adjacent concrete;

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Table 19 (*Continued*)

Sl No.	Common Defects	Possible Causes	Recommendations	Remedial Measures
(1)	(2)	(3)	(4)	(5)
			dimensions of the elements where possible; e) Mechanical couplers or sleeves can be used to simplify the reinforcement layout and to minimize rebar congestion; and f) Defective mould forms and accessories should be repaired or replaced to prevent grout leakage during concrete.	e) Formwork is to be put up where it is necessary to contain the patch mix or grout; and f) The affected area should be protected from any disturbance during the curing period.
v)	*Missing or wrong details* — such as cast-in items, architectural nib and groove details, lifting hooks, reinforcement/starter bars/block- out/corrugated pipes.	a) Items may not have been included in the shop drawing; and b) Quality checks may not be properly in place.	a) All items should be reflected in the shop drawings for production. Any changes should be made known to the production team; and b) The use of checklists during inspection can help to ensure that all items specified in the drawing are included before casting.	a) Certain details such as missing starter bars/reinforcement and lifting hooks can be replaced by welding additional reinforcement bars after hacking off the concrete at the affected area; and b) Other items such as cast-in items, groove and block-out can be provided by chasing or chiselling out the face of the precast panels.
vi)	Strand slippage which exceed allowable design values (applicable to pre-stressed elements only) (Slippages of pre-stressing strands can be detected by visual inspection).	a) Insufficient bond strength between concrete and the pre-stressing strands; and b) Poor compaction of concrete around pre-stressed strands.	a) Required concrete strength of the precast elements should be attained and verified by cube test results before de-tensioning of strands; b) Proper compaction method should be adopted and carried out during casting;	a) It is not possible to rectify the elements from strand slippage; and b) Design verification should be carried out to ascertain the reduced capacity of the elements due to slippage, if adopted.

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Table 19 (*Concluded*)

Sl No.	Common Defects	Possible Causes	Recommendations	Remedial Measures
(1)	(2)	(3)	(4)	(5)
			c) Concrete mix design and workability should be reviewed and adjusted when needed; and b) Appropriate concrete vibrator to attain better compaction may be provided.	
vi)	Alignment	a) Inaccurate setting out and positioning of precast. elements during erection; and b) Deviation in the dimensions of the precast element.	a) Appropriate surveying and levelling equipment should be used to achieve better alignment; b) Required alignment and level should be confirmed before permanent jointing; and c) Critical dimensions of precast concrete elements should be verified before installation.	a) Minor adjustments of the element/panel alignment can be done during installation. However the effects on the final alignment and deviation of the building should be evaluated; b) For minor deviations, corrective measures such as surface grinding, trimming/ hacking and skim coat application can be appropriately used to rectify the precast elements before installation; and c) Precast elements that are not within acceptable tolerance limits and have significant effects on the structural integrity or architectural performance should not be used.

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ANNEX G

(*Foreword*, *Clauses* 5, 7.1, 10.1.2, 10.9.1 *and* 11)

SUSTAINABILITY IN PRECAST CONCRETE CONSTRUCTION

G-1 GENERAL

This annex presents sustainable practices in precast concrete construction, emphasizing methods for reducing environmental impacts, promoting resource conservation and ensuring long-term viability. The following key sustainability factors can enables the adoption of greener and efficient solutions provided the challenges involved are appropriately addressed.

G-2 USE OF RECYCLED AGGREGATES IN PRECAST CONCRETE

G-2.1 Environmental Benefits

G-2.1.1 *Conservation of Natural Resources*

By reusing construction and demolition debris as recycled aggregates, the need for extracting virgin raw materials like sand and granite is significantly reduced, preserving ecosystems and minimizing environmental disruption.

G-2.1.2 *Reduction in Energy Use*

Recycled aggregates require less energy to process compared to natural aggregates, which leads to a lower carbon footprint during production. This also reduces the energy demand associated with quarrying, transportation and processing.

G-2.1.3 *Circular Economy Impact*

Incorporating recycled aggregates into precast concrete non-structural components-such as paving slabs, kerbs, and drainage of systems, etc, supports a circular economy, minimizing waste sent to landfills and promoting the reuse of construction materials.

G-2.2 Challenges and Innovations

G-2.2.1 *Ensuring Consistent Quality*

Recycled aggregates may exhibit variability in size and composition, potentially impacting concrete strength and durability. Innovations in sorting, crushing and refining technologies can improve the consistency and reliability of recycled aggregates.

G-2.2.2 *Efficient Use of Water*

In general, excessive water is required to be used while using recycled aggregate and can have negative environmental impacts. By adopting optimized mix designs and water-efficient technologies, the water demand for concrete production can be significantly reduced.

G-2.2.3 *Advancement in Sustainable Mixes*

New research into mix designs is focusing on achieving better integration of recycled aggregates without compromising the material's strength or durability. This can lead to a more sustainable, high-performance concrete product.

G-3 ADOPTION OF RENEWABLE ENERGY SOURCES

G-3.1 Environmental Benefits

G-3.1.1 *Decarbonizing Operations*

Shifting precast concrete manufacturing facilities to renewable energy sources, such as solar, wind and

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hydropower, significantly reduces reliance on fossil fuels, helping to lower overall greenhouse gas emissions and mitigate climate change impacts.

G-3.1.2 *Electric and Hybrid Machinery*

By replacing conventional diesel-powered equipment with electric and hybrid alternatives, the precast concrete industry can reduce air pollution and carbon emissions associated with construction machinery.

G-3.2 Challenges and Innovations

G-3.2.1 *High Initial Costs*

The transition to renewable energy systems and low-emission machinery involves substantial initial investment. However, long-term operational savings, combined with available financial incentives for green technologies, can help offset the upfront costs.

G-3.2.2 *Energy Storage and Smart Grids*

Installing energy storage solutions, such as batteries, in conjunction with on-site renewable energy systems can help mitigate energy supply issues during periods of high demand or when renewable energy generation is low, further reducing dependence on the traditional energy grid.

G-4 REDUCING SITE POLLUTION AND WASTE

G-4.1 Environmental Benefits

G-4.1.1 *Waste Minimization*

Precast construction reduces on-site waste generation because most components are manufactured off-site in controlled factory settings. This reduces generation of waste and their dispersal at landfills and minimizes the potential for soil and water contamination and helps in achieving undisturbed site.

G-4.1.2 *Reduced Emissions and Noise Pollution*

Precast methods reduce the need for heavy on-site machinery and long construction timelines. As a result, there is less air pollution, noise pollution, and disturbance to nearby ecosystems or communities.

G-4.2 Challenges and Innovations

G-4.2.1 *Reducing Transportation Emissions*

Transporting precast components to construction sites can contribute to emissions, especially when distances are long. This can be minimized by optimizing transportation logistics, using hybrid or electric vehicles and focussing on the use of modular components that reduce transportation needs.

G-4.2.2 *On-Site Pollution Prevention*

During installation, on-site pollution, such as dust or runoff, can still occur. Simple practices like water misting to suppress dust and covering concrete surfaces to prevent runoff can significantly reduce the environmental impact of installation activities.

G-5 USE OF SUPPLEMENTARY CEMENTITIOUS MATERIALS (SCMs) AND OPTIMIZED CONCRETE MIXES

G-5.1 Environmental Benefits

G-5.1.1 *Low-Carbon Alternatives to Cement*

Replacing traditional Portland cement with supplementary cementitious materials (SCMs) such as fly ash, slag,

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or natural pozzolans can drastically reduce the carbon footprint of concrete production by minimizing the need for energy-intensive cement manufacturing.

G-5.1.2 *Efficient Resource Use*

Advanced mix designs, including high-performance concrete (HPC) and ultra-high-performance concrete (UHPC), allow for the reduction of material consumption without sacrificing strength or durability. These innovations help to optimize the use of raw materials, ensuring sustainability in production.

G-5.2 Challenges and Innovations

G-5.2.1 *Durability and Performance*

While reducing cement content or using alternative binders can lower the carbon footprint, it is essential to ensure the long-term durability and performance of the concrete.

G-5.2.2 *Geopolymer Concrete*

Geopolymer concrete, made from industrial by-products has a lower carbon footprint compared to traditional cement-based concrete and can be used in precast applications without compromising strength and durability.

G-6 SUSTAINABLE DESIGN AND PRODUCTION OPTIMIZATION

G-6.1 Environmental Benefits

G-6.1.1 *Material Efficiency through Design*

Precast elements can be optimised for material efficiency by carrying out optimum design. Building information modelling (BIM) allows for detailed digital models that reduce waste during both the design and construction phases.

G-6.1.2 *Modular and Prefabricated Systems*

The use of modular designs with standardization in precast construction can reduce material waste, reducing production time, reuse of mould and optimize the process of precasting.

G-6.2 Challenges and Innovations

G-6.2.1 *Customization versus Standardization*

While standardization helps reduce waste and increase efficiency, some projects require customised solutions. The challenge lies in finding a balance between benefits of standardized components and the need for bespoke, project-specific designs.

G-6.2.2 3*D Printing and Additive Manufacturing*

Emerging technologies like 3D printing could revolutionise precast construction (*refer* Annex B). By using digital fabrication techniques, custom shapes and optimized designs can be produced efficiently, with less material wastage.

G-7 TRANSPARENCY, CERTIFICATION AND ETHICAL PRACTICES

G-7.1 Sustainability Governance

G-7.1.1 *Environmental Certifications*

Precast manufacturers and construction companies can pursue certifications such as environmental management systems, to demonstrate their commitment towards achieving sustainability. These certifications set a global benchmark for eco-friendly practices in construction.

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G-7.1.2 *Ethical Sourcing and Labour Practices*

Sustainable construction practices require ethical sourcing of materials, ensuring that raw materials are extracted responsibly and workers are treated fairly. This also includes adherence to applicable labour and governance standards/practices.

G-7.2 Challenges and Innovations

G-7.2.1 *Data Transparency*

As consumer and regulatory demand for sustainability data increases, companies are under pressure to disclose their environmental impacts. Technologies like block-chain offer innovative solutions for ensuring transparency and accountability in reporting environmental performance.

G-7.2.2 *Community and Stakeholder Engagement*

Actively engaging with local communities, government bodies, and customers to ensure that sustainability goals align with the social expectations and needs of stakeholders.

ANNEX H

(*Clause* 7.9.4.8)

DESIGN AND DETAILING GUIDELINES FOR PRECAST STRUCTURES WITH HOLLOW CORE SLABS

H-1 GENERAL

This annex presents the design guidelines and detailing parameters essential for structures using hollow core slabs as the flooring system. This annexure ensures that all parameters pertaining to structural stability and end user safety of hollow core slab systems are attended to.

H-2 DESIGN GUIDELINES

H-2.1 Hollow core slabs are to be designed as a simply supported slab system. The use of hollow core slabs is not permitted as a continuous flooring system.

H-2.2 Hollow core slabs are to be mandatorily designed using structural topping as mentioned in **6.3.6.1**. This topping shall be considered acting composite with the hollow core slab provided conditions mentioned in **6.3.6.3** are met.

H-2.3 For flooring systems consisting of hollow core slabs, only the structural topping shall be considered as part of the diaphragm as noted in **6.3.6.3**. No part of the hollow core slab shall be considered as part of diaphragm or considered participating in lateral load transfer. Structural topping design to follow guidelines as prescribed in **7.10.8**.

H-2.4 The design engineer is to assess the torsional rotation of the hollow core slab caused by the differential settlement of the slab supports. Special attention is to be undertaken by the structural engineer since hollow core slabs have limited torsional resistance capacity.

H-3 DETAILING PARAMETERS

H-3.1 Low friction bearing shims or strips are to be placed under the hollow core slabs to mitigate the formation of flexural crack close to the support in the hollow core element and to mitigate spalling of support ledge concrete.

H-3.2 Concrete ledges to be used as bearing support for hollow core slabs must have the ledge reinforcement tied with the primary supporting beam or wall as shown in Fig. 57A.

H-3.3 Hollow core slabs to have minimum bearing of 80 mm to mitigate loss of support due to spalling and relative movement of the slabs.

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H-3.4 Nominal starter bars must extend for a length of 600 mm from the face of the supporting members and tied to the structural topping reinforcement to mitigate failure due to negative moments close to the support during seismic events as shown in Fig. 57B. Starter bar reinforcement diameter may be 8 mm for slabs up to 300 mm depth and 10 mm for slabs over 300 mm depth at a spacing not more than 250 mm.

H-3.5 Structural topping to be preferably reinforced with reinforcement with elongation more than 14.5 percent to mitigate brittle failure of topping. Welded mesh may be permitted as topping reinforcement provided the mesh is fabricated from reinforcement having elongation more than 14.5 percent.

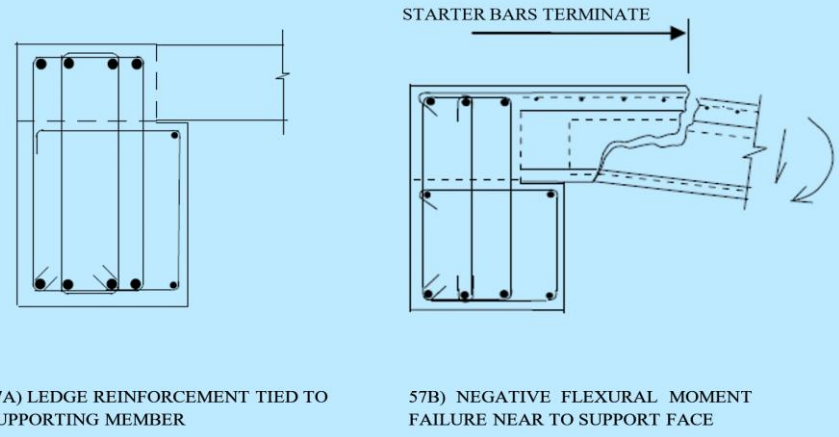

FIG. 57 DETAILING FOR HOLLOW CORE SLAB AND SUPPORTING ELELMENTS

H-3.6 A minimum of 2 cores to be opened for a length of 600 mm at both span ends of the hollow core slab, they are to be reinforced with 8 mm reinforcement for slabs up to 300 mm depth and 10 mm for slabs over 300 mm depth and filled simultaneously with the structural topping concrete. Figure 58 elaborates on the detailing that may be adopted at open core locations.

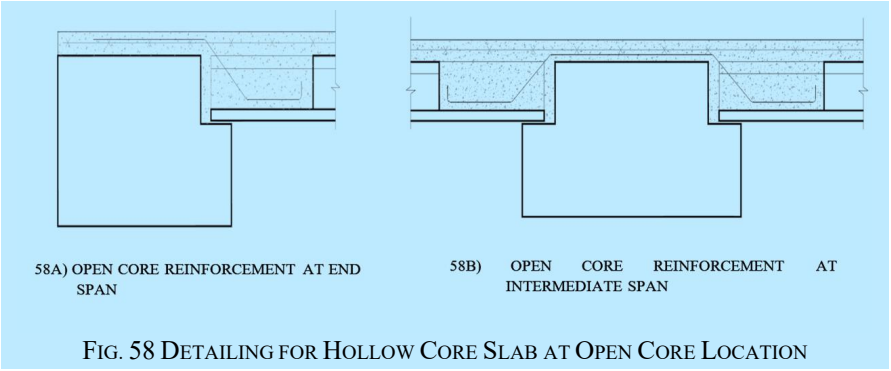

FIG. 58 DETAILING FOR HOLLOW CORE SLAB AT OPEN CORE LOCATION

H-3.7 Linking slabs to be provided to connect structural members to non-bearing ends of hollow core slabs. The length of the linking slab to be a minimum of 6 times the structural topping thickness as described in Fig. 59. The depth of the linking slab will be as per structural or load requirements.

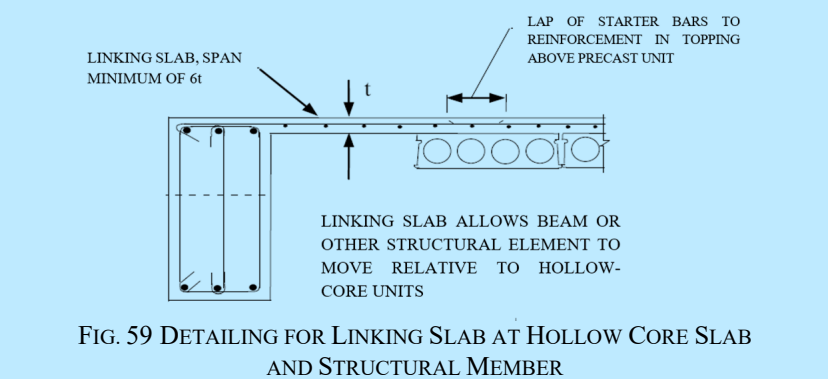

FIG. 59 DETAILING FOR LINKING SLAB AT HOLLOW CORE SLAB AND STRUCTURAL MEMBER

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LIST OF STANDARDS

The following list records those standards which are acceptable as 'good practice' and 'accepted standards' in the fulfilment of the requirements of this NBCS. The latest version of a standard shall be adopted at the time of enforcement of this NBCS. The standards listed may be used as a guide in conformance with the requirements of the referred clauses in this NBCS.

In the following list, the number appearing in the first column within parentheses indicates the number of the reference in this Section.

	IS No.	*Title*
(1)	IS 13920 : 2016	Ductile design and detailing of reinforced concrete structures subjected to seismic forces — Code of practice (*first revision*)
(2)	IS 456 : 2000	Plain and reinforced concrete — Code of practice (*fourth revision*)
(3)	IS 1343 : 2012	Prestressed concrete — Code of practice (*second revision*)
(4)	IS 383 : 2016	Coarse and fine aggregate for concrete — Specification (*third revision*)
	IS 456 : 2000	Plain and reinforced concrete — Code of practice (*fourth revision*)
(5)	IS 9103 : 1999	Concrete admixtures — Specification (*first revision*)
	IS 456 : 2000	Plain and reinforced concrete — Code of practice (*fourth revision*)
(6)	IS 10262 : 2019	Concrete mix proportioning — Guidelines (*second revision*)
(7)	IS 1199 : 1959	Methods of sampling and analysis of concrete
(8)	IS 10262 : 2019	Concrete mix proportioning — Guidelines (*second revision*)
	IS 1199 : 1959	Methods of sampling and analysis of concrete
(9)	IS 9417 : 2018	Welding of high strength steel bars for reinforced concrete construction — Recommendations (*second revision*)
(10)	IS 16172 : 2023	Reinforcement couplers for mechanical splices of steel bars in concrete — Specification (*first revision*)
(11)	IS 18256 : 2023	Solid round glass fibre reinforced polymer (GFRP) bars for concrete Reinforcement — Specification
(12)	IS 1566 : 1982	Specification for hard-drawn steel wire fabric for concrete reinforcement (*second revision*)
(13)	IS 14343 : 1996	Choice of grouting, materials for alluvial grouting — Guidelines
(14)	IS 4671 : 2018	Expanded polystyrene for thermal insulation purposes — Specification (*second revision*)
(15)	IS 8183 : 2024	Bonded mineral wool — Specification (*second revision*)
(16)	IS 6598 : 2018	Cellular concrete for thermal insulation — Specification (*first revision*)
(17)	IS 10297 : 1982	Code of practice for design and construction of floors and roofs using precast reinforced/prestressed concrete ribbed or cored slab units
(18)	IS 16700 : 2023	Criteria for structural safety of tall concrete buildings (*first revision*)

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	IS No.	*Title*
(19)	IS 1641 : 2013	Fire safety of buildings (general): General principles of fire grading and classification — Code of practice (*second revision*)
	IS 1642 : 2013	Fire safety of buildings (general): Details of construction — Code of practice (*second revision*)
(20)	IS 4326 : 2013	Earthquake resistant design and construction of buildings — Code of practice (*third revision*)
(21)	IS 4082 : 1996	Stacking and storage of construction materials and components at site — Recommendations (s*econd revision*)
(22)	IS 11384 : 2022	Composite construction in structural steel and concrete — Code of Practice (*first revision*)
(23)	IS 3201 : 1988	Criteria for design and construction of precast concrete trusses and purlins (*first revision*)
(24)	IS 6332 : 1984	Code of practice for construction of floor and roofs using precast doubly-curved shell units
	IS 10297 : 1982	Code of practice for design and construction of floors and roofs using precast reinforced/prestressed concrete ribbed or cored slab units
	IS 10505 : 1983	Code of practice for construction of floors and roofs using precast concrete waffle units
	IS 13994 : 1994	Design and construction of floor and roof with precast reinforced concrete planks and joists — Code of practice
	IS 14142 : 1994	Design and construction of floors and roofs with prefabricated brick panel — Code of practice
	IS 14215 : 1994	Design and construction of floors and roofs with precast reinforced concrete channel units — Code of practice
	IS 14242 : 1995	Design and construction of roofs using precast reinforced concrete L-Panels — Code of practice
(25)	IS 6061	Code of practice for construction of floor and roof with joists and filler blocks:
	(Part 1) : 1971	With hollow concrete filler blocks
	(Part 2) : 1981	With hollow clay filler blocks (*first revision*)
	(Part 3) : 1981	With precast hollow clay block joists and hollow clay filler blocks
	(Part 4) : 1981	With precast hollow clay block slab panels
(26)	IS 14687 : 1999	Falsework for concrete structures — Guidelines
(27)	IS 2185 (Part 1) : 2005	Concrete masonry units — Specification: Part 1 Hollow and solid concrete blocks (*third revision*)
(28)	IS 3952 : 2013	Burnt clay hollow bricks and blocks for walls and partitions — Specification (*third revision*)
(29)	IS 2185 (Part 3) : 1984	Specification for concrete masonry units: Part 3 Autoclaved cellular (Aerated) concrete blocks (*first revision*)

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	IS No.	*Title*
(30)	IS 4101 (Part 2) : 1967	Code of practice for external facings and veneers: Part 2 Cement concrete facing
(31)	IS 1950 : 1962	Code of practice for sound insulation of non-industrial buildings
(32)	IS 3483 : 1965	Code of practice for noise reduction in industrial buildings
	IS 2526 : 1963	Code of practice for acoustical design of auditoriums and conference halls
(33)	IS 1904 : 2021	General requirements for design and construction of foundations in soils — Code of practice (*fourth revision*)
(34)	IS 516 (Part 1/Sec 1) : 2021	Hardened concrete — Methods of test: Part 1 Testing of strength of hardened concrete, Section 1 Compressive, flexural and split tensile strength (*first revision*)
(35)	IS 3414 : 1968	Code of practice for design and installation of joints in buildings
(36)	IS 1786 : 2008	High strength deformed steel bars and wires for concrete reinforcement — Specification (*fourth revision*)
(37)	SP 30 : 2023	National Electrical Code of India 2023 (*second revision*)
	IS 732 : 2019	Code of practice for electrical wiring installations (*fourth revision*)
(38)	IS/IEC 62305-1 : 2010	Protection against lightning: Part 1 General principles
(39)	IS 2065 : 1983	Code of practice for water supply in buildings (*second revision*)
	IS 1742 : 1983	Code of practice for building drainage (*second revision*)
(40)	IS 1742 : 1983	Code of practice for building drainage (*second revision*)
	IS 12183 (Part 1) : 1987	Code of practice for plumbing in multi-storeyed buildings: Part 1 Water supply

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 8 GLASS AND GLAZING

BUREAU OF INDIAN STANDARDS

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National Building Code Sectional Committee, CED 46

FOREWORD

This NBCS (Part C/Section 8) deals with the use of glass in buildings. Glazing has become an important item in building construction. Glass, the primary glazing element has to be selected to cater to several requirements and the glazing has to be designed to meet various engineering requirements. Fixing of glass, a specialized operation, when properly done, will avoid the hazards of broken glass. Growing trend in resorting to glazed windows/doors in buildings and structures has considerably increased the importance of glazing and the need for proper workmanship. Considering all these, this Section was introduced in the third revision of the Code in 2016 as part of the National Building Code of India.

This Section provides guidance in the selection of appropriate glazing for various types for building construction; guidance on the energy and light aspects while choosing glazing; guidance on determining the appropriate thickness of glass used in building glazing and guidance with respect to human impact safety while using glazing in buildings.

Use of organic coated films including safety films on glass shall not classify the glass as safety glass as per this NBCS, and hence users should exercise caution in choosing their options.

Information on installation of glass mirrors and glass railings are given in the informative Annex H and Annex J, respectively.

A conformity assessment sheet for evaluating the compliance of glass and glazing systems with the requirements of this Section has been included in informative Annex K.

This Section was introduced in the Code in 2016 as part of the National Building Code of India. As a result of experience gained in implementation of 2016 version of this Section, feedback received as well as in view of formulation of new standards in the field of glass and glazing and revision of some existing standards, a need to revise this Section was felt. This revision was, therefore, prepared to take care of these aspects.

In this revision, this publication has been renamed as National Building Construction Standards (NBCS).

The significant changes incorporated in this revision include:

a) Definitions of various existing terms have been reviewed and updated, wherever required;

b) Provisions related to the requirements for laminated safety glass, toughened (tempered) safety glass and heat strengthened glass in **4** have been updated;

c) Clause **5** on energy and light has been reviewed and updated, and new provisions on energy design considerations and window design guidelines for non-conditioned/mixed mode buildings have been included;

d) The types of glass permitted for fire-resistant glazing within the scope of the Section in **6** have been updated;

e) Inclusion of provisions for determining the fire resistance performance of perimeter fire barrier joint systems, thereby enhancing fire safety measures;

f) New provisions addressing anti-fallout features, window restrictors for child safety, and dynamic wind restraint mechanisms, have been included in **7.2**, **7.3** and **7.4** respectively;

g) A new clause **9** on security glass has been introduced, covering glass for security applications such as bullet resistance, burglary resistance, and blast resistance;

j) Incorporation of provisions for assessing curtain walls and storefront systems under inter-storey drift conditions in Annex A;

h) Provisions related to railings with glass infill panels and free-standing glass protective railings have been modified in Annex J;

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k) Addition of a conformity assessment sheet to evaluate the compliance of glass and glazing systems with the requirements outlined in this Section in Annex K; and

m) References to all the concerned Indian Standards have been updated.

The information contained in this Section is largely based on the following Indian Standards:

IS No.	*Title*
IS 16231	Use of glass in buildings — Code of practice:
(Part 1) : 2019	General methodology for selection (*first revision*)
(Part 2) : 2019	Energy and light (*first revision*)
(Part 3) : 2019	Fire and loading (*first revision*)
(Part 4) : 2019	Safety related to human impact (*first revision*)

Assistance has also been derived from the following publications in the formulation of this Section:

ISO 9845-1 : 2022	Solar energy — Reference solar spectral irradiance at the ground at different receiving conditions — Part 1: Direct normal and hemispherical solar irradiance for air mass 1,5
ISO 10291 : 1994	Glass in building — Determination of steady-state U values (thermal transmittance) of multiple glazing — Guarded hot plate method
ISO 10292 : 1994	Glass in building — Calculation of steady-state U values (thermal transmittance) of multiple glazing
ISO 10293 : 1997	Glass in building — Determination of steady-state U values (thermal transmittance) of multiple glazing — Heat flow metre method

All standards, whether given herein above or cross-referred to in the main text of this Section, are subject to revision. The parties to agreement based on this Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

As the regulation of land development and buildings is a State subject and is dealt by the States and local bodies such as urban local bodies within their jurisdiction through building regulatory documents like building regulations, building byelaws, and fire regulations, this document is voluntary in nature and is non-binding. The implementation depends on adoption by concerned parties or stipulations in a contract or by appropriate adoption by the concerned authorities.

For the purpose of deciding whether a particular requirement of this Section is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with in the rounded off value should be the same as that of the specified value in this Section.

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Important Explanatory Note for Users of this NBCS

In any Part/Section of this NBCS, where reference is made to **'good practice'** in relation to design, constructional procedures or other related information, and where reference is made to **"accepted standard"** in relation to material specification, testing, or other related information, the Indian Standards listed at the end of the Part/Section shall be used as a guide to the interpretation.

At the time of publication, the editions indicated in the standards were valid. All standards are subject to revision and parties to agreements based on any Part/ Section are encouraged to investigate the possibility of applying the most recent editions of the standards.

In the list of standards given at the end of a Part/Section, the number appearing within parentheses in the first column indicates the number of the reference of the standard in the Part/Section. For example:

a) Accepted standards [C-8(1)] refers to the Indian Standards given at serial number (1) of the list of standards given at the end of this Part C/Section 8, that is, IS 14900 : 2018 'Specification for transparent float glass (*first revision*)' and IS 2553 (Part 1) : 2018 'Specification for safety glass: Part 1 Architectural, building and general uses' (*fourth revision*).

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NATIONAL BUILDING CONSTRUCTION STANDARDS

PART C STRUCTURAL DESIGN

SECTION 8 GLASS AND GLAZING

1 SCOPE

1.1 This NBCS (Part C/Section 8) covers,

a) selection and application of glass in buildings, different types of glass, their requirements and associated glazing materials;

b) guiding provision for glazing in buildings with respect to their effect on energy, visual (light) and solar environments in the building;

c) selection of glass in buildings, subject to wind loading, seismic loading and special considerations for fire rated glass and related materials;

d) provisions for the selection and manifestation of glass in buildings, subject to safety with respect to human impact of the occupants;

e) provisions relating to glazing systems such as selection, design, fabrication, installation, testing and maintenance; and

f) requirement and selection criteria for security glass in buildings.

1.2 This Section does not cover patent glazing.

NOTE — The provision of this Section pertains to glass and installation thereof. However, parties, as per their mutual agreement, may appropriately utilize the relevant provisions for installation of alternative sheeting materials used for similar purposes such as plastic glazing sheets/polycarbonate sheets taking also into account requirements of safety issues as applicable with the specific material.

1.3 The provisions in respect of sustainable use of glass in buildings are covered in relevant guidelines/best practices (Handbook on ‘Sustainability in Buildings’).

2 TERMINOLOGY

For the purpose of this Section, the following definitions shall apply, and for other terms those given in accepted standards [C-8(1)] shall apply.

2.1 Annealed Glass (*see* **Glass**) — Also known as ‘normal’ glass, which has not been subjected to toughening, lamination and heat strengthening. It is the product obtained from the process of slow and steady cooling of hot glass in order to relieve internal stresses or thermal induced stresses.

NOTE — This term is most commonly used for flat glass.

2.2 Annealed Fire Resistant Glass — Type of glass which is made out of annealed glass combination and is tested for the required duration and satisfies the intended criteria of fire resistance. These types of glasses may not possess the best of the impact resistance characteristics and may be used as vision panels.

2.3 As-Cut Finished Sizes — Finished panes of flat glass cut from stock/standard sizes. They may be subject to further processing, such as edge working, drilling, face decoration, etc.

2.4 Aspect Ratio — The ratio of a longer side of glass pane to its shorter side.

2.5 Balustrade — A low barrier forming a parapet to a stair, ramp, balcony, raised level or a change in level.

2.6 Beads or Glazing Beads — A strip of wood, metal or other suitable material attached to the rebate to retain the glass in position in the frame.

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2.7 Bite — The minimum distance by which the inner edge of a frame (or a stop) overlaps the edge of the glass panel.

2.7.1 *Glazing Bite* — The dimension by which the framing system overlaps the edge of the glazing infill. Minimum glazing bite shall be 10 mm for non-structural sealant applications.

2.8 Block (Setting Block) — A small piece of wood, plastic, rubber or other suitable material used between the edges of the glass (generally the bottom edge only) to centralize the glass in the frame (frequently called a setting block).

2.9 Chair Rail — A fixed rigid bar that provides protection from human impact.

2.10 Clear Glass — A transparent glass with neutral/near colourless appearance.

2.11 Coefficient of Linear Expansion — It is expressed as the strain (change in length per unit length) per unit variation of temperature of a material.

2.12 Combustible Material — The material which when burnt adds heat to a fire when tested for combustibility in accordance with accepted standard [C-8(2)].

2.13 Corridor — A common passage or circulation space including a common hall.

2.14 Coupled Glazing (also known as Secondary Glazing) — Two panes of glazing spaced apart in an opening, either in the frame or glazed separately, to form an unsealed cavity.

2.15 Curtain Wall — Non-load bearing structure/partition of glass attached to the building frame, usually on the exterior face. Curtain wall vertical framing members run past the face of floor slabs, and provision for anchorage is typically made at vertical framing members only.

NOTE — In contrast to combination assemblies and composite units, curtain wall systems often need to meet additional performance requirements for inter-storey differential movement, seismic drift (*see* Annex A), dynamic water infiltration, etc. Operating vents and entrance doors are provided as separate inserts.

2.16 Curtain Wall Assembly — A fire rated or non-fire rated, non-load bearing exterior wall assembly secured to the structural members of the building.

2.17 Distance Piece — A small piece of wood or other suitable material used to locate the glass between the bead and the back of the rebate to prevent lateral movement.

2.18 Door or Shutter Assembly Door-Set — A pedestrian door-set or industrial type door-set including any frame or guide, door leaf or leaves, rolling or folding curtain, etc, which is provided to give a fire resisting capability when used for the closing of permanent openings in fire resisting separating elements, which includes any side panel(s), vision panel(s), flush over panel(s) and/or transom panel(s) together with the building hardware and any seals (whether provided for the purpose of fire resistance or smoke control or for other purposes such as draught or acoustics) which form the assembly.

2.19 Double Glazed Fire Resistant Glass — A fire resistant glass that is used in a double glazed unit.

NOTE — These shall be tested as a complete Double Glazed Unit (DGU) or other multiple glazed units as the case may be. Double glazed fire resistant glass can be a combination of intumescent laminated fire rated or tempered fire resistant glass. Fire resistant glass cannot be combined with another glass and used as a DGU unless the combined double glazed or multiple glazed units are also tested.

2.20 Double Glazing — Glazing formed by an assembly of two glass panes separated by a spacer and the gap may be filled by vacuum, air or inert gases and hermetically sealed along the periphery to improve the thermal insulation.

2.21 Edge Deterioration — The discolouration of the reflective coating at the edge of the silvered glass.

2.22 Edge Faults — The faults that affect the as-cut edge of the glass. They may include entrant/emergent faults, shelling, corners on/off and vents.

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2.23 Edge Polished — It is usually applied to flat glass, the edges of which have been polished after cutting.

2.24 Edging — Grinding the edge of flat glass, to a desired shape or size.

2.25 Exposed Edge — A glass panes' edge that is not covered.

2.26 Facade — It is the front or face of the building which is part of framed or frameless system.

2.27 Faceted Glazing — It is a type of glazing in which flat panes of glass are installed vertically at an angle to each other to form a faceted surface.

2.28 Fenestration — All area (including frame) in the building envelope that let in light, including window, plastic panels, clerestories, skylight, fanlight and glass doors that are generally more than one-half the floor height, and glass block walls.

2.29 Fin — A piece of glass positioned and fastened to provide lateral support.

2.30 Fire Separation — The shortest distance, in metre, measured from the external wall of the building concerned to the external wall of any other building on the site, or from other site, or from the opposite side of street or other public space for the purpose of preventing the spread of fire.

2.31 Float Glass — Flat, transparent, clear or tinted soda-lime silicate glass having parallel and polished surfaces obtained by continuous casting and floatation on a metal bath.

NOTE — Flat glass is a general term covering sheet glass, float glass and various forms of rolled and plate glass in which shape of the glass is flat and commonly used for windows, glass doors, transparent walls and other architectural applications.

2.32 Flush Over Panel — A panel which is incorporated within a door-set or openable window and fitted above the leaf or leaves within the frame head and the jambs and with no transom.

2.33 Frame — An element made of timber/metal/aluminium or other durable material/combinations of materials, such as glass fins and structural sealant, supporting the full length of a glazed panel edge.

2.34 Frameless Glazing — An unframed glazing which maintains the integrity through pointed support structures.

2.35 Front Putty — The compound forming a triangular fillet between the surface of the glass and the front edge of the rebate.

2.36 Fully Framed Glazing — The glazing whose panels have all its four edges framed.

2.37 Gap — A clearance between two adjacent surfaces and/or edges, for example between the edge of the leaf and the reveal of the frame or between the face of the leaf and the frame stop.

2.38 Glass — An inorganic product of fusion which has cooled to a rigid condition without crystallizing. It is typically hard and brittle, and has a conchoidal fracture. It may be colourless or tinted and transparent to opaque. The term glass refers to monolithic glass unless specified otherwise.

2.39 Glass Appearance Faults — Faults such as spot and/or linear and/or enlarged area faults which alter the visual quality of glass.

2.40 Glazing — Act of securing of glass or other glazing material into a building in prepared openings in windows, door panels, partitions, etc.

2.41 Guard Rail — A permanent physical barricade used to prevent human impact on glass/glazing.

2.42 Hairline Scratch — Very fine scratch that can hardly be seen with naked eye and is associated with glass cleaning techniques.

2.43 Halo — It is the distortion zone around a spot fault.

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2.44 Heat Soaking — It is done on toughened/tempered glass by reheating to a temperature of 290 ºC and keeping it at this temperature for 8 h and cooling it gradually. The glass can break spontaneously and without provocation due to possible impurity of nickel sulphide in basic glass used for toughening/tempering. This risk of spontaneous breakage can be minimised by heat soaking process by forcing such glasses to break during the test itself.

2.45 Heat Strengthened Glass — A single piece of glass within which a permanent surface compressive stress, additionally to the basic mechanical strength, has been induced by a controlled heating and cooling process in order to give it increased resistance to mechanical and thermal stress and prescribed fracture characteristics {*see* **6.1.5** of accepted standard [C-8(3)]}.

2.46 Hermetic Seal — A complete seal (especially against the escape or entry of air which is impervious to outside interference or influence) done on insulating glass units.

2.47 High Activity Area — An area where multiple and major human activity takes place.

2.48 High Risk Area — An area prone to human injury and casualty.

2.49 Infill Balustrades — The balustrades in which the supported glass resists an infill pressure and/or point load applied to the glass panel.

2.50 Insulating Glass Unit (IGU) — An assembly consisting of at least two panes of glass, separated by one or more spaces, hermetically sealed along the periphery, mechanically stable and durable.

2.51 Interlayer — Layer or material acting as an adhesive and separator between plies of glass and/or plastic glazing sheets.

NOTE — It can be designed to give additional performance to the finished product, for example impact resistance, fire resistance, solar control and acoustic insulation.

2.52 Internal Partition — An interior dividing wall or such portion of an interior dividing wall that is not a door, side panel, shop front or atrium wall.

2.53 Light Transmission — It is the measure of light passing through a transparent or a translucent material.

2.54 Light Transmittance — It is the measure of percent of visible light transmitted through glass pane which depends on type of body substrate and coating done on glass.

2.55 Linear Defects — They are the scratches and extended spot faults on the glass.

2.56 Manifestation — Any technique for enhancing a person's awareness of the presence of transparent glass.

2.57 Maximum Thickness — The thickness of a pane of glass at the maximum thickness tolerance.

2.58 Minimum Thickness — The thickness of a pane of glass at the minimum thickness tolerance.

2.59 Modulus of Elasticity (Young's Modulus) — This modulus expresses the tensile force that theoretically has to be applied to a glass specimen to stretch it by an amount equal to its original length. It is expressed as a force per unit area (for glass, $E = 0.7 \times 10^5$ N/mm^2 = 70 GPa).

2.60 Monolithic Glass — A single sheet of flat glass which could be either annealed, toughened or heat strengthened.

2.61 Mullion — Vertical framing member supporting the edge or surface of the glass. It is responsible for transferring loads to the sub-structure.

2.62 Nominal Thickness — A numeric designation that indicates the approximate thickness of glass.

2.63 Non-Combustible Material — A material not liable to burn itself or to add heat to a fire when tested for combustibility in accordance with good practice [C-8(2)].

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2.64 Organic Coated Glass — A sheet of glass coated on one or both sides with an applied organic coating or sheeting.

2.65 Pane — A single piece of glass or plastic glazing sheet material in a finished size ready for glazing.

2.66 Panel — An assembly containing one or more panes of glass.

2.67 Partition — It means an interior non-load bearing divider, one storey in height or part.

2.68 Partly Framed or Unframed Glazing — The panels that have one or more edges unframed.

2.69 Passive Solar Gain — The solar radiation in the form of energy and light which is transmitted through the glazing into the building and which can be utilized as a source of energy to reduce the need for artificial lighting.

2.70 Patterned Glass — A rolled glass having a distinct pattern on one or both surfaces.

2.71 Perimeter Fire Barrier — The perimeter joint protection that provides resistance to prevent the passage of fire and/or smoke from floor to floor within the building at the opening between the exterior wall assembly and the floor assembly (*see also* 6.4.3.7).

2.72 Protective Coating(s) Faults — The faults where the metallic layer is exposed. They can be scratches or loss of adhesion of the protective coating(s).

2.73 Rebate — The part of a surround; the cross-section of which forms an angle into which the edge of the glass is received.

2.74 Reflective Coated Glass — It is a glass with reflective coating which uses the principle of increasing the direct reflection to maximize solar energy attenuation. In comparison with clear glass, its absorption of solar energy is also increased.

NOTE — In comparison with float glass surface, these reflecting coatings (due to their composition) exhibit lower level of emissivity which improves their *U*-value.

2.75 Reflective Silver Coating Faults — It is the fault on reflective silver layer which shall alter the appearance of the silvered glass. They consist of scratches, stains, colour spots and edge deteriorations.

2.76 Residual Protection — It is the protection provided to avoid the human impact on glass. It is provided on the side of glass where there are chances of human impact. It can be achieved by providing a sill structure or a grill inside.

2.77 Safety Glass

2.77.1 *Laminated Safety Glass* — Two or more pieces of glass held together by an interleaving layer or layers of plastic sheet material. The laminated safety glass will crack and break under sufficient impact, but the pieces of glass tend to adhere to the plastic and do not fly.

NOTE — The railing and overhead glazing shall consist exclusively of laminated safety glass.

2.77.2 *Toughened* (*Tempered*) *Safety Glass* — A single piece of specially heat treated glass, with a stress pattern such that the piece when fractured reduces to numerous granular fragments, with no large jagged edges.

2.78 Shading Coefficient — The ratio of the rate of solar heat gain through a specific unit assembly of glass to the solar heat gain through a single light of 3 mm clear glass in the same environment.

2.79 Sheet Glass — Glass made in large sheet directly from furnace or by making a cylinder and then flattening it.

NOTE — These transparent glass have a glossy, fire-finished, apparently plane and smooth surfaces, but having a characteristic waviness of surface.

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2.80 Skylight — A fenestration surface having a slope of less than 60º from the horizontal plane. Other fenestration, even if mounted on roof of a building is considered as either a vertical glazing or sloped glazing depending upon the angle of the glazing.

2.81 Sloped Overhead Glazing — A glazing that is inclined at less than 75º to the horizontal and located, wholly or partially, directly above an area that may be used by people.

2.82 Solar Energy Absorption — The percentage of the solar spectrum energy (ultraviolet, visible, and near-infrared) from 300 nm to 2 500 nm that is absorbed by a glass product.

2.83 Solar Energy Transmittance (Direct) — The percentage of energy in the solar spectrum, ultraviolet, visible, and near-infrared energy, 300 nm to 2 500 nm, that is directly transmitted through the glass.

2.84 Solar Heat Gain Coefficient (SHGC) — The SHGC is the fraction of incident solar radiation admitted through a fenestration, both directly transmitted, and absorbed and subsequently released inward through conduction, convection and radiation.

2.85 Spandrel — A non-vision portion of the exterior wall of a building.

2.86 Spot Faults — The nuclei (solid or gaseous inclusions), deposits, crush marks, etc, in the glass. In certain instances, spot faults are accompanied by a distortion zone called 'halo'. The nucleus of the spot fault is measurable.

2.87 Stain — An alteration of the reflective coating characterized by a more or less brownish, yellowish or greyish colouration of zones which can sometimes cover the whole reflective surface.

2.88 Stock/Standard Sizes — The panes of flat glass such as coated, mirror, enamelled glass, etc supplied with as-cut edges which are intended for further processing. These are generally corresponded with manufacturer's production size.

2.89 Tempered Fire Resistant Glass — A single piece of specially thermally-treated or chemically treated glass, tested for fire resistance for the required duration and having such a stress pattern that the piece when fractured, reduces to numerous granular fragments, with no large jagged edges.

2.90 Test Assembly — The complete assembly of the test specimen together with the test apparatus.

2.91 Thermal Transmittance (*U*) — Thermal transmission through unit area of the given building unit divided by the temperature difference between the air or other fluid on either side of the building unit in steady state conditions. It is also called as *U*-value. Its unit is W/m^2K.

NOTES

1 Thermal transmittance differs from thermal conductance insofar as temperatures are measured on the two surfaces of a building unit in the latter case and in the surrounding air (or other fluid) of the material on the two sides, in the former case. Thermal conductance is a characteristic of the building unit whereas thermal transmittance depends on conductance and surface coefficients of the building unit under the conditions of use.

2 In the case of glazing, thermal transmission is taken through the central part of the glazing, that is, without edge effects.

2.92 Through Component/Connection — The internal spacer or fixing that either penetrates through the door-set or openable window from one face to another or directly connects the faces of one to the other.

2.93 Tinted Glass — A normal flat glass to which colourants (normally metal oxides) are added during manufacturing process to achieve tinting and solar radiation absorption properties. It is also referred to as body tinted glass {*see* accepted standard [C-8(4)]}.

2.94 Transom — A horizontal cross bar connecting two vertical members in a glazing frame.

2.95 Unframed Glazing — A glazing with one or more edges unframed.

2.96 UV Transmittance — The percentage of energy in the ultraviolet (UV) spectrum from 300 nm to 380 nm that is directly transmitted through the glass.

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2.97 Vertical Fenestration — All fenestration other than skylights. Trombe wall assemblies, where glazing is installed within 300 mm of a mass wall are considered walls and not fenestration.

2.98 Wired Glass — It is a type of glass into which a wire mesh is embedded during production. Wired glass has an impact resistance similar to that of normal glass, but in case of breakage, the mesh retains the pieces of glass.

3 APPLICATION

The type of glass chosen shall satisfy the following criteria:

a) For a given application, the type and thickness of glass selected shall be in accordance with the requirements provided herein and the thickness calculation as subjected to wind load shall also satisfy requirements for human impact safety;

b) Installation of glass to the building is done in accordance with the various provisions available in this Section and in the absence of any specific provisions the same shall be as per manufacturer's instructions;

c) Optical, solar, thermal performance of glazing (clear/tinted/coated reflective glass) shall have the characteristic performances, such as:

   1) spectral transmittance, external and internal spectral reflectance;

   2) light transmittance, external and internal light reflectance;

   3) solar direct transmittance and reflectance;

   4) UV transmittance; and

   5) better thermal transmittance (*U*-value).

4 GENERAL METHODOLOGY FOR SELECTION

4.1 Selection

4.1.1 Right from the initial project evaluation stage, a suitable methodology that incorporates the implications of use of glass materials and their influence on performance of the building shall have to be looked into. A typical flowchart outlining such a methodology is provided (*see* Fig. 1).

It is assumed that,

a) the location of area to be glazed and its preliminary size/shape have been decided as part of normal building design process,

b) the designer has gained information on glass/and plastic glazing sheet materials used in similar circumstances, and

c) the designer is cognizant of implications of any innovation in design.

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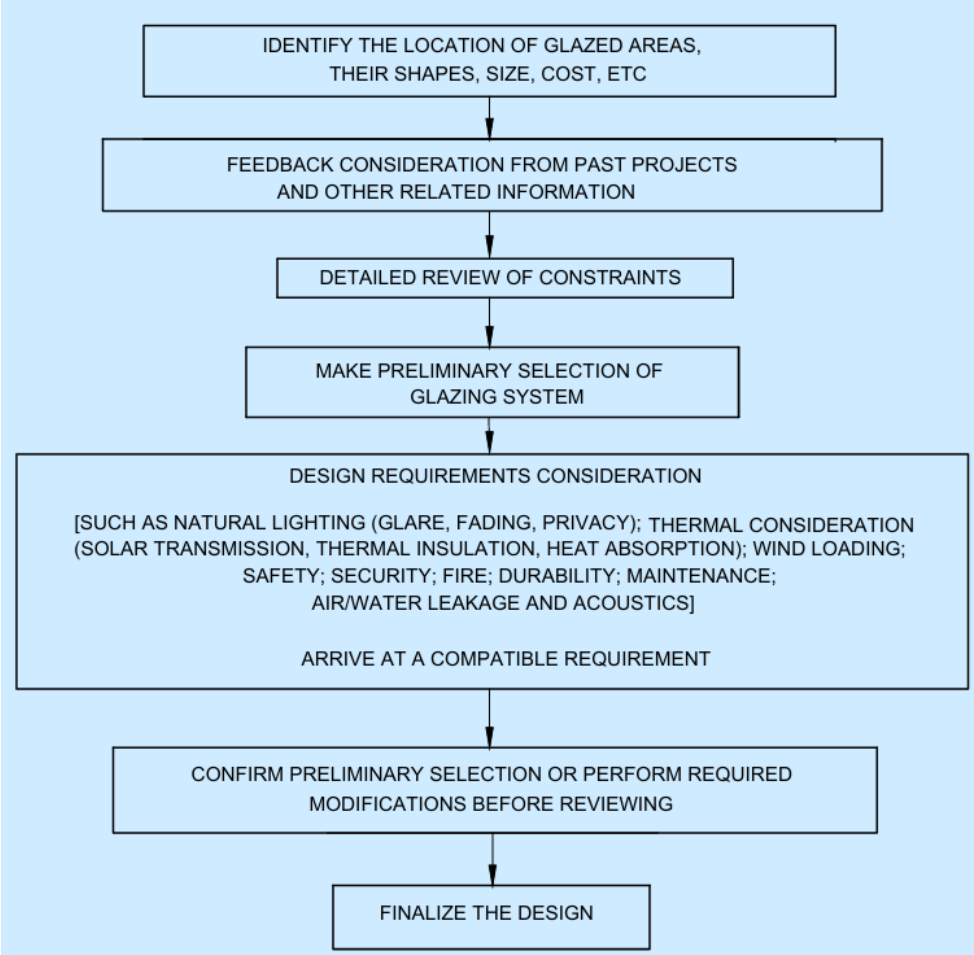

FIG. 1 TYPICAL SEQUENCE OF DESIGN EVALUATION FOR GLAZED AREAS

4.1.2 Design requirement including aesthetic considerations and client specified requirements such as security and maintenance are to be considered.

4.1.3 Effect of design on cost such as initial cost of glazing (specification of glass/plastic sheeting material; glazing method; access for initial glazing; work schedule; protection during construction); effect of glass/plastic sheeting material on capital/running cost related to building heating/cooling, lighting and ventilation; and maintenance cost of glazed areas (access for cleaning and reglazing) should be considered.

4.1.4 Selection of appropriate glass may be done considering the cost benefit analysis in respect of the benefit accrued against the alternatives available. Also, the comparative cost of glazing materials should be considered together with their life expectancy and probability of need for maintenance. In the context of selection of glass, the provisions given in the relevant guidelines/best practices (Handbook on 'Sustainability in Buildings') may also be considered in addition to the cost benefit analysis on the use of glass *vis-a-vis* use of other alternatives.

4.2 Types of Glass

4.2.1 The soda lime silicate glass is classified based on the type of internal stress which determines the strength of the glazing towards breakage, design and installation procedure and based on application. Table 1 gives the typical properties of various types of glass.

4.2.2 *Normal* (*Annealed*) *Glass*

Normal (Annealed) glass shall be as per accepted standard [C-8(5)] for flat transparent sheet glass or as per accepted standard [C-8(6)] for transparent float glass.

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4.2.3 *Laminated Safety Glass*

Two or more pieces of glass held together by an interleaving layer or layers of plastic sheet material. The laminated safety glass will crack and break under sufficient impact, but the pieces of glass tend to adhere to the plastic and do not fly. For requirements of laminated safety glass, accepted standard [C-8(7)] shall be referred.

Table 1 Typical Properties of Various Types of Glass

(*Clause* 4.2.1)

Sl No.	Properties	Annealed Glass	Float Glass	Annealed Laminated Glass	Tempered or Toughened Glass
(1)	(2)	(3)	(4)	(5)	(6)
i)	Modulus of rupture	40 N/mm^2	40 N/mm^2	40 N/mm^2	120 N/mm^2 to 200 N/mm^2
ii)	Thickness	As per accepted standard [C-8(5)]	As per accepted standard [C-8(6)]	6.38 mm to 19 mm	3 mm to 19 mm
iii)	Density (approximate)	← 2 400 kg/m^3 to 2 550 kg/m^3 →			
iv)	Modulus of elasticity	← 70 GPa →			
v)	Coefficient of linear expansion	← 9 × 10^{-6} m/m K →			
vi)	Compressive strength	← 1 000 N/mm^2 →			

NOTE — Conventionally, the following types of glass exist:

a) Annealed (clear) glass;
b) Extra clear glass;
c) Body tinted glass;
d) Coated glass (including Lacquered glass); and
   1) Online coated glass; and
   2) Offline coated glass.
e) Processed glass — Each of the glasses (a) to (e) above can be subjected to further processing to produce the following types:
   1) Heat strengthened;
   2) Toughened or tempered;
   3) Double glazed unit;
   4) Laminated glass (single laminated);
   5) Laminated double glass unit;
   6) Ceramic/screen printed fritted glass (patterned glass); and
   7) Fire safety glass.

4.2.4 *Toughened* (*Tempered*) *Safety Glass*

A single piece of specially heat treated glass, with a stress pattern such that the piece when fractured reduces to numerous granular fragments, with no large jagged edges. For requirements of toughened safety glass, accepted standard [C-8(7)] shall be referred.

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4.2.5 *Heat Strengthened Glass*

A single piece of glass within which a permanent surface compressive stress, additionally to the basic mechanical strength, has been induced by a controlled heating and cooling process in order to give it increased resistance to mechanical and thermal stress and prescribed fracture characteristics {*see* **6.1.5** of accepted standard [C-8(3)]}. Heat strengthened glass shall have a surface compression between 31 MPa and 52 MPa (4 500 and 7 500 psi) when tested as prescribed in **6.5** of accepted standard [C-8(3)]. For requirements of heat strengthened glass, accepted standard [C-8(8)] shall be referred.

NOTE — Heat strengthened glass is not a safety glass.

4.2.6 *Coated Glass*

A metallic coating is applied to one side of the glass in order to significantly increase the amount of reflection by the glass of both the visible and infrared (light and heat) ranges of the electromagnetic spectrum. This metallic coating can be applied to clear or body tinted glass, without adversely affecting the transparency of the glass. The reflective glass imparts an enhanced appearance to the exterior of buildings due to the coating of metal oxides on the glass. It is widely applied as an aesthetic and energy efficient product in buildings for its highly reflective surface and wide palette of colours. It reduces heat gain and glare from the exterior and allows optimum visible light transmission to the interior. If designed/used properly, reflective glass can aid in reducing the air conditioning load of buildings.

NOTE — Highly reflective glass should be avoided, especially in internal environments. The reflections from these surfaces can be particularly confusing for persons with vision impairment.

4.2.7 *Insulating Glazing Unit*

Insulating glazing units are two or more panels of glass spaced apart and hermetically sealed to form a single unit with air or inert gas filled space between the panels. The spacer is made of aluminium that is either mill-finished or painted/anodized in different colours or finishes, or an equivalent alternative material. Desiccants are filled inside the spacer framework to prevent condensation inside the unit. The primary sealant acts as a barrier to moisture and vapour penetration. The secondary sealant provides structural integrity to the unit. For requirements of insulating glazing unit, accepted standard [C-8(9)] shall be referred.

NOTE — Moisture penetration test and argon/krypton gas level test are typically carried out to determine the characteristic of IGUs.

4.2.8 *Rolled Glass — Patterned*, *Extra Clear Patterned*, *Wired and Wired-Patterned Glass*

The general requirements including thickness, dimensional tolerance and tests for wired glass as specified in accepted standard [C-8(10)] are valid.

4.2.9 *Glass Fins*

When glass fins are used as a decorative member, it can be either toughened (tempered) or laminated glass.

When glass fins are used as a structural member, it shall be toughened and laminated. The depth and thickness may vary as per elevation and wind load requirements. Design should also accommodate movements and verified for lateral torsional buckling. It is recommended to use finite element analysis to determine the appropriate glass fin sizes.

4.3 Application

Glass can be designed/suited to use in building occupancies such as in residential, assembly and mercantile in elements such as facades, windows, doors, partitions, display units, shop fronts, greenhouses (where plants are grown) and atrium. Different types of glass are selected based on requirements. For safety against accidental breakage and strength requirement (mechanical and thermal), toughened glass or laminated glass may be considered over annealed glass. For increased high security requirements, acoustic insulation, sloped glazing and skylights, laminated glass may be considered.

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4.4 Associated Glazing Materials

4.4.1 *Structural Sealant*

The structural sealant could be of silicone type with requirements meeting the design criteria. Structural sealant shall be shelf-stable, natural-cured, elastomeric adhesive exhibiting the desired adhesion to building substrates. The structural sealant may require cleaning and/or priming to get good adhesion for structural glazing application. For laminated glass applications where glass edges are sealed with a sealant, compatibility shall be ensured with the interlayer to avoid delamination.

Structural sealant shall be as per accepted standard [C-8(11)].

4.4.2 *Gaskets*

Gasket is a mechanical seal that fills the space between two mating surfaces, to prevent leakage from or into the joined objects and will be able to deform while under compression and tightly fills the space it is designed for, including any slight irregularities. Gaskets shall be as per **5.10** of accepted standard [C-8(12)].

4.4.3 *Preformed Tape*

Typical requirements of preformed butyl and foam tapes shall be as given in Table 2.

4.4.4 *Setting Blocks, Location Blocks and Distance Pieces*

Setting blocks, location blocks, and distance pieces shall be designed in accordance with **6** which are:

a) resilient, load-bearing, non-absorbent and rot-proof materials; and

b) compatible with all other materials that may come in contact with them.

4.5 General Requirements

4.5.1 Glazing and associated materials used for the proper installation shall satisfy the minimum dimensional and framing requirements in accordance with the procedures given in this Section. The dimensional and thickness tolerance of the glass used in exterior and interior is determined from the provisions given hereunder.

4.5.2 *Dimensional Requirement*

4.5.2.1 The dimensions for edge clearance, edge cover, and front and back clearance, as defined in Fig. 2, shall be not less than the values given in Table 3.

NOTE — For glazing systems where the framing member is aluminium or steel and gaskets are used to capture the glazing, minimum glazing bite of 12.7 mm is recommended for architectural glazing.

4.5.2.2 *Front putty width*

The front putty width shall be not less than 10 mm for panels up to 1 m^2, and not less than 12 mm for panels between 1 m^2 and 2 m^2.

Table 2 Typical Properties of Preformed Tapes

(*Clause* 4.4.3)

Sl No.	Properties	Requirement
(1)	(2)	(3)
i)	Low temperature flex	No cracks at – 23 °C
ii)	Weight loss	2 percent, *Max*
iii)	Vehicle migration	1 paper stained maximum and this stain shall be no more than 3.2 mm from edge of sample maximum

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Table 2 (*Concluded*)

Table 2 Typical Properties of Preformed Tapes (*Clause* 4.4.3)		
Sl No.	**Properties**	**Requirement**
(1)	(2)	(3)
iv)	Backing removal	No transfer of tape compound to the paper
v)	Yield strength	41.4 kPa, *Min*
vi)	Compression/Recovery	Compression index, 1.22 N/mm^2, *Max*
vii)	Water absorption	Weight gain maximum after boiling, 40 kg/m^3
viii)	Flow test	Loss of height, 60 percent, *Max*

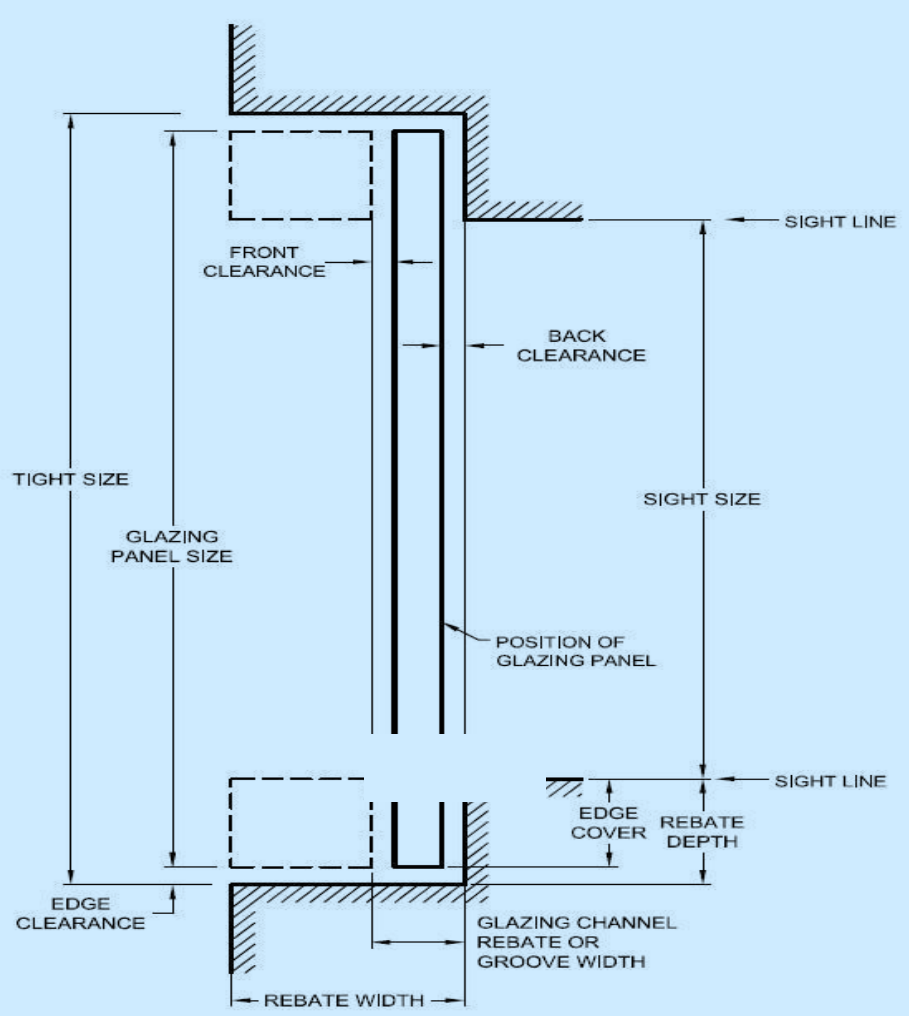

FIG. 2 SIZES AND REBATES

4.5.2.3 *Dimensions of rebates and grooves* (*width and depth*)

Dimensions of rebates and grooves shall accommodate the requirements of Table 3 and allow for the appropriate setting and location blocks (*see* **6**).

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4.5.2.4 *Glass dimensional tolerance*

The tolerance for dimensions shall be as per those in the respective Indian Standards.

4.6 Frame Requirements

4.6.1 When completely assembled and glazed, the secondary design action effects due to deflection of the frame member supporting the edge of the glass shall be allowed for.

4.6.2 *Deflection Limits*

The frame deflection limits under serviceability loading shall satisfy the conditions for four sided frames as given in **6.3.6.2**(c).

NOTE — In framing members, where reinforcement/fixing brackets are required to meet the deflection/stress criteria, it may be of aluminium or steel. When steel is used, proper dielectric galvanic separation shall be used. The material chosen for reinforcement shall be in close proximity in the galvanic series, to ensure dielectric separation.

4.6.3 *Panels Glazed into the Building Structure*

A panel glazed directly into a building structure by means of appropriate beads or stops shall be considered to be framed, provided the assembly complies with the deflection requirements of **4.6.2**.

4.6.4 *Mixed Framing*

Glass supported along the top and bottom edges by one means and along the vertical edges by another means shall be considered to be framed provided each frame member of the assembly complies with the deflection requirements of **4.6.2**.

4.7 Glass Dimensions

4.7.1 The actual thickness for each pane of glass shall be measured using screw calliper as per **4.3** of accepted standard [C-8(6)].

The actual thickness of glass, rounded to the nearest 0.1 mm shall not vary from the nominal thickness by more than the tolerances specified in respective Indian Standard or the values provided in this Section (where Indian Standard specifications are not available).

The minimum thickness allowed in insulated glazing unit (IGU) should be the sum of minimum thickness allowed of two panes as per accepted standard [C-8(6)] plus thickness of air-gap.

As per the accepted standard [C-8(7)], the nominal thickness of laminated safety glass shall be the sum of the nominal thickness of constituent panes of glass and interlayer material. The limit deviation of the interlayer shall not be taken into account if the total interlayer thickness is < 2 mm.

If the total interlayer thickness is > 2mm, a limit deviation of ± 0.2 mm shall apply. For plastic glazing sheet material, the limit of deviation on thickness shall be the same as applicable to a float glass of the same nominal thickness.

The limit deviations on thickness of laminated glass shall not exceed the sum of the limit deviations of the constituent glass panes as specified in accepted standard [C-8(6)].

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Table 3 Minimum Dimensions for Glazing Material

(*Clauses* 4.5.2.1 *and* 4.5.2.3)

All dimensions in millimetres.

Sl No.	Nominal Thickness	Front and Back Clearance			Edge Clearance	Edge Cover	Rebate Depth
		Type A (*See* Note 1)	Type B (*See* Note 2)	Type C (*See* Note 3)			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	3 (for pane area up to 0.1 m^2)	2	—	—	2	4	6
ii)	3 (for pane area > 0.1 m^2)	2	—	—	3	6	9
iii)	3	—	2	1	3	6	9
iv)	4	2	—	—	2	6	8
v)	4	—	2	1	3	6	9
vi)	5	2	2	2	4	6	10
vii)	6	2	2	2	4	6	10
viii)	8	—	3	2	5	8	13
ix)	10	—	3	2	5	8	13
x)	12	—	3	2	6	9	15
xi)	15	—	5	4	8	10	18
xii)	19	—	5	4	10	12	22
xiii)	25	—	5	4	10	15	15

NOTES

1 Type A applies to linseed oil and metal casement putties.

2 Type B applies to non-setting glazing compounds, plastic glazing compounds, two-part rubberizing compounds, sealants and preformed strip materials.

3 Type C applies to gaskets made from extruded materials such as butyl strip, PVC, neoprene and sanoprene held in position by pressure upon the glass.

4 For non-standard glass thicknesses the nearest values of nominal thickness, shall be used.

5 Timber and PVC frames may not require the specified front and back clearances provided the waterproofing performance requirements are met.

6 The dimensions are the minimum necessary for the structural integrity of glass only but do not apply to insulating glass units.

4.7.2 Length and width are defined with reference to the direction of draw of the flat glass ribbon (*see* Fig. 3).

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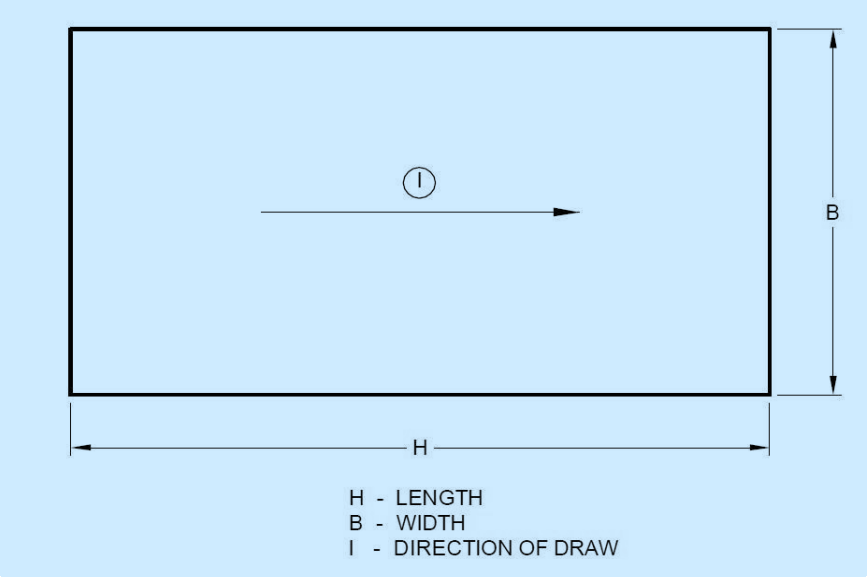

FIG. 3 REPRESENTATION OF LENGTH, WIDTH AND DIRECTION OF DRAW

4.7.3 *Jumbo Sizes*

Glass is available commonly in the following sizes, nominal length (H) 4 500 mm to 6 000 mm and nominal width, (B) 3 660 mm or 3 210 mm.

NOTE — The usual width is 3 660 mm or 3 210 mm. Other sizes may be mutually agreed to between the manufacturer and the user based on their requirement.

4.7.4 *Stock Size*

Glass is available commonly in the following sizes, nominal length (H) 2 250 mm or 2 440 mm and nominal width, (B) 3 660 mm or 3 210 mm.

NOTE — Maximum glass size available from manufacturer are one consideration, the processing equipment limitation, capabilities of contract fabricator to install unit, availability of specialized transport and handling equipment to deliver unit should be checked in addition.

4.7.5 *Split Sizes*

Glass is delivered in equal or unequal split sizes from the jumbo or stock size.

4.7.6 *Squareness*

For given the nominal dimensions for length (H) and width (B), the pane shall not be larger than a prescribed rectangle resulting from the nominal dimensions increased by the permissible plus tolerance or smaller than the prescribed rectangle reduced by the permissible minus tolerance.

The sides of the prescribed rectangles (*see* Fig. 4) shall be parallel to one another and these rectangles shall have a common centre. For stock/standard sizes the tolerances on nominal dimensions length (H) and width (B), are ± 5 mm.

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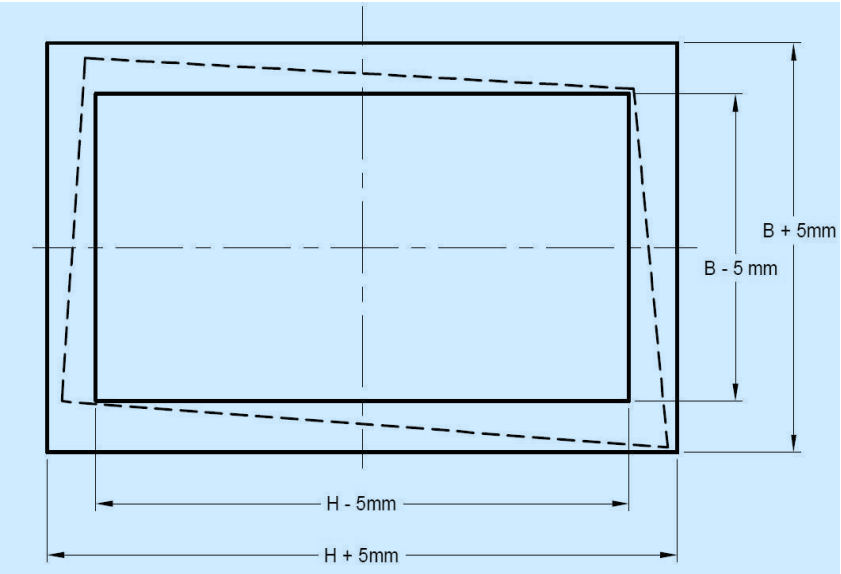

FIG. 4 DETERMINATION OF LENGTH, WIDTH AND SQUARENESS FOR STOCK/SQUARENESS DIRECTION OF DRAW

4.7.7 *As-Cut Finished Size*

The nominal dimension for width and length being given, the finished pane or as-cut finished size shall not be greater than the prescribed rectangle from the nominal dimension increased by the tolerance '*t*' or smaller than the prescribed rectangle reduced by the tolerance '*t*'. The sides of the prescribed rectangles are parallel to one another and these shall have a common centre in accordance with Fig. 5. The limits of the squareness are also mentioned. Tolerances shall be as given in respective accepted standards. An indicative list of such accepted standards is provided in Table 4.

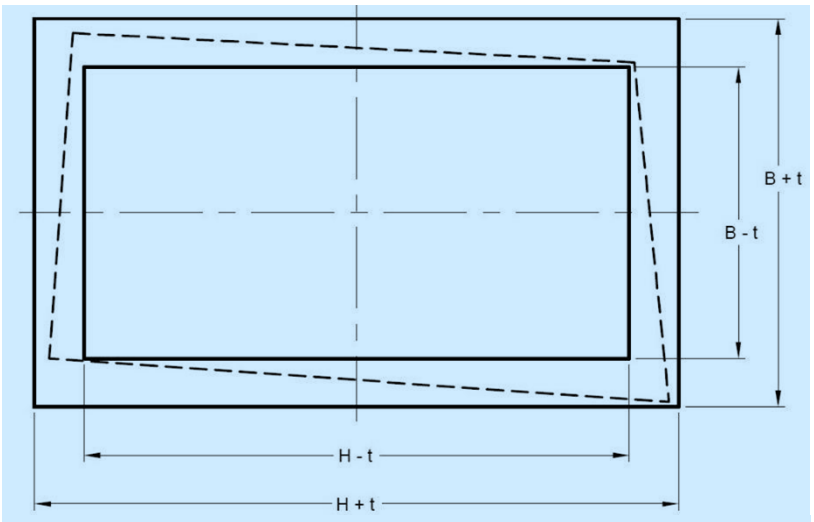

FIG. 5 TOLERANCE (t) FOR WIDTH (B) AND LENGTH (H)

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Table 4 Indicative List of Accepted Standards for Different Types of Glass

(*Clause* 4.7.7)

Sl No.	Type of Glass	Accepted Standard
(1)	(2)	(3)
i)	Flat transparent sheet glass	[C-8(5)]
ii)	Transparent float glass	[C-8(6)]
iii)	Rolled glass — Patterned, Extra Clear patterned, Wired and Wired patterned glass	[C-8(10)]
iv)	Heat strengthened glass	[C-8(8)]
v)	Insulating glazing unit	[C-8(9)]
vi)	Safety glass for architectural, building and general uses	[C-8(7)]
vii)	Tinted float glass	[C-8(4)]

5 ENERGY AND LIGHT

5.1 General

Energy and light performance of glazing system is defined in terms of U-value, solar heat gain coefficient, visible light transmission, internal and external reflection. The overall performance is determined by the efficiency of glass, frame, shading systems and the air tightness of the system. The performance requirement for glazing system shall be suitably selected. Minimum performance requirement is mentioned in the Annex B.

Specific methods of calculating light and energy factors for glass in buildings are covered here and their characteristic data can serve as a basis for light, heating and ventilation calculations of rooms and can permit comparison between different types of glazing. It is applicable to all flat glass material.

5.2 Thermal Energy Transmission

5.2.1 *General*

Glazed areas in buildings should be designed considering overall energy balance, the effects on thermal comfort of occupants and the cooling and heating energy requirements. Solar energy gain is due to transmission of solar radiant energy through the glass into the building. Solar energy is all at relatively short wavelengths and is controlled by using the glass to absorb or reflect the energy. Energy loss is due to the transfer of energy by conduction, convection and long wavelength radiation. The solar energy gain and loss can be regulated by several factors, for example, incorporating air cavities, including low emissivity glass and use of shading systems. Solar energy gain through glazing systems can be advantageous in cold climates for improving thermal comfort and reducing the heating energy requirements.

5.2.2 *Thermal Comfort*

Energy transmission through glazing can significantly influence occupant comfort by raising or lowering room air temperatures. For example, hot sunny weather can lead to excessive energy gains and energy losses can occur through the glazing during cold weather or at night. Thermal comfort can also be influenced by direct solar radiation through the glazing and/or by radiation exchange between the glazing and the occupants. Appropriate choice of glazing and opaque wall systems is essential to avoid radiant asymmetry related thermal discomfort.

5.2.3 *Solar Energy Gain*

The solar energy transmitted through the glass from the outside environment is the part of solar radiation which is transmitted directly through the glass and the part which is re-emitted from the glass after being absorbed. In warm and hot climates, this heat energy which is transmitted through the glass has to be limited in-order to achieve

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thermal comfort for the occupants and also to reduce the energy demands of the building. Factors that can influence the amount of solar gain through the glazing include:

a) orientation;

b) glazed area;

c) shading devices, both internal and external; and

d) properties of the glazing.

5.2.4 *Total Solar Energy Transmittance*

Total solar energy transmittance is the proportion of solar radiation at normal incidence transferred through the glazing. It is composed of the direct transmittance (short wave component) and the part of the solar absorptance dissipated inwards by long wave radiation and convection (long wave component). The total solar energy transmission properties of solar control glazing can be described by their shading coefficients and solar heat gain coefficient (SHGC)/solar factor (SF). The shading coefficient is derived by comparing the properties of the solar control glazing with a clear float glass having a total solar energy transmittance of 0.87 (that is clear glass between 3 mm and 4 mm thick).

Solar heat gain coefficient (SHGC)/solar factor (SF) is the measure of amount of heat transfer from outside to inside by direct transmission (short wave radiation) and internal remitted radiation (long wave radiation). The SHGC (*see* Fig. 6) or SF depends on the tint of the substrate and the type of coating that is done on the base glass. The amount of solar energy gained through glass is between 0 and 1. Multiplying the SHGC or SF by 100 gives the percentage of solar energy allowed into the building.

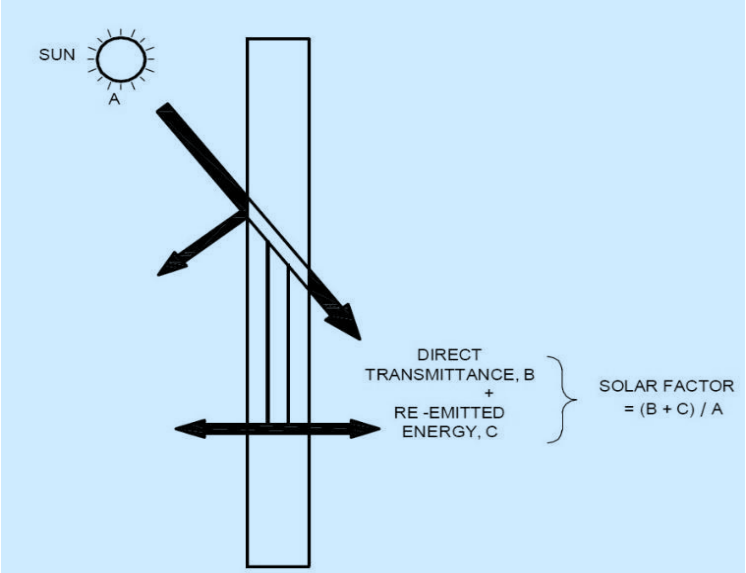

FIG. 6 SOLAR HEAT GAIN COEFFICIENT

The total solar energy transmittance g is the sum of the solar direct transmittance τ_e and the secondary heat transfer factor q_i towards the inside, the latter resulting from heat transfer by convection and long-wave IR-radiation of that part of the incident solar radiation which has been absorbed by the glazing and is expressed as:

$$g = \tau_e + q_i$$

5.2.5 *Division of Incident Solar Radiation Flux*

The incident solar radiant flux per unit area φ_e is divided into the following three parts (*see* Fig. 7):

a) transmitted part, $\tau_e\varphi_e$;

b) reflected part, $\rho_e\varphi_e$; and

c) absorbed part, $\alpha_e\varphi_e$.

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where

τ_e = solar direct transmittance;
ρ_e = solar direct reflectance; and
α_e = solar direct absorptances.

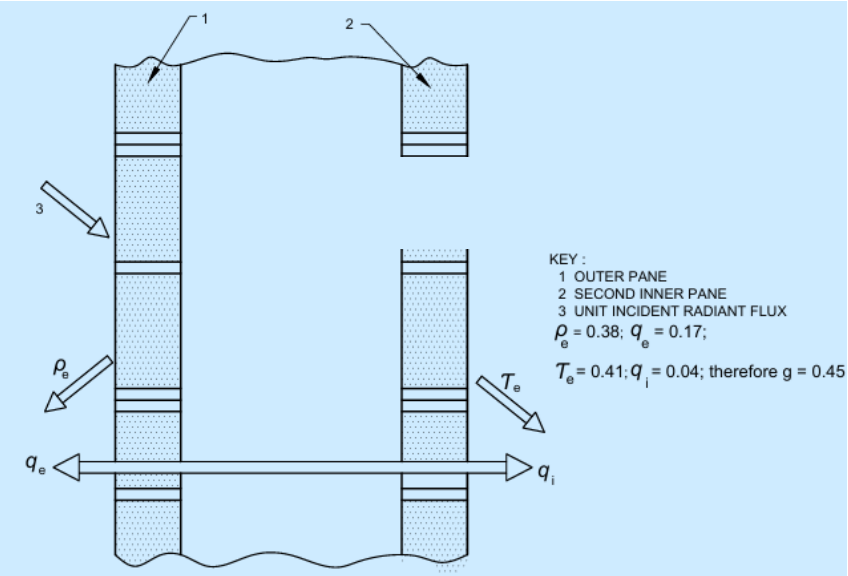

FIG. 7 COMPONENTS OF INCIDENT SOLAR RADIANT FLUX

The relationship between the three characteristics is:

$$\tau_e + \rho_e + \alpha_e = 1$$

The absorbed part $\alpha_e\varphi_e$ is subsequently divided into two parts $q_i\varphi_e$ and $q_e\varphi_e$, which are energy transferred to the inside and outside, respectively:

$$\alpha_e = q_i + q_e$$

where

q_i = secondary heat transfer factor of the glazing towards the inside; and

q_e = secondary heat transfer factor of the glazing towards the outside.

5.2.6 *Solar Direct Transmittance*

The solar direct transmittance τ_e of glazing shall be calculated using the following formula:

$$\tau_e = \frac{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} \tau(\lambda) S_\lambda \Delta\lambda}{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} S_\lambda \Delta\lambda}$$

where

S_λ = relative spectral distribution of the solar radiation;

$\tau(\lambda)$ = spectral transmittance of the glazing; and

$\Delta\lambda$ = integration procedure are the same as in **5.4.5.4** except that the data points shall be chosen at the wavelengths given in Table 5.

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5.2.7 *Solar Direct Reflectance*

The solar direct reflectance ρ_e of the glazing shall be calculated using the following formula:

$$\rho_e = \frac{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} \rho_o\,(\lambda) S_\lambda\,\Delta\lambda}{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} S_\lambda\,\Delta\lambda}$$

where

S_λ = relative spectral distribution of the solar radiation (*see* **5.2.6**);

$\rho_o(\lambda)$ = spectral external reflectance of the glazing; and

$\Delta\lambda$ = integration procedure are the same as in light transmittance (*see* **5.4.5.4**) except that the data points shall be chosen at the wavelengths given in Table 5.

In the case of multiple glazing, the spectral external reflectance $\rho_o\,(\lambda)$ is calculated in accordance with **5.5.1**.

Table 5 Normalized Relative Spectral Distribution of Global Solar Radiation
(*Clauses* 5.2.6, 5.2.7, 5.2.9.3 *and* 5.2.9.4)

Sl No.	λ nm	$S_\lambda\Delta\lambda$
(1)	(2)	(3)
i)	300	0
ii)	305	0.000 057
iii)	310	0.000 236
iv)	315	0.000 554
v)	320	0.000 916
vi)	325	0.001 309
vii)	330	0.001 914
viii)	335	0.002 018
ix)	340	0.002 189
x)	345	0.000 260
xi)	350	0.002 445
xii)	355	0.002 555
xiii)	360	0.002 683
xiv)	365	0.003 020
xv)	370	0.003 359
xvi)	375	0.003 509
xvii)	380	0.003 600
xviii)	385	0.003 529
xix)	390	0.003 551
xx)	395	0.004 294
xxi)	400	0.007 812
xxii)	410	0.011 638
xxiii)	420	0.011 877
xxiv)	430	0.011 347
xxv)	440	0.013 245
xxvi)	450	0.015 343
xxvii)	460	0.016 166
xxviii)	470	0.016 178
xxix)	480	0.016 402
xxx)	490	0.015 794

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Table 5 (*Concluded*)

Sl No.	λ nm	$S_\lambda \Delta\lambda$
(1)	(2)	(3)
xxxi)	500	0.015 801
xxxii)	510	0.015 973
xxxiii)	520	0.015 357
xxxiv)	530	0.015 857
xxxv)	540	0.015 827
xxxvi)	550	0.015 844
xxxvii)	560	0.015 590
xxxviii)	570	0.015 255
xxxix)	580	0.014 745
xl)	590	0.014 330
xli)	600	0.014 663
xlii)	610	0.015 030
xliii)	620	0.014 859
xliv)	630	0.014 622
xlv)	640	0.014 526
xlvi)	650	0.014 445
xlvii)	660	0.014 313
xlviii)	670	0.014 023
xlix)	680	0.012 838
l)	690	0.011 788
li)	700	0.012 453
lii)	710	0.012 798
liii)	720	0.010 589
liv)	730	0.011 233
lv)	740	0.012 175
lvi)	750	0.012 181
lvii)	760	0.009 515
lviii)	770	0.010 479
lix)	780	0.011 381
lx)	790	0.011 262
lxi)	800	0.026 718
lxii)	850	0.048 240
lxiii)	900	0.040 297
lxiv)	950	0.021 384
lxv)	1 000	0.036 097

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Table 5 (*Concluded*)

Sl No.	λ nm	$S_\lambda \Delta\lambda$
(1)	(2)	(3)
lxvi)	1 050	0.034 110
lxvii)	1 100	0.018 861
lxviii)	1 150	0.013 228
lxix)	1 200	0.022 551
lxx)	1 250	0.023 376
lxxi)	1 300	0.017 756
lxxii)	1 350	0.003 743
lxxiii)	1 400	0.000 741
lxxiv)	1 450	0.003 792
lxxv)	1 500	0.009 693
lxxvi)	1 550	0.013 693
lxxvii)	1 600	0.012 203
lxxviii)	1 650	0.010 615
lxxix)	1 700	0.007 255
lxxx)	1 750	0.007 183
lxxxi)	1 800	0.002 157
lxxxii)	1 850	0.000 395
lxxxiii)	1 900	0.000 082
lxxxiv)	1 950	0.001 087
lxxxv)	2 000	0.003 024
lxxxvi)	2 050	0.003 988
lxxxvii)	2 100	0.004 229
lxxxviii)	2 150	0.004 142
lxxxix)	2 200	0.003 690
xc)	2 250	0.003 592
xci)	2 300	0.003 436
xcii)	2 350	0.003 163
xciii)	2 400	0.002 233
xciv)	2 450	0.001 202
xcv)	2 500	0.000 475

NOTE — The values in this table are calculated according to the trapezoidal rule.

5.2.8 *Solar Direct Absorptance*

The solar direct absorptance α_e shall be calculated from the following formula:

$$\tau_e + \rho_e + \alpha_e = 1$$

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5.2.9 *Secondary Heat Transfer Factor Towards the Inside*

5.2.9.1 *Boundary conditions*

For the calculation of the secondary heat transfer factor towards the inside, q_i, the heat transfer coefficients of the glazing towards the outside, h_e, and towards the inside, h_i, are needed. These values mainly depend on the position of the glazing, wind velocity, inside and outside temperatures and, furthermore, on the temperature of the two external glazing surfaces.

As the purpose of this Section is to provide basic information on the performance of glazing, the following conventional conditions have been stated for simplicity:

a) Position of the glazing: vertical;

b) Outside surface: wind velocity approximately 4 m/s; corrected emissivity 0.837;

c) Inside surface: natural convection; emissivity optional; and

d) Air spaces are unventilated.

Under these conventional, average conditions, standard values for h_e and h_i are obtained:

$$h_e = 23 \text{ W/(m}^2\text{K)}$$

$$h_i = \left(3.6 + \frac{4.4\,\varepsilon_i}{0.837}\right) W/(m^2.K)$$

where

ε_i = corrected emissivity of the inside surface [for soda lime glass, $\varepsilon_i = 0.837$ and $h_i = 8$ W/(m^{2}K)].

If other boundary conditions are used to meet special requirements they shall be stated in the test report. Values for ε_i lower than 0.837 (due to surface coatings with higher reflectance in the far infrared) should only to be taken into account if condensation on the coated surface can be excluded.

5.2.9.2 *Single glazing*

The secondary heat transfer factor towards the inside, q_i, of single glazing shall be calculated using the following formula:

$$q_i = \alpha_e \frac{h_i}{h_e + h_i}$$

where

α_e = solar direct absorptance obtained from the in accordance with **5.2.5**, and

h_e, h_i = heat transfer coefficients towards the outside and inside, respectively, in accordance with **5.2.9.1**.

5.2.9.3 *Double glazing*

The secondary heat transfer factor towards the inside, q_i, of double glazing shall be calculated using the following formula:

$$q_i = \frac{\left(\frac{\alpha_{e1} + \alpha_{e2}}{h_e} + \frac{\alpha_{e2}}{\Lambda}\right)}{\left(\frac{1}{h_i} + \frac{1}{h_e} + \frac{1}{\Lambda}\right)}$$

where

α_{e1} = solar direct absorptance of the outer (first) pane within the double glazing;

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α_{e2} = solar direct absorptance of the second pane within the double glazing;

Λ = thermal conductance between the outer surface and the innermost surface of the double glazing (*see* Fig. 8), in watts per square metre per Kelvin ($W/m^2.K$); and

h_e, h_i = heat transfer coefficients towards the outside and the inside, respectively in accordance with **5.2.9.1**.

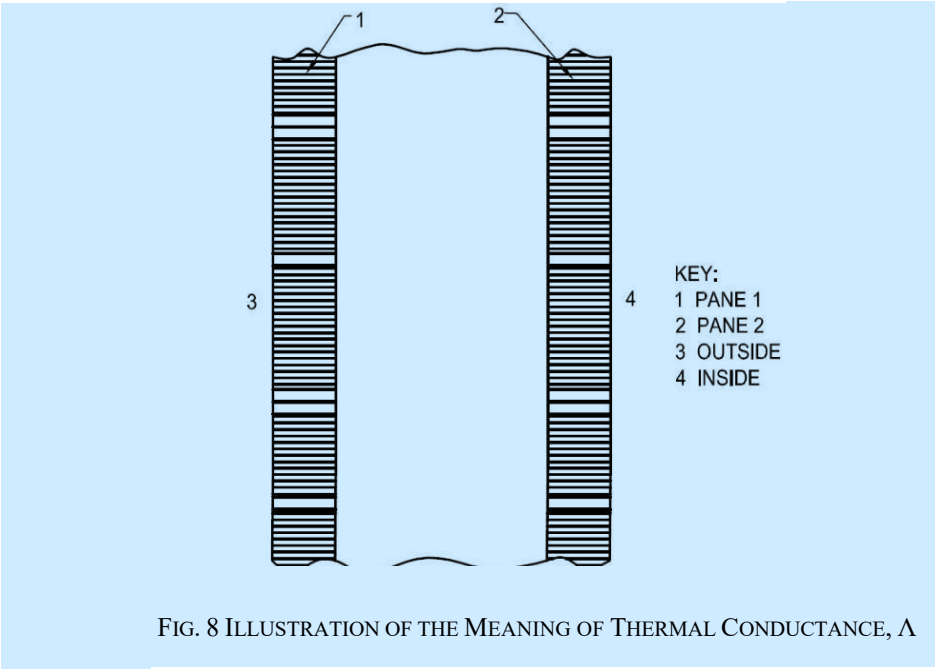

FIG. 8 ILLUSTRATION OF THE MEANING OF THERMAL CONDUCTANCE, Λ

Characteristics αe1 and αe2 are calculated as follows:

$$\alpha_{e1} = \frac{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} \left\{\alpha_1(\lambda) + \frac{\alpha_1'(\lambda)\tau_1(\lambda)\rho_2(\lambda)}{1 - \rho_1'(\lambda)\,\rho_2(\lambda)}\right\} S_\lambda \Delta\lambda}{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} S_\lambda \Delta\lambda}$$

$$\alpha_{e2} = \frac{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} \left\{\frac{\alpha_2(\lambda)\tau_1(\lambda)}{1 - \rho_1'(\lambda)\,\rho_2(\lambda)}\right\} S_\lambda \Delta\lambda}{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} S_\lambda \Delta\lambda}$$

where

$\tau_1(\lambda), \tau_2(\lambda), \rho_1(\lambda), \rho_1'(\lambda), \rho_2(\lambda)$ are as defined in light transmittance (*see* **5.4.5.4**)

$\alpha_1(\lambda)$ = spectral direct absorptance of the outer pane, measured in the direction of the incident radiation, given by the relationship:

$$\alpha_1(\lambda) = 1 - \tau_1(\lambda) - \rho_1(\lambda)$$

$\alpha_1'(\lambda)$ = spectral direct absorptance of the outer pane, measured in the opposite direction to the incident radiation, given by the relationship:

$$\alpha_1'(\lambda) = 1 - \tau_1(\lambda) - \rho'_1(\lambda)$$

$\alpha_2(\lambda)$ = spectral direct absorptance of the second pane, measured in the direction of the incident radiation, given by the relationship:

$$\alpha_2(\lambda) = 1 - \tau_2(\lambda) - \rho_2(\lambda)$$

$\Delta\lambda$ and the integration procedure are the same as in **5.4.5.4** except that the data points shall be chosen at the wavelengths given in Table 5.

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The thermal conductance Λ shall be determined for a temperature difference of ΔT = 15 °C across the sample and a mean temperature of the sample of 10 °C, or by measuring methods using the standard guarded hot plate method, or the standard heat flow metre method.

If another temperature difference ΔT across the sample and/or another mean temperature of the sample is used for the determination of the thermal conductance Λ to meet special requirements, this shall be stated in the test report.

5.2.9.4 *Multiple glazing with n > 2 components*

The secondary heat transfer factor towards the inside, q_i, of a multiple glazing with more than two components shall be calculated using the following formula:

$$q_i = \frac{\frac{\alpha_{e1}+\alpha_{e2}+\alpha_{e3}+\cdots+\alpha_{en}}{h_e} + \frac{\alpha_{e2}+\alpha_{e3}+\cdots+\alpha_{en}}{\Lambda_{12}} + \frac{\alpha_{e3}+\cdots+\alpha_{en}}{\Lambda_{23}} + \cdots + \frac{\alpha_{en}}{h_e}}{\frac{1}{h_i}+\frac{1}{h_e}+\frac{1}{\Lambda_{12}}+\frac{1}{\Lambda_{23}}+\cdots+\frac{1}{\Lambda_{(n-1)n}}}$$

where

α_{e1} = solar direct absorptance of the outer (first) pane within the *n*-fold glazing;

α_{e2} = solar direct absorptance of the second pane within the *n*-fold glazing;

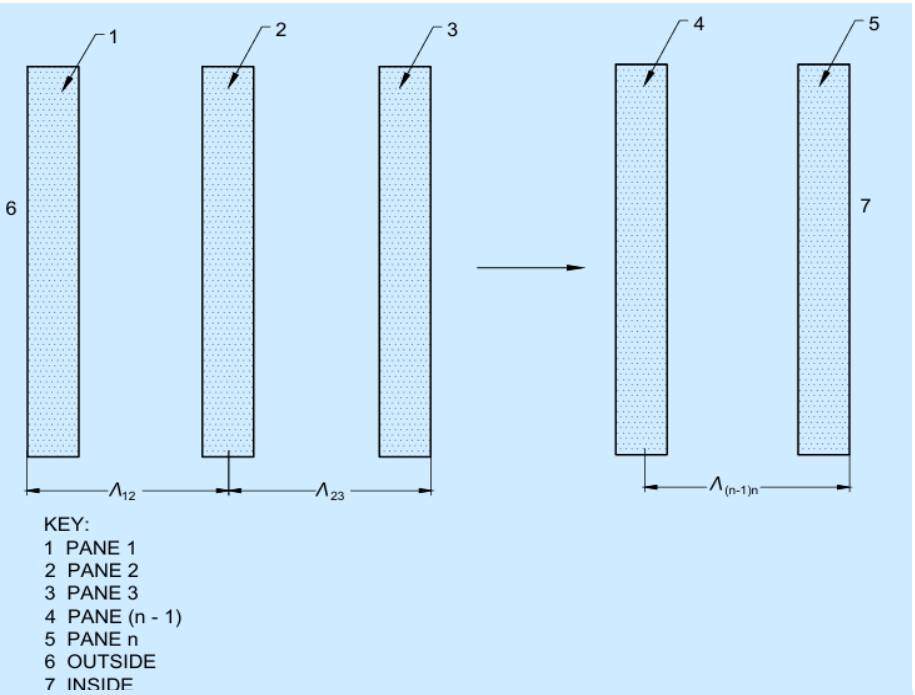
α_{en} = solar direct absorptance of the n^{th} (inner) pane of the *n*-fold glazing;

h_e, h_i = heat transfer coefficients towards the outside and towards the inside, respectively in accordance with **5.2.9.1**;

Λ_{12} = thermal conductance between the outer surface of the outer (first) pane and the centre of the second pane (*see* Fig. 9);

$\Lambda_{(n-1)n}$ = thermal conductance between the centre of the $(n-1)^{th}$ pane and the outer surface of the n^{th}(inner) pane (*see* Fig. 9); and

Λ_{23} = thermal conductance between the centre of the second pane and the centre of the third pane (*see* Fig. 9);



NOTE — For triple glazing, pane 3 corresponds to pane n.

FIG. 9 ILLUSTRATION OF THE MEANING OF THE THERMAL CONDUCTANCES Λ_{12}, Λ_{23}……$\Lambda_{(n-1)n}$

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The thermal conductances Λ_{12}, $\Lambda_{23\ldots\ldots\ldots\Lambda(n-1)n}$ shall be determined by iteration technique. The calculation of the direct solar absorptances $\alpha_{e1,}$ $\alpha_{e2,\ldots}\alpha_{en}$ shall be performed using the methods given in **5.2.9.3**.

As an example for the calculation of the direct solar absorptances the following procedure is given which consist of the following (n-1) steps for a glazing consisting of n components:

a) First step — calculate the spectral characteristics of a unit consisting of (n-1) components 2,3,.....n according to what has been prescribed in **5.4.5.4** and **6.1**. Then combine this unit with the first (outer) pane as a double glazing;

b) Second step — calculate the spectral characteristics of a unit consisting of the (n-2) components 3,....,n and furthermore, those of a double glazing consisting of pane 1 and pane 2. These units are then combined as a double glazing. The sum $\alpha_{e1} + \alpha_{e2}$ is then obtained. This procedure is continued up to the last (n-1)th step; and

c) (n-1)th step — combine the (n-1) panes 1,2,... (n-1) and determine the spectral characteristics of this unit. This unit is then combined with the nth (inner) pane as a double glazing. From the second equation in **5.2.9.3**, the sum $\alpha_{e1,}$ $\alpha_{e2\ldots}\alpha_{e(n-1)}$ is obtained, that is with the known values $\alpha_{e1,}$ $\alpha_{e2,\ldots}$ $\alpha_{e(n-2)}$ from the previous steps $\alpha_{e(n-1)}$ is determined. α_{en} is obtained according to the expression of α_{e2} in **5.2.9.3**.

In the case of triple glazing for the solar absorptances α_{e1}, α_{e2} and α_{e3} as a function of the spectral characteristics of the individual components of the unit, the following formulae are obtained:

$$\alpha_{e1} = \frac{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} \left\{\alpha_1(\lambda) + \frac{\tau_1(\lambda)\alpha_1'(\lambda)\rho_2(\lambda)\,[1-\rho_2'(\lambda)\,\rho_3(\lambda)] + \tau_1(\lambda)\tau_2^2(\lambda)\alpha_1'(\lambda)\,\rho_3(\lambda)}{[1-\rho_1'(\lambda)\,\rho_2(\lambda)].[1-\rho_2'(\lambda)\,\rho_3(\lambda)] - \tau_2^2(\lambda)\rho_1'(\lambda)\,\rho_3(\lambda)}\right\} S_\lambda \Delta\lambda}{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} S_\lambda \Delta\lambda}$$

$$\alpha_{e2} = \frac{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} \left\{\frac{\tau_1(\lambda)\alpha_2(\lambda)\,[1-\rho_2'(\lambda)\,\rho_3(\lambda)] + \tau_1(\lambda)\tau_2(\lambda)\alpha_2'(\lambda)\,\rho_3(\lambda)}{[1-\rho_1'(\lambda)\,\rho_2(\lambda)].[1-\rho_2'(\lambda)\,\rho_3(\lambda)] - \tau_2^2(\lambda)\rho_1'(\lambda)\,\rho_3(\lambda)}\right\} S_\lambda \Delta\lambda}{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} S_\lambda \Delta\lambda}$$

$$\alpha_{e3} = \frac{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} \left\{\frac{\tau_1(\lambda)\tau_2(\lambda)\alpha_3(\lambda)}{[1-\rho_1'(\lambda)\,\rho_2(\lambda)].[1-\rho_2'(\lambda)\,\rho_3(\lambda)] - \tau_2^2(\lambda)\rho_1'(\lambda)\,\rho_3(\lambda)}\right\} S_\lambda \Delta\lambda}{\sum_{\lambda=300\text{ nm}}^{2\,500\text{ nm}} S_\lambda \Delta\lambda}$$

where

$\tau_1(\lambda)$, $\tau_2(\lambda)$, $\tau_3(\lambda)$, $\rho_1(\lambda)$, $\rho'_1(\lambda)$, $\rho_2(\lambda)$, $\rho'_2(\lambda)$, $\rho_3(\lambda)$ are as defined in light transmittance;

$\alpha_1(\lambda)$, $\alpha'_1(\lambda)$ and $\alpha_2(\lambda)$ are as defined in **5.2.9.3** for double glazing;

$\alpha'_2(\lambda)$ = spectral direct absorptance of the second pane, measured in the opposite direction to the incident radiation, given by the relationship:

$$\alpha'_2(\lambda) = 1 - \tau_2(\lambda) - \rho'_2(\lambda)$$

$\alpha_3(\lambda)$ = spectral direct absorptance of the third pane, measured in the direction of the incident radiation, given by the relationship:

$$\alpha_3(\lambda) = 1 - \tau_3(\lambda) - \rho_3(\lambda)$$

$\Delta\lambda$ and the integration procedure are the same as in **5.4.5.4** except that the data points shall be chosen at the wavelengths given in Table 5.

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For a glazing with more than three components, the formulae for the solar absorptances $\alpha_{e1}, \alpha_{e2}, \ldots, \alpha_{en}$ as a function of the individual components.

5.2.9.5 *Mean radiant temperature* (*MRT*)

Besides solar energy transmittance, heat gain and losses occur by radiation exchange between the glazing which influences thermal comfort of the occupants. Mean radiant temperature (MRT), which is defined as the uniform temperature of an imaginary enclosure in which radiant heat transfer from the human body is equal to the radiant heat transfer in the actual non-uniform enclosure, is used as a metric in thermal comfort assessment. MRT can be calculated by various methods, among which measuring with globe thermometer is the most common and is given by:

$$MRT = \sqrt[4]{T_g{}^4 + \frac{h_{cg}}{\varepsilon_g \sigma}(T_g - T_a)^4}$$

where

T_a = air temperature, in Kelvin;

T_g = black globe temperature, in Kelvin;

ε_g = emissivity of black globe;

σ = Stefane-Boltzmann constant; and

h_{cg} = convective heat transfer coefficient of the black globe.

5.2.10 *Thermal Transmittance due to Temperature Difference*

The other part to be looked at is the transfer of heat from outside to inside environment through the glass, due to the temperature difference. This is known as U-value, which is the rate of heat flow through one square metre of glazing when there is a temperature difference of 1 °C. The lower the U-value, the lesser is the heat transfer.

5.2.11 *Total Solar Energy Transmission*

The total solar energy transmitted into the room per unit area of glazing φ_{ei} is given by the relationship:

$$\varphi_{ei} = \varphi_e g$$

where

φ_e = incident solar radiation flux per unit area;

g = total solar energy transmittance of the glazing; and

The values of φ_e can be obtained from appropriate tables in meteorological literature.

NOTE — The SHGC/SF requirement for the glazing should be selected based on the prescriptive requirements are mentioned in Annex B.

5.2.11.1 *Additional heat transfer*

If the room temperature T_i differs from the outside temperature T_o, an additional heat transfer occurs in addition to φ_{ei}. This additional heat flow q_z can be calculated as follows:

$$q_z = U\,(T_o - T_i)$$

where U is the U-value (thermal transmittance) of glazing.

5.2.12 *Frames*

These can contribute to a significant amount of energy gain and energy loss through the glazing system. For improved efficiency, some frames are designed with internal thermal breaks that reduce energy flow through the

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frame (*see* Fig. 10). The sealing between the openings and the window frames and between the opening sashes of windows and fixed portions are important areas to be addressed as major heat loss happens from this area due to poor design, selection and application of sealant material. Adequate provision shall also be made to ensure that the frames are not susceptible to water ingress in case of driving rain.

NOTE — As the evaluation of structural property differs from a monolithic profile, for evaluation of the structural property of thermally broken profile, the method described in Annex G of accepted standard [C-8(12)] shall be followed.

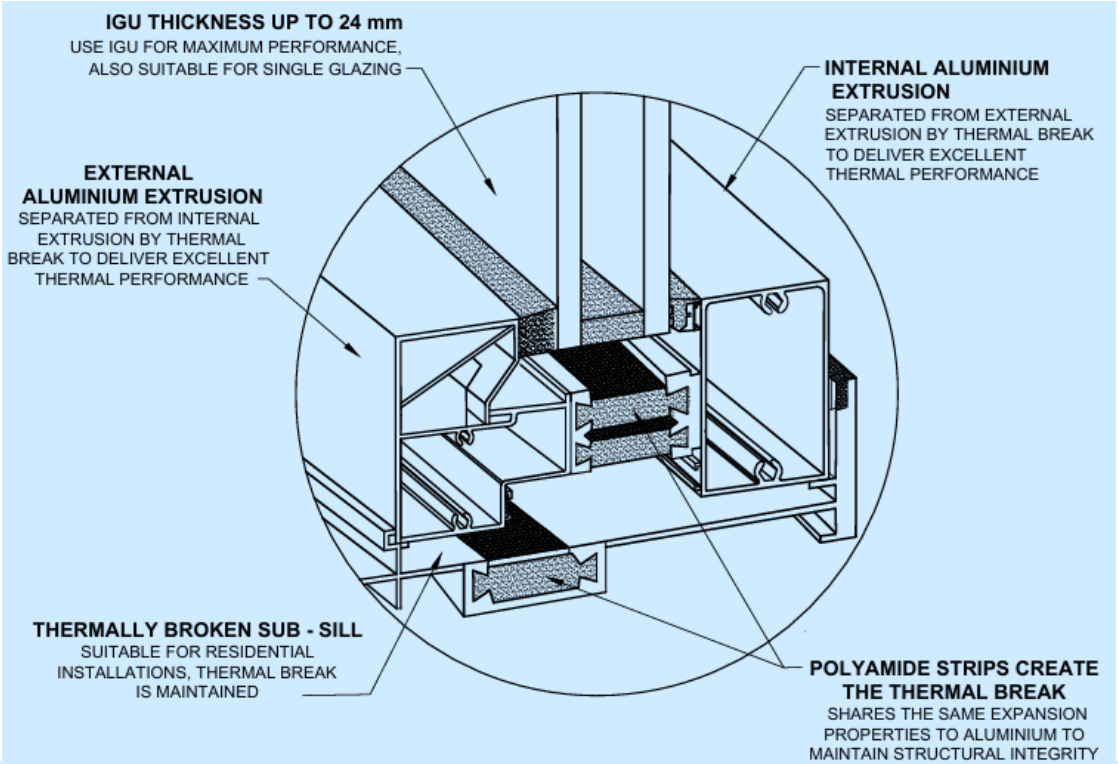

FIG. 10 THERMALLY BROKEN ALUMINIUM FRAME ASSEMBLY FOR IMPROVED THERMAL PERFORMANCE IN GLAZING SYSTEMS

5.2.13 *Shading Systems*

These regulate the solar radiation incident on the glazing thereby reducing the effective solar heat gain coefficient of the glazing system. Besides reduction in direct solar gains inside the building, shading systems reduce the radiant heat gain from the glazing surface. Shading system design requires due consideration for the geographic location of the building, orientation of the facade, location of the system, and solar geometry. External shading systems are effective in solar control compared to internal shading systems (*see* Fig. 11). Shading systems may impact the daylight availability and the views available for occupants. Kinetic systems, either automated or manually operated, can be used to vary shading system positions dynamically based on solar position, daylight requirements and occupant preferences.

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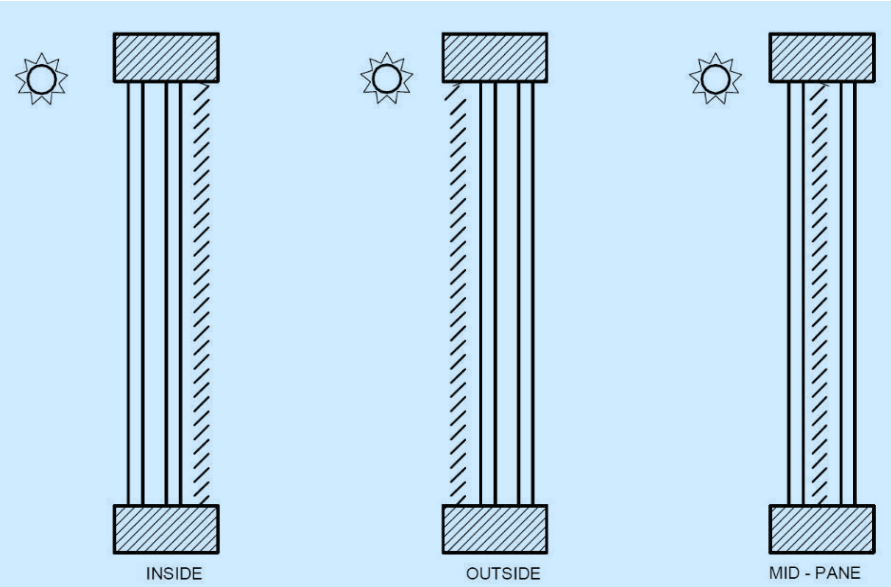

FIG. 11 PLACEMENT OF SHADING DEVICES FOR OPTIMAL SUN CONTROL IN GLAZING SYSTEMS

5.2.14 *Air Infiltration*

Inadequate control of airflow through the building envelope is often a primary factor contributing to premature building envelope failures. Air infiltration may be caused by wind pressure, stack pressure or fan pressure (*see* Fig. 12). Wind creates a positive pressure on the windward face and negative pressure on the leeward face, causing infiltration and exfiltration. Besides the positive or negative pressure created by the heating, ventilation and air conditioning system may lead to exfiltration or infiltration, respectively.

The stack effect is when warm air moves upward in a building because it is lighter than cold air and escapes out of the upper levels of the building, through penetrations and cracks in the building envelope or other openings. The rising warm air reduces the pressure in the base of the building, forcing cold air to infiltrate through open doors, windows, or other openings. The stack effect basically causes air infiltration on the lower portion of a building and exfiltration on the upper part. Mechanical equipment such as fans and blowers cause the movement of air within buildings and through enclosures, which can generate pressure differences. If more air is exhausted from a building than is supplied, a net negative pressure is generated, which can induce unwanted airflow through the building envelope.

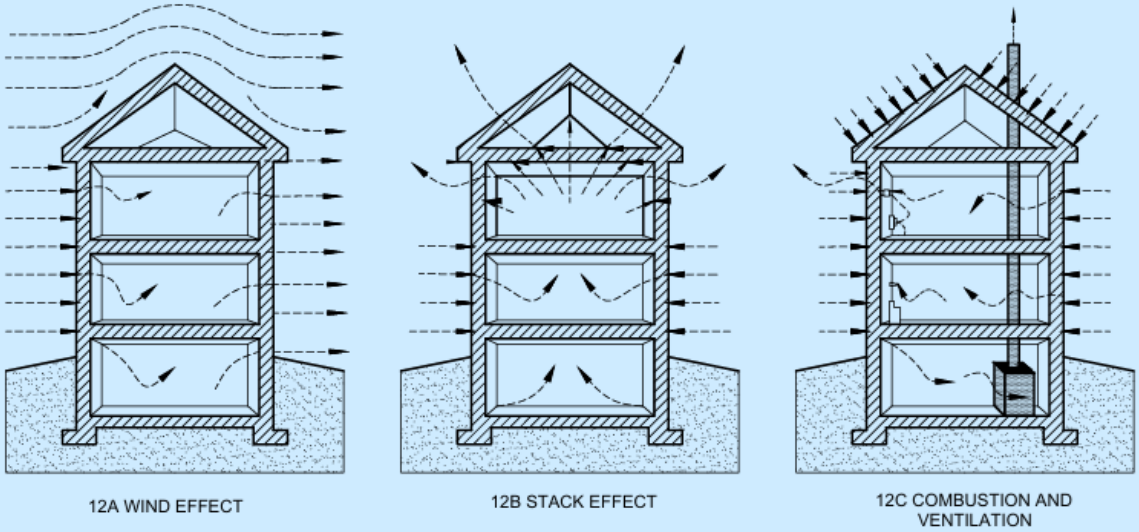

FIG. 12 MECHANISMS OF AIR INFILTRATION: WIND EFFECT, STACK EFFECT, AND COMBUSTION AND VENTILATION IN BUILDING ENCLOSURES

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5.2.14.1 *Considerations for controlling air infiltration*

a) Ensuring the continuity of each component serving its role in resisting infiltration, such as a window assembly or a curtain, and ensuring they are interconnected to prevent air leakage at the joints between materials, components, assemblies, and systems; and

b) Effective structural support shall be ensured so that all components can resist the positive or negative structural loads imposed on them by wind, stack effect, and HVAC fan pressures without rupture, displacement or undue deflection, thereby resisting air infiltration through them. Design considerations shall ensure that these loads are safely transferred to the structure and provision of adequate resistance to these pressures by use of fasteners, tapes, adhesives, etc.

Inadequate sealing of the glazing system may lead to energy gains and losses, thereby affecting thermal comfort and increasing the cooling and heating energy demand. If moisture-laden air infiltrates through the glazing system, the moisture may, under certain environmental conditions condense, especially in cold climates.

5.3 Methods for Improving Thermal Insulation

Glass readily conducts energy and so is a poor insulator. To improve resistance to energy transfer, insulating glass units, coupled glazing or high performance coatings can be used. Increasing the thickness of the glass makes little difference to the *U*-value of the glazing.

5.3.1 *Use of Low Emissivity Coatings*

Low emissivity (low-E) coatings have surface emissivity of less than 0.2. The use of such a solar control low-E coating on glass improves the thermal insulation. They are most efficient when used on the cavity surfaces of insulating glass units.

5.3.2 *Solar Control Glass*

Solar control glass can be manufactured in several forms. The function of solar control glass is to reduce the total solar energy transmittance, which usually leads to a decrease in the transmission of the visible part of the solar spectrum. However, some tints and coatings are able to attenuate preferentially non-visible solar radiation, leaving the transmission of the greater proportion of the visible radiation largely unchanged.

The application of ceramic frit-fired into the surface of the glass can also be used to modify the energy and light transmission of the glass. The effect can be varied across a pane depending on the screen print pattern. Manufacturers should be consulted for specific details.

Solar control glass can be used in single glazed or double glazed unit depending upon the site requirements. The relationship between light transmission and total solar energy transmission is expressed as the light/energy ratio.

There is no optimum light/energy ratio. The selection of an appropriate product depends on the requirements of the building. Solar gain may be relatively high for making use of passive solar gains, or low to reduce air conditioning loads. Light transmission will depend on the extent to which daylight is used to obviate artificial lighting.

Some solar control glasses can be toughened or heat strengthened and gives a means of raising the design stress and enhances safety of thermal fracture. The following are the common varieties.

NOTE — Some manufacturers use this as a descriptive code, quoting a light transmission figure followed by a total solar energy transmission figure.

a) *Body tinted glass* (*for increased absorption*) — Solar control properties and colour vary with the thickness of the glass. When used in insulating glass units, body tinted glass should be positioned as the outer pane because the energy due to the absorbed radiation is more easily dissipated to the outside.

b) *Reflective coated glass* — It uses the principle of increasing the direct reflection to maximize solar energy attenuation. In comparison with clear glass, its absorption of solar energy is also increased. An informative annex on test requirements is given at Annex C.

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The advantages of such glass types are:

1) Greater performance range than body tinted glass;
2) Higher performances (greater solar energy attenuation);
3) Light/energy ratios nearer to the theoretical limit; and
4) A range of colour appearances in transmission and reflection.

The coatings may be placed on to body-tinted glass to extend the range of performances.

NOTES

1 Compared with a float glass surface, these reflective coatings (owing to their composition) exhibit lower levels of emissivity which improves their *U*-value.
2 Performance data in comparison to clear glass can be had from manufacturers.

c) *Laminated glass* — It is commonly manufactured with clear glass and clear interlayers, but solar control properties can be incorporated into laminated glass by including either solar control glass or tinted interlayers or both.

Laminated glass with clear interlayers and solar control glass exhibit similar properties to the solar control glass from which it is made.
Laminated glass with a tinted interlayer acts in a similar manner to body tinted glass, by absorbing the solar radiation, but with a different range of colours and performances.

NOTE — Performance data in comparison to clear glass can be had from manufacturers. Performances of a typical range of laminated glass products with clear glass and tinted interlayers can be had from manufacturers.

d) *Insulating glass units* — These are used primarily for improved thermal transmittance (*U*-value) (*see* **5.2.10**), but can also improve the total solar energy transmittance. This improvement is a result of the incorporation of a second pane of glass together with the hermetically sealed air space. The second glass pane can be of any glass type. Outer pane can be solar control low emissivity glass (surface 2) and inner pane can be clear glass (*see* Fig. 13A). The other option is outer pane can be solar control glass (surface 2) and inner pane with low emissivity coating glass (surface 3) (*see* Fig. 13B).

NOTES

1 Soft coat low emissivity glass can be used as the hermetically sealed airspace protects the coating.
2 Performances of a typical range of insulating glass units with clear float inner pane or with hard/soft coat low emissivity glass inner pane can be had from manufacturers.

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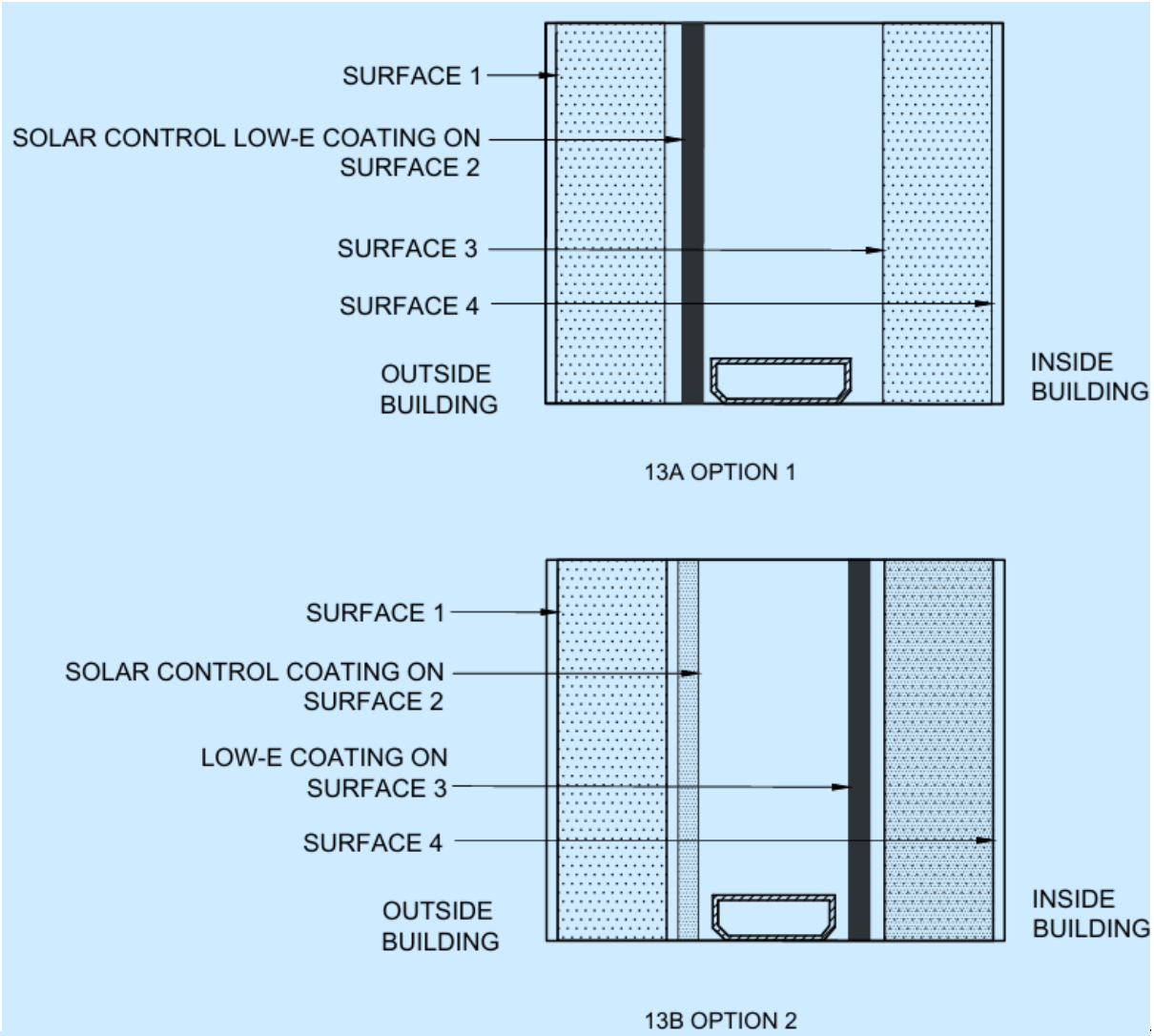

FIG. 13 CONFIGURATIONS OF INSULATING GLASS UNITS

5.3.3 *Reducing Thermal Transmittance of Glazing System*

5.3.3.1 *Increasing the width of the air space*

Enhanced thermal insulation can be achieved by increasing the width of the airspace. However, there is a convection of the gas in the cavity.

5.3.3.2 *Using gases of lower thermal conductivity*

Replacing the air in the cavity with, for example, argon/krypton, can improve the thermal insulation.

5.3.3.3 *Inhibiting convection within the air space*

Filling the cavity with cellulose material reduces convection and makes the cavity a more efficient insulator. However, this usually results in loss of vision, since the materials are, at best, translucent.

5.3.3.4 *Evacuation of the air space*

In theory, a vacuum shall eliminate energy transfer by conduction and convection. However, a vacuum puts high demands on the glass from the external air pressure.

NOTE — Evacuation of the air space is currently considered to be technically impractical.

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5.3.3.5 *Evaluation of type of spacer used in double glazed unit*

Thermal performance of the glazing system for conduction can be improved by changing the material of the spacer.

5.3.3.6 *Typical U-values of glass products*

Table 6, Table 7 and Table 8 give typical *U*-values.

Table 6 Thermal Transmittance (*U*-Value) of Glass Products: Single Glazing

(*Clause* 5.3.3.6)

Sl No.	Glass Thickness mm	*U*-Value W/m²K
(1)	(2)	(3)
i)	4	5.8
ii)	6	5.7
iii)	10	5.6
iv)	12	5.5

Table 7 Thermal Transmittance (*U*-Value) of Glass Products: Insulating Glass Units

(*Clause* 5.3.3.6)

Sl No.	Clear glass Thickness and type mm	Cavity Width mm	*U*-Value W/m²K	
			Air	Argon
(1)	(2)	(3)	(4)	(5)
i)	4+4	6	3.3	3.1
		12	2.9	2.7
		16	2.7	2.6
		20	2.8	2.6
ii)	4+4 Low E (ϵ_d = 0.15)	6	2.7	2.3
		12	1.9	1.6
		16	1.7	1.5
		20	1.7	1.5
iii)	4 + 4 Low E (ϵ_d = 0.04)	6	2.5	2.0
		12	1.6	1.3
		16	1.4	1.2
		20	1.4	1.2
iv)	4 + 4 + 4	6	2.4	2.1
		12	1.9	1.8
		16	1.8	1.7
		20	1.8	1.7
v)	4 Low E (ϵ_d = 0.15) + 4 + 4 Low E (ϵ_d = 0.15)	6	1.7	1.4
		12	1.7	1.0
		16	1.0	0.8
		20	0.9	0.8
vi)	4 Low E (ϵ_d = 0.04) + 4 + 4 Low E (ϵ_d = 0.04)	6	1.6	1.2
		12	1.0	0.7
		16	0.8	0.6
		20	0.7	0.6

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Table 7 (*Concluded*)

Table 7 Thermal Transmittance (*U*-Value) of Glass Products: Insulating Glass Units

(*Clause* 5.3.3.6)

Sl No.	Clear glass Thickness and type mm	Cavity Width mm	*U*-Value W/m^{2}K	
			Air	Argon
(1)	(2)	(3)	(4)	(5)

NOTES

1 *U*-values for argon gas-filled cavity based on 90 percent argon/10 percent air.
2 ϵ_d is the declared (normal) emissivity.

Table 8 Thermal Transmittance (*U*-Value) of Glass Products: Coupled Glazing

(*Clause* 5.3.3.6)

Sl No.	Clear Glass Thickness and Type mm	Separation mm	*U*-Value W/m^{2}K
(1)	(2)	(3)	(4)
i)	4 + 4	25	2.8
		75	2.8
ii)	4 + 4 Low E	25	1.7
		75	1.9

NOTE — ϵ_d is the declared (normal) emissivity.

The *U*-value requirement for the glazing system should be selected in accordance with the Table 9 and Annex B.

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Table 9 Thermal Performance of Different Glass Shading Devices

(*Clause* 5.3.3.6)

Sl No.	Name of the Shading Device	Transmittance, *U*-Value	Shade Factor
(1)	(2)	(3)	(4)
i)	Plain glass sheet (3.0 mm thick)	5.23	1.00
ii)	Plain glass + Wire mesh outside	5.00	0.65
iii)	Painted glass:		
	a) White paint	5.22	0.35
	b) Yellow paint	5.22	0.37
	c) Green paint	5.22	0.40
iv)	Heat absorbing glass	4.65	0.45
v)	Plain glass sheet + Venetian blind inside:	3.14	
	a) Light colour		0.35
	b) Dark colour		0.40
vi)	Plain glass sheet which is,	5.23	
	a) 100 percent shaded		0.14
	b) 75 percent shaded		0.34
	c) 60 percent shaded		0.56

NOTE — Indicative values, actual value of the system to be measured using either approved simulation software or testing at laboratory.

5.3.4 *Thermal Safety of Glass*

Thermal safety of glass should be assessed considering the amount of radiation incident on the surface and the thermal capabilities of the glass. For example, the solar radiation intensity on the glass surface should be determined along with the air temperature range applicable to the location of the building. These measurements, together with the energy transfer coefficients and the glass absorption allow determination of the appropriate basic temperature difference between the central area of the glass and its edge. The difference is related to the thermal stress and then modified for the type of glazing system, taking account of extraneous effects resulting from curtains, blinds, back-up walls, proximity to heaters, etc to derive a stress in actual service conditions.

NOTES

1 High air temperatures, low rates of air movement, and the insulation provided by curtains, blinds, back-up walls and multiple glazing tend to reduce the loss of energy and uphold the centre temperature. Low temperatures at the edges are maintained by conduction from the glass through the frame to a cold building structure with a large thermal capacity.

2 Advice may be sought from the manufacturer of glass as to the methods for assessing the thermal safety of the glass.

The resultant service stress should then be compared with the design stress for the glass. If on comparison, the service stress is less than or equal to the design stress, the glass and glazing system may be accepted as thermally safe provided, that the edges of the glass are of adequate quality.

NOTE — Where the application of a solar control film is being considered to existing glazing, advice should be sought from the manufacturer on the effect of any additional thermal stress likely to be induced in the glass.

The normal mode of thermal breakage of glass is by the action of tensile stress located in and parallel to an edge, and so the breaking stress of the glass is mainly dependent on the extent and position of flaws in the edges. The condition of the glass edge is therefore extremely important.

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Solar control glasses should not be nipped to size and any panes with shelled or vented edges should not be accepted for glazing in orientations subject to direct sunlight. Although a wheel-cut edge is the most satisfactory, laminated glasses with worked edges may be used.

Where clean-cut edges are not permitted, arises should be created by a wet process, working parallel to the edge and not across the thickness, and the design implications of such an action should be examined.

Where solar control glasses are to be used in sliding doors and windows there is always the possibility that, when opened during sunny periods, the overlapping will function as double glazing with little ventilation in the air space, and it is this condition that should be assumed in assessing the thermal safety of glass.

Thermal safety assessment is based on the behaviour of glass in good condition and properly glazed. Even if the glass is shown to be thermally safe, this depends on close adherence to the recommend glazing procedures. All necessary precautions should be taken to see that only glass with edges of an acceptable condition is used. The glass should be stored and handled so that no contact with hard bodies can damage the edges and each pane or insulating glass unit should be carefully examined immediately before glazing.

5.3.5 *UV-Transmittance*

The UV-transmittance of glazing is the fraction of the incident solar radiation transmitted by the glazing in the 300 nm to 380 nm range (UV-B range from 300 nm to 315 nm and UV-A range from 315 nm to 380 nm). The UV-transmittance τ_{UV} is calculated as follows:

$$\tau_{UV} = \frac{\sum_{\lambda=300\ nm}^{380\ nm} \tau\ (\lambda) S_\lambda\ \Delta\lambda}{\sum_{\lambda=300\ nm}^{380\ nm} S_\lambda\ \Delta\lambda}$$

where

S_λ = relative spectral distribution of UV radiation;

$\tau(\lambda)$ = spectral transmittance of the glazing; and

$\Delta\lambda$ and the integration procedure are the same as **5.4.5.4** except that the data points shall be chosen at the wavelength given in Table 10.

This average extends over the defined UV-portion of the solar spectrum. It may not be correlated with solar radiation damage of materials and skin.

Table 10 Normalized Relative Spectral Distribution of Global Solar Radiation

(*Clause* 5.3.5)

Sl No.	λ nm	$S_\lambda \Delta\lambda$
(1)	(2)	(3)
i)	300	0
ii)	305	0.001 859
iii)	310	0.007 665
iv)	315	0.017 961
v)	320	0.029 732
vi)	325	0.042 466
vii)	330	0.062 108
viii)	335	0.065 462
ix)	340	0.071 020
x)	345	0.073 326
xi)	350	0.079 330

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Table 10 (*Concluded*)

Table 10 Normalized Relative Spectral Distribution of Global Solar Radiation

(*Clause* 5.3.5)

Sl No.	λ nm	$S_\lambda \Delta\lambda$
(1)	(2)	(3)
xii)	355	0.082 894
xviii)	360	0.087 039
xix)	365	0.097 963
xx)	370	0.108 987
xxi)	375	0.113 837
xxii)	380	0.058 351

NOTE — The values in this table are calculated according to the trapezoidal rule.

5.4 Daylight Transmission

Good glazing system design can, by reducing reliance on artificial lighting, can be one of the largest single means of saving energy and should be carefully considered.

Account should be taken of shape of the room, window orientation, occupancy patterns and task, together with the relationship of windows to surrounding buildings and other obstruction. As with task lighting, possible solar overheating might result and similar precautions should be taken.

5.4.1 *Light Transmission Properties of Window*

Light transmission is defined as the fraction of visible light at normal incidence transmitted through the glazing. For properties of plastic glazing sheet materials, manufacturers may be consulted.

Dirt on glazing reduces the light transmission, often by an appreciable extent before becoming noticeable. To ensure day lighting levels are adequate, an allowance for the reduced light transmission should be made in day lighting calculation by introducing a ‘dirt factor’ between 0.7 and 1.0. Regular cleaning of glass and plastic glazing sheet materials is therefore important.

5.4.2 *Glare*

5.4.2.1 *General*

Glare results from excessive contrast of illumination, or from an excess of illumination in the field of view. Reaction to it is subjective. When correctly designed, natural lighting should not result in glare problem.

NOTE — Contrasts in excess of 10 : 1 in illumination in different parts of the field of view might give rise to glare in some form.

5.4.2.2 *Disability glare*

Disability glare is glare which impairs the visibility of objects without necessarily causing discomfort.

During a sunny day, even a light transmission of the glazing resulting in 10 000 lux or lower, is likely to cause disability glare.

Glare can be reduced by some form of mechanical shading, for example a canopy, an overhanging floor, a balcony or a louvre system. Alternatively, internal screening can be provided by louvres or blinds.

NOTE — Any fixed shading system reduces the amount of natural light entering the building throughout the year, irrespective of whether there is a glare problem at any particular time.

It may also be possible to re-orientate the glazing to avoid entry of direct solar radiation. Alternatively, the interior layout can be suitably designed to eliminate glare.

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5.4.2.3 *Discomfort glare*

Glazing products with light transmission lower than 50 percent can ameliorate discomfort glare. These products decrease the sky luminance components but permanently reduce the admission of daylight. Alternatively, shading devices, internal or external, movable or fixed, may be used.

Other methods of reducing the problems of glare should be considered, including,

a) installation of windows in more than one wall to raise the general background illumination and, in so doing, to reduce the contrast between a window and its surrounding surfaces;

b) use of light coloured matt finishes for the window frames and the surrounding surfaces;

c) splayed reveals, to assist in reducing the contrast between the window and its surroundings;

d) use of slender glazing bars and transoms of high reflectance; and

e) lowering window sills to allow increased illumination to enter, which increases the adaptation level and reduces the likelihood of discomfort glare.

5.4.3 *Diffusion and Obscuration*

The nature of some glazing products (for example, patterned or acid etched glass) can cause the direct incident solar beam to be scattered diffusely. Hence the window might assume an uncomfortable high brightness and become a discomfort glare source in its own right. Diffusing glazing used within the normal field of view should be used with caution.

5.4.4 *Fading*

Most materials can fade when subjected to either daylight (particularly direct sunlight) or artificial light. Fading is a complex phenomenon involving many chemical reactions, initiated or accelerated by light of different wavelengths. Generally, the better quality dyes and pigments fade relatively slowly and react only to the shorter wavelengths (ultraviolet and the blue end of the visible spectrum). Other materials can fade quickly and might do so under light of much longer wavelengths. It is the combination of wavelength, available light and transmission which determines glass selection to minimise fading, not simply the UV transmission.

5.4.5 *Determination of Characteristic Parameters*

5.4.5.1 *General*

The characteristic parameters are determined for quasi-parallel, almost normal radiation incidence.

5.4.5.2 *Test setup*

The samples shall be irradiated by a beam whose axis is at an angle not exceeding 10° from the normal to the surface. The angle between the axis and any ray of the illuminating beam shall not exceed 5°.

The characteristic parameters are as follows:

a) spectral transmittance $\tau(\lambda)$, the spectral external reflectance $\rho_o(\lambda)$ and the spectral internal reflectance $\rho_i(\lambda)$ in the wavelength range of 300 nm to 2 500 nm;

b) light transmittance τ_V, the external light reflectance $\rho_{v,0}$ and the internal light reflectance $\rho_{v,i}$ for illuminant D65;

c) solar direct transmittance τ_e and the solar direct reflectance ρ_e;

d) total solar energy transmittance (solar heat gain co efficient SHGC) g; and

e) UV-transmittance τ_{UV};

When calculating the characteristic parameters of multiple glazing, the spectral data of each glass component is to be used instead of integrating data of the complete unit.

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5.4.5.3 *Performance requirements of optical measurements*

Optical measurements in transmission and reflection require special care to achieve accuracy in transmittance and reflectance of about ± 0.01.

The wavelength calibration and the photometric linearity of commercial spectrophotometers shall be checked periodically using reference materials obtained from metrological laboratories.

The wavelength calibration shall be performed by measuring glass plates which feature relatively sharp absorption bands at specified wavelengths; the photometric linearity shall be checked using grey filters with a certified transmittance.

For reflectance measurements, reference materials having reflection behaviour (that is, reflectance level and ratio of diffuse and direct reflectance).

Thick samples (that is laminated glass or insulating units) can modify the optical path of the instrument's beam as compared to the path in air and therefore the sample beam hits an area of the detector having a different responsivity.

A similar source of inaccuracy occurs in case of samples with significant wedge angles which deflect the transmitted (and reflected) beams. It is recommended to check the reproducibility by repeating the measurement after rotating the sample.

Additionally, in the case of reflectance measurements, glass sheets cause a lateral shear of the beam reflected by the second surface, causing reflectance losses (whose extent is particularly evident in the case of thick and/or wedged samples). This source of inaccuracy shall be taken into account particularly in the case of reflectance measurements through the uncoated side. In order to quantify and correct systematic errors, it is recommended to use calibrated reflectance standards with a thickness similar to the unknown sample.

In the case of diffusing samples (or samples with a non-negligible diffusing component or wedged samples), transmittance and reflectance measurements shall be performed using integrating spheres whose openings are sufficiently large to collect the entire diffusely transmitted or reflected beam. The sphere diameter shall be adequate and the internal surface adequately coated with a highly diffusing reflectance material, so that the internal area can provide the necessary multiple reflections. Reference materials with characteristics similar to the unknown sample as specified above shall be used. If the transmittance or reflectance curve recorded by the spectrometer exhibits a high level of noise for some wavelengths, the values to be considered for those wavelengths should be obtained after a smoothing of the noise.

5.4.5.4 *Light transmittance*

Light transmittance is the measure of the amount of light passing through the glazing. The visible light lies between the wavelength 380 nm and 780 nm. The light transmission of the glazing depends on the substrate type and coating layer done on the base glass.

The light transmittance, τ_v of glazing shall be calculated using the following formula:

$$\tau_v = \frac{\sum_{\lambda=380\text{ nm}}^{780\text{ nm}} \tau(\lambda) D_\lambda V(\lambda)\,\Delta\lambda}{\sum_{\lambda=380\text{ nm}}^{780\text{ nm}} D_\lambda V(\lambda)\Delta\lambda}$$

where

D_λ = relative spectral distribution of illuminant D65;

$\tau(\lambda)$ = the spectral transmittance of glazing;

$V(\lambda)$ = spectral luminous efficiency for photopic vision defining the standard observer for photometry; and

$\Delta\lambda$ = wavelength interval.

Table 11 indicates the values for $D_\lambda V(\lambda)\Delta\lambda$ for wavelength intervals of 10 nm. The table has been drawn up in such a way that $\Sigma D_\lambda V(\lambda)\Delta\lambda = 1$.

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In the case of multiple glazing, the spectral transmittance $\tau(\lambda)$ shall be obtained by calculation from the spectral characteristics of the individual components. Alternatively, measurements on non-diffusing multiple units may be performed using an integrating sphere. This may be achieved after reducing the interspaces under conditions that allow the collection of the whole transmitted beam.

The calculation of the spectral transmittance $\tau(\lambda)$ shall be performed using methods such as algebraic manipulation. Any algorithm that can be shown to yield consistently the correct solution is acceptable.

For the calculation of $\tau(\lambda)$ as well as for the calculation of spectral reflectance the following symbols for the spectral transmittance and spectral reflectance of the individual components are used:

$\tau_1(\lambda)$ = spectral transmittance of the outer (first) pane;

$\tau_2(\lambda)$ = spectral transmittance of the second pane;

$\tau_n(\lambda)$ = the spectral transmittance of the n^{th} (inner) pane (for example, for triple glazing n= 3);

$\rho_1(\lambda)$ = spectral reflectance of the outer (first) pane measured in the direction of incident radiation;

$\rho'_1(\lambda)$ = spectral reflectance of the outer (first) pane measured in the opposite direction of incident radiation;

$\rho_2(\lambda)$ = spectral reflectance of the second pane measured in the direction of incident radiation;

$\rho'_2(\lambda)$ = spectral reflectance of the second pane measured in the opposite direction of incident radiation;

$\rho_n(\lambda)$ = spectral reflectance of the n^{th} (inner) pane measured in the direction of incident radiation;

$\rho'_n(\lambda)$ = spectral reflectance of the n^{th} (inner) pane measured in the opposite direction of incident radiation.

For the spectral transmittance $\tau(\lambda)$ as a function of the spectral characteristics of the individual components of the unit, the following formulae are obtained:

a) For double glazing; and

b) For triple glazing.

For multiple glazing with more than three components, relationships similar to above expressions are found to calculate $\tau(\lambda)$ of such glazing from the spectral characteristics of the individual components.

As an example for calculating $\tau(\lambda)$ according to the procedures of this Section, a glazing composed of five components may be treated as follows:

1) First consider the first three components as triple glazing and calculate the spectral characteristics of this combination;

2) Next, run the same procedure for the next two components as double glazing; and

3) Then calculate $\tau(\lambda)$ for the five component glazing, considering it as double glazing consisting of the preceding triple and double glazing.

The light transmission requirement for the glazing should be selected in accordance with relevant Indian Standards.

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Table 11 Normalized Relative Spectral Distribution $D_\lambda V(\lambda)\Delta\lambda$

(*Clause* 5.4.5.4)

Sl No.	λ nm	$D_\lambda V(\lambda)\Delta\lambda$ x 10^2
(1)	(2)	(3)
i)	380	0
ii)	390	0.000 5
iii)	400	0.003 0
iv)	410	0.0103
v)	420	0.035 2
vi)	430	0.094 8
vii)	440	0.227 4
viii)	450	0.419 2
ix)	460	0.666 3
x)	470	0.985 0
xi)	480	1.518 9
xii)	490	2.133 6
xiii)	500	3.349 1
xiv)	510	5.139 3
xv)	520	7.052 3
xvi)	530	8.799 0
xvii)	540	9.442 7
xviii)	550	9.807 7
xix)	560	9.430 6
xx)	570	8.689 1
xxi)	580	7.899 4
xxii)	590	6.330 6
xxiii)	600	5.354 2
xxiv)	610	4.249 1
xxv)	620	3.150 2
xxvi)	630	2.081 2
xxvii)	640	1.381 0
xxviii)	650	0.807 0
xxix)	660	0.461 2
xxx)	670	0.248 5
xxxi)	680	0.125 5
xxxii)	690	0.053 6
xxxiii)	700	0.027 6
xxxiv)	710	0.014 6

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Table 11 (*Concluded*)

Sl No.	λ nm	$D_\lambda V(\lambda)\Delta\lambda$ x 10^2
(1)	(2)	(3)
xxxv)	720	0.005 7
xxxvi)	730	0.003 5
xxxvii)	740	0.002 1
xxxviii)	750	0.000 8
xxxix)	760	0.000 1
xxxx)	770	0.000 0
xxxxi)	780	0.000 0

NOTE — Normalized relative spectral distribution D_λ of illuminant D65 multiplied by the spectral luminous efficiency $V(\lambda)$ and by the wavelength interval $\Delta\lambda$. The values in this table are calculated according to the trapezoidal rule.

5.5 Light Reflectance

5.5.1 *External Light Reflectance of Glazing*

External light reflectance corresponds to the amount of sunlight reflected by the external glass surface. The amount of light reflection depends on the type of base glass (substrate tint) and the position of coating on the glass surface.

The external light reflectance of glazing $\rho_{v,o}$ shall be calculated using the following formula:

$$\rho_{v,o} = \frac{\sum_{\lambda=380\text{ nm}}^{780\text{ nm}} \rho_o(\lambda) D_\lambda V(\lambda)\,\Delta\lambda}{\sum_{\lambda=380\text{ nm}}^{780\text{ nm}} D_\lambda V(\lambda)\Delta\lambda}$$

where

$\rho_o(\lambda)$ = spectral external reflectance of glazing, and D_λ, $V(\lambda)$, $\Delta\lambda$ and the integration procedure are defined in 5.4.5.4.

For multiple glazing, the calculation of the spectral external reflectance $\rho_o(\lambda)$ shall be performed using the same methods as given in 5.4.5.4 for the calculation of the spectral transmittance $\tau(\lambda)$.

For the spectral external reflectance $\rho_o(\lambda)$ as a function of the spectral characteristics of the individual components of the unit, the following formulae are applied:

a) For double glazing:

$$\rho_o(\lambda) = \rho_1(\lambda) + \frac{\tau_1^2(\lambda)\,\rho_2(\lambda)}{1 - \rho_1'(\lambda)\,\rho_2(\lambda)}$$

b) For triple glazing:

$$\rho_o(\lambda) = \rho_1(\lambda) + \frac{\tau_1^2(\lambda)\,\rho_2(\lambda)[1 - \rho_2'(\lambda)\,\rho_3(\lambda)] + \tau_1^2(\lambda)\,\tau_2^2(\lambda)\,\rho_3(\lambda)}{[1 - \rho_1'(\lambda)\,\rho_2(\lambda)].[1 - \rho_2'(\lambda)\,\rho_3(\lambda)] - \tau_2^2(\lambda)\rho_1'(\lambda)\,\rho_3(\lambda)}$$

For multiple glazing with more than three components, relationships similar to above equations are found to calculate $\rho_o(\lambda)$ of such glazing from the spectral characteristics of the individual components.

As an example for calculating $\rho_o(\lambda)$, a glazing composed of five components may be treated in the same way as described in 5.4.5.4.

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5.5.2 *Internal Light Reflectance of Glazing*

Internal light reflectance corresponds to the amount of light reflected by the inner glass surface. The amount of light reflection depends on the type of base glass (substrate tint) and the position of coating on the glass surface.

The internal light reflectance of glazing $\rho_{v,i}$ shall be calculated using the following formula:

$$\rho_{v,i} = \frac{\sum_{\lambda=380\text{ nm}}^{780\text{ nm}} \rho_i(\lambda) D_\lambda V(\lambda)\,\Delta\lambda}{\sum_{\lambda=380\text{ nm}}^{780\text{ nm}} D_\lambda V(\lambda)\Delta\lambda}$$

where

$\rho_i(\lambda)$ = spectral internal reflectance of glazing; and

$D\lambda$, $V(\lambda)$, $\Delta\lambda$ and the integration procedure are as defined in light transmittance in 5.4.5.4.

For multiple glazing, the calculation of the spectral internal reflectance $\rho_i(\lambda)$ shall be performed using the same methods as given in 5.4.5.4 for the calculation of the spectral transmittance $\tau(\lambda)$.

For the spectral internal reflectance $\rho_i(\lambda)$ as a function of the spectral characteristics of the individual components of the unit, the following formulae are applied.

b) *For double glazing:*

$$\rho_i(\lambda) = \rho_2'(\lambda) + \frac{\tau_2^2(\lambda)\,\rho_1'(\lambda)}{1 - \rho_1'(\lambda)\,\rho_2(\lambda)}$$

c) *For triple glazing:*

$$\rho_i(\lambda) = \rho_3'(\lambda) + \frac{\tau_3^2(\lambda)\,\rho_2'(\lambda)[1 - \rho_2(\lambda)\rho_1'(\lambda)] + \tau_3^2(\lambda)\,\tau_2^2(\lambda)\rho_1'(\lambda)}{[1 - \rho_3(\lambda)\,\rho_2'(\lambda)].[1 - \rho_2(\lambda)\rho_1'(\lambda)] - \tau_2^2(\lambda)\,\rho_3(\lambda)\rho_1'(\lambda)}$$

For multiple glazing with more than three components, relationships similar to above expressions are found to calculate $\rho_i(\lambda)$ of such glazing from the spectral characteristics of the individual components.

5.6 Energy Design Considerations

5.6.1 *General*

Glass and glazing system design shall enhance daylighting into the building, improve the views to outdoor environments for the occupants, minimize the energy gains in hot climates, energy losses in cold climates and facilitate air movement in naturally ventilated spaces.

Fenestration, glazing and shading systems shall be designed by considering the climate characteristics, solar geometry, design sky conditions, orientation of the facade and building use. Effective design of the glazing system can minimize energy gains during summer and energy losses during winter. Height of the vision panel and the lintel height can be effectively adjusted to optimize the daylight penetration into the building.

5.6.2 *Design factors*

The following factors/components of a glazing system shall be considered during design for achieving energy efficiency:

a) Light transmittance and glare,

b) Thermal transmittance (solar heat gain co-efficient and U-value),

c) Fenestration size and placement,

d) Thermal properties of the frames,

e) Shading devices, and

f) Air infiltration.

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5.6.3 *Analysis for Glazing System Selection*

5.6.3.1 *Building orientation and solar geometry analysis*

Optimal building orientation is a critical aspect for climate responsive design. Knowledge of the solar geometry at a site is essential for designing building facades to regulate energy and daylight transmission as well as reduce glare and overheating inside the building. Choice of glass and the shading system can considerably vary based on orientation of the facade and the solar geometry. A well oriented building maximises daylighting through its facades, thereby reducing the need for artificial lighting. Such building regulates the energy gains and losses through the facade, thereby minimizing thermal discomfort and cooling and heating energy demand.

5.6.3.2 *Solar insolation and shading analysis*

Design of glazing system is driven by the insolation levels on the facade, the desirable energy gains/losses and daylight requirements for a given building. Solar insolation varies with geographic location, orientation of the facade, time of the day and season. Direct solar incident radiation is the energy transmitted from the sun, which lies between the wavelength range of 250 nm and 2 500 nm. Solar radiation typically consists of 3 percent ultraviolet light, 42 percent visible light, and 55 percent infrared light. Infrared radiation is the major heat carrying radiation, followed by visible light. Analysing solar radiation helps to assess its impact on the building's elevations, allowing for a better understanding of how this influences the building's performance. Solar insolation and shading analysis are employed for determining optimum SHGC and U-value of glass, U-value of the frame and the design of shading system necessary for minimizing the energy gains and losses through the facade.

5.6.3.3 *Daylight analysis*

Daylight analysis is used to assess the penetration of natural light on the building floor plan or at the working plane level. A good lighting strategy involves optimizing the glazed area, increasing the daylight autonomy and thereby reducing the dependency on artificial lighting. Using a glass with optimal light transmission can go a long way in curtailing the dependence on artificial lighting. Additionally, the presence of natural light alleviates the claustrophobic feeling often associated with buildings that have limited glazing and outdoor views. Daylight calculations are conducted for overcast conditions to represent the worst-case scenario for outdoor lighting levels. Detailed methods for daylight analysis are provided in good practice [C-8(13)].

Any glass with enhanced energy performance will inevitably reduce the amount of daylight transmitted, even though it remains transparent for outward vision. The higher the demand for energy performance, the greater the compromise on daylight. Therefore, the selection of glass should begin by assessing the daylight requirements and deciding the optimum visible light transmittance (VLT). This approach helps narrow down the options, allowing for the selection of glass that provides the best performance for the specified VLT while also considering cost and aesthetics.

Floor plate geometry plays an important role as the daylight distribution will depend on the depth of the plate from the glazed area. Planforms incorporating courtyard and atrium have a great advantage of harnessing maximum daylight.

5.6.4 *Integrated Design Approach*

Glass and glazing system design shall duly consider the trade-offs in enhancing daylight availability, maximizing or minimizing the heat gains/losses, reduction in glare and improving views for the occupants. Table 12 provides the relation between glazing system parameters and different performance attributes.

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Table 12 Relation Between Glazing System Parameters and Different Performance Attributes

(*Clause* 5.6.4)

Sl No.	Performance Attributes	Glazing System Parameters				
		VLT	U-Value	SHGC	Shading system	Fenestration size
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	Daylight	$\propto$	–	ID	$\frac{1}{\propto}$	$\propto$
ii)	View	$\propto$	–	ID	$\frac{1}{\propto}$	$\propto$
iii)	Heat transfer	–	$\propto$	$\propto$	$\frac{1}{\propto}$	$\propto$
iv)	Ventilation	–	–	–	$\frac{1}{\propto}$	$\propto$

$\propto$: directly proportional; $\frac{1}{\propto}$: Inversely proportional; ID: Indirect relationship

NOTE — The effects may vary depending on the geographic location, orientation of the facade and on-site and off-site features. The factors also have inter-dependencies.

5.7 Window Design Guideline for Non-Conditioned/Mixed Mode Buildings

In non-conditioned spaces, windows typically use single-glazed units, as they are often opened to allow ventilation. In such buildings, shading devices play a critical role in the thermal performance of windows. Considerations for selecting type of window and the necessary design considerations for energy efficiency and natural ventilation may be made in accordance with the relevant guidelines/best practices (Handbook on 'Sustainability in Buildings').

NOTE — Air infiltration Class B in accordance with accepted standard [C-8(14)] is adequate for non-conditioned buildings.

6 STRUCTURAL PARAMETERS AND FIRE SAFETY

6.1 Determination of Appropriate Glass Thickness

6.1.1 *General*

This Section gives provisions to determine the minimum glass thickness to be used to resist the ultimate limit state design wind pressures. However, the location of the glass within the building require additional considerations, such as safety with respect to human which may impose either a minimum glass thickness above the basic calculation of this part or a provision for a specific glass type or both.

In addition to the provisions herein, the design requirements such as wind loads determined in accordance with good practice [C-8(15)] and the serviceability deflection requirement limit given in this Section shall have to be complied.

The provisions under 6 are applicable provided the following parameters are satisfied:

a) Ultimate limit state wind pressure shall not be greater than 10 kPa;

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b) For laminated glass, the two sheets may be of equal or unequal thickness and the interlayer material is either polyvinyl butyral or an equivalent type of interlayer with a modulus of elasticity of about 24 MPa and a Poisson's ratio of 0.50 at 20 °C;

c) Design flexural tensile strength of glass depends upon the design strength obtained after applying a factor of safety of 2.5. The minimum design strength of normal glass for thickness up to 6 mm is 16.7 N/mm^2 and for thickness above 6 mm is 15.2 N/mm^2.

6.1.2 *Design Considerations*

The following design considerations shall be taken into account in determination of appropriate thickness of glass:

a) Maximum area of glass panel is restricted to 15 m^2;

b) Maximum span of glass panel is restricted to 4 m;

c) Aspect ratio of the glass panel should be greater than 1.5. If it is less than 1.5, next higher available thickness should be selected in order to reduce the stress level;

d) Factor of safety applied is 2.5 considering the variability in strength of glass;

e) Applicable to normal, reflective, laminated, tempered and insulating glass;

f) Applicable to rectangular panels properly secured;

g) Design minimum thickness of laminated glass will be the maximum value of thickness in accordance with Table 2 of accepted standard [C-8(6)]; and

h) The parameters listed in (a) to (g) are valid for calculating glass thickness using the empirical relationship in **6.1.3**. For glass sizes larger than those mentioned, any recognized method of analysis may be used for design, in consultation with the manufacturer and the expert in the field.

6.1.3 *Empirical Relationship*

Empirical relation between the wind pressure, area of the glass panel and the required glass thickness can be used:

$$P_{net}.A = 200T^k \text{ (for } T \leq 6 \text{ mm)}$$

$$P_{net}.A = 200T^k + 1\,900 \text{ (for } T > 6 \text{ mm)}$$

where

P_{net} = Net design wind pressure as per **6.1.4**, in N/m^2;

A = Area of glass panel, in m^2;

T = Standard nominal thickness (SNT) of glass from Table 13, in mm; and

k = Constant from the Table 13.

Table 13 Value of k for the Corresponding Standard Nominal Thickness of Float Glass

(*Clause* 6.1.3)

T, mm	3	4	5	6	8	10	12	15	19	25
k	1.683	1.732	1.753	1.765	1.57	1.578	1.583	1.579	1.569	1.569

6.1.3.1 *Glass supported on two opposite sides*

a) Normal and laminated glass panels supported on two opposite sides can be designed using following empirical relations:

1) For $T \leq 6$ mm :

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$$b = \frac{4.39\,T}{\sqrt{\left(\frac{P_{net}}{P_f}\right)}}$$

2) For $T > 6$ mm :

$$b = \frac{4.22\,T}{\sqrt{\left(\frac{P_{net}}{P_f}\right)}}$$

b) Tempered/toughened panel supported on two opposite sides can be designed using following empirical relations:

1) For $T \leq 6$ mm :

$$b = \frac{3.268\,8\,T}{\sqrt{\left(\frac{P_{net}}{P_f}\right)}}$$

2) For $T > 6$ mm :

$$b = \frac{2.906\,9\,T}{\sqrt{\left(\frac{P_{net}}{P_f}\right)}}$$

where

b = span, in m; and

P_f = strength factors as in Table 14.

For insulated glass, thickness of only one glass pane shall be considered. If the glass panes are of different thickness, the minimum of the two thicknesses shall be considered.

For laminated glass, the thickness of PVB has not been considered. The value to be used is:

T = Standard nominal thickness —Thickness of PVB.

6.1.3.2 For non-linear analysis, specialist literature may be consulted for various shapes, sizes, and types of support conditions of glass.

6.1.4 *Determination of Design Wind Pressure*

Net design wind pressure (P_{net}) is an important parameter governing the thickness of glass to be used in the window panels. It depends on factors as given below:

a) Location of the building (wind zone),

b) Construction patterns around buildings (terrain category),

c) Topography of site,

d) Building plan and height, etc.

Net design wind pressure (P_{net}), may be defined using the following equation:

$$P_{net} = p_z.C_p$$

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where

C_p= net pressure coefficient as per good practice [C-8(15)]; and

p_z = design wind pressure as per good practice [C-8(15)], in N/m^2.
= 0.6 V_z^2

$V_z = V_b.k_1.k_2.k_3.k_4$

where

V_b = basic wind speed based on location {as per Fig. 1 of good practice [C-8(15)]};

k_1 = risk co-efficient factor;

k_2 = terrain factor;

k_3 = topography factor;

k_4 = importance factor for the cyclonic region; and

V_z = design wind speed, in m/s.

6.1.4.1 *Laminated/tempered/insulating glass*

To determine the thickness of laminated/ tempered/ insulating glass, the design wind pressure P_{net} is modified by dividing it with the strength or pressure factor P_f, dependent on the type of glass. The values of the P_f are given in Table 14.

Using the modified values of P_{net}, the thickness of other glass types can be obtained in accordance with **6.1.3**.

Table 14 Strength Factor, P_f

[*Clauses* 6.1.3.1(b), 6.1.4.1, 6.3.5 *and* E-1.1.1]

Sl No.	Glass Type	P_f
(1)	(2)	(3)
i)	Normal (Annealed)	1.00
ii)	Laminated	0.80
iii)	Tempered	2.50
iv)	Insulated	1.50
v)	Heat strengthened	1.60
vi)	Wired	0.50

6.1.5 *Aspect Ratio* (AR_{max})

The design of the thickness using empirical relation in accordance with **6.1.3** shall be valid up to a limiting aspect ratio AR_{max}. The value of AR_{max} for different thickness (T) of glass is given in Table 15.

Table 15 Thickness and Corresponding AR_{max} Values (*Clause* 6.1.5 *and* E-1.1.1)

T, in mm	3	4	5	6	8	10	12	15	19	25
AR_{max}	7.3	6.8	6.5	6.3	5.9	4.9	4.3	3.8	3.3	2.9

6.1.6 *Seismic Design*

6.1.6.1 *Seismic load* (*EQ*)

The design for seismic forces shall be performed in steps to establish upper and lower bound force and movement response parameters. In addition, the response of the structure sensitive to certain parameters such as the damping and the stiffness of the support points may be evaluated.

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The principal design criteria for the seismic design are,

a) Structure will remain elastic during the design seismic event.

b) 'Hangers' will stiffen the cantilever 'tree' structure.

c) 'Hangers' will not go slack.

6.1.6.2 *Architectural elements and utilities*

Depending on response sensitivity, architectural elements and utilities can be classified as deformation sensitive, acceleration sensitive, or both deformation and acceleration sensitive. Table 16 classifies non-structural elements according to their response sensitivity.

Table 16 Response Sensitivity of Architectural Component

(*Clause* 6.1.6.2)

Sl No.	Component	Sensitivity	
		Acceleration Sensitive	Deformation Sensitive
(1)	(2)	(3)	(4)
i)	Exterior skin:		
	a) Adhered veneer	S	P
	b) Anchored veneer	S	P
	c) Glass blocks	S	P
	d) Prefabricated panel	S	P
	e) Glazing system	S	P
ii)	Partitions:		
	a) Heavy	S	P
	b) Light	S	P
iii)	Interior veneers:		
	a) Stone, including marble	S	P
	b) Ceramic tile	S	P
iv)	Parapets and appendages	P	—
v)	Canopies and marquees	P	—
vi)	Stairs	P	S

NOTES

1 P = Primary response

2 S = Secondary response

6.1.6.3 *Design seismic force*

Design seismic force on the architectural elements and utilities shall be calculated as per Part C 'Structural Design, Section 1 Loads and Load Effects' of this NBCS.

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6.1.6.4 *Seismic relative displacement*

a) For two connection points on the same structure *A*, one at a height h_x, and other at a height h_y, seismic relative displacement shall be determined as:

$$D_p = \delta_{xA} - \delta_{yA}$$

D_p is not required to be taken as greater than $R(h_x - h_y)\frac{\Delta_{aA}}{h_{sx}}$

where

δ_{xA} = deflection at building level x of structure A due to design seismic load determined by elastic analysis, and multiplied by response reduction factor R of the building as per Table 9 of good practice [C-8(16)];

δ_{yA} = deflection at building level y of structure A due to design seismic load determined by elastic analysis, and multiplied by response reduction factor R of the building as per Table 9 of good practice [C-8(16)];

h_x = height of level x to which upper connection point is attached;

h_y = height of level y to which lower connection point is attached;

Δ_{aA} = allowable storey drift for structure A calculated as per **7.11.1.1** of good practice [C-8(16)]; and

h_{sx} = storey height below level x.

b) For two connection points on separate structures A and B, or separate structural systems, one at height, h_x and the other at a height, h_y , D_p shall be determined as:

$$D_p = |\delta_{xA}| + |\delta_{yA}|$$

D_p is not required to be taken as greater than

$$R\left(h_x \frac{\Delta_{aA}}{h_{sx}} + h_y \frac{\Delta_{aB}}{h_{sx}}\right)$$

where

δ_{yB} = deflection at building level y of structure *B* due to design seismic load determined by elastic analysis, and multiplied by response reduction factor, R, of the building as per Table 9 of good practice [C-8(16)]; and

Δ_{aB} = allowable storey drift for structure *B* calculated as per **7.11.1.1** of good practice [C-8(16)].

6.1.7 *Imposed Loads*

The uniform distributed load and point load is to be considered based on application and is to be selected in accordance with good practice [C-8(17)]. Appropriate uniform distributed load (UDL) and concentrated load are selected in accordance to good practice [C-8(17)] based on nature of building (for example, residential, institutional, industrial building, etc) and nature of activity under each building type.

6.1.8 *Load Contribution*

6.1.8.1 *Wind load*

The wind load is the effect of wind pressure which is dependent on the location, building height, topography and ground roughness factor. The wind speed is taken from good practice [C-8(15)] for the geographic location.

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6.1.8.2 *Dead load*

In case of inclined sloped glazing, the dead weight of the glazing should be considered. In case of component of sloped glazing, the following corresponding component of the total glazing may be used:

a) Point load shall be applied as a uniformly distributed load over a circular area of 0.01 m^2;

b) For a glazed panel supported on all edges, the point loading shall be applied at the centre;

c) For free glazing edges, the point load shall be applied adjacent to the centre of the free edge; and

d) Ultimate limit state design wind pressure shall not be greater than 1.2 kPa. For greater design wind pressures, the glass shall be separately designed to withstand wind loading in accordance with **6.1.4**.

6.1.9 *Reference Chart Relating Wind Load, Maximum Allowable Area for a Thickness of Glass with Respect to Terrain Category*

Table 17 to Table 25 provides relation between design wind pressure and maximum allowable area for a thickness of glass with respect to specific terrain category. User may select the type of glass depending on the support conditions.

Specialist literature may be consulted for use of non-linear analysis.

NOTE — Specialist literature shall mean any national/international publication, codes or peer reviewed research papers on the topic.

The deflection at the centre of glass pane can be calculated using the formula in accordance to Annex D.

Typical model calculation to calculate thickness and determine wind load and deflection at the centre of the glazing is given at Annex E.

Table 17 Maximum Areas for Annealed Glass Fixed on All Four Sides, m^2

(*Clauses* 6.1.9 *and* 6.3.6.1)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
i)	500	2.13	3.86	6.03	8.63	13.06	15.00	15.00	15.00	15.00
ii)	550	1.93	3.51	5.48	7.85	11.88	15.00	15.00	15.00	15.00
iii)	600	1.77	3.21	5.02	7.19	10.89	14.61	15.00	15.00	15.00
iv)	650	1.64	2.97	4.64	6.64	10.05	13.48	15.00	15.00	15.00
v)	700	1.52	2.75	4.31	6.17	9.33	12.52	15.00	15.00	15.00
vi)	750	1.42	2.57	4.02	5.76	8.71	11.69	14.75	15.00	15.00
vii)	800	1.33	2.41	3.77	5.40	8.16	10.96	13.83	15.00	15.00
viii)	850	1.25	2.27	3.55	5.08	7.68	10.31	13.01	15.00	15.00
ix)	900	1.18	2.14	3.35	4.80	7.26	9.74	12.29	15.00	15.00
x)	950	1.12	2.03	3.17	4.54	6.88	9.23	11.64	15.00	15.00
xi)	1 000	1.06	1.93	3.01	4.32	6.53	8.76	11.06	15.00	15.00
xii)	1 050	1.01	1.84	2.87	4.11	6.22	8.35	10.53	14.38	15.00
xiii)	1 100	0.97	1.75	2.74	3.92	5.94	7.97	10.06	13.73	15.00
xiv)	1 150	0.93	1.68	2.62	3.75	5.68	7.62	9.62	13.13	15.00

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Table 17 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
xv)	1 200	0.89	1.61	2.51	3.60	5.44	7.30	9.22	12.58	15.00
xvi)	1 250	0.85	1.54	2.41	3.45	5.23	7.01	8.85	12.08	15.00
xvii)	1 300	0.82	1.48	2.32	3.32	5.02	6.74	8.51	11.61	15.00
xviii)	1 350	0.79	1.43	2.23	3.20	4.84	6.49	8.19	11.18	14.98
xix)	1 400	0.76	1.38	2.15	3.08	4.67	6.26	7.90	10.78	14.44
xx)	1 450	0.73	1.33	2.08	2.98	4.50	6.04	7.63	10.41	13.95
xxi)	1 500	0.71	1.29	2.01	2.88	4.35	5.84	7.37	10.07	13.48
xxii)	1 550	0.69	1.24	1.94	2.78	4.21	5.65	7.14	9.74	13.05
xxiii)	1 600	0.67	1.21	1.88	2.70	4.08	5.48	6.91	9.44	12.64
xxiv)	1 650	0.64	1.17	1.83	2.62	3.96	5.31	6.70	9.15	12.25
xxv)	1 700	0.63	1.13	1.77	2.54	3.84	5.16	6.51	8.88	11.89
xxvi)	1 750	0.61	1.10	1.72	2.47	3.73	5.01	6.32	8.63	11.55
xxvii)	1 800	0.59	1.07	1.67	2.40	3.63	4.87	6.14	8.39	11.23
xxviii)	1 850	0.58	1.04	1.63	2.33	3.53	4.74	5.98	8.16	10.93
xxix)	1 900	0.56	1.01	1.59	2.27	3.44	4.61	5.82	7.95	10.64
xxx)	1 950	0.55	0.99	1.55	2.21	3.35	4.49	5.67	7.74	10.37
xxxi)	2 000	0.53	0.96	1.51	2.16	3.27	4.38	5.53	7.55	10.11
xxxii)	2 050	0.52	0.94	1.47	2.11	3.19	4.28	5.40	7.36	9.86
xxxiii)	2 100	0.51	0.92	1.44	2.06	3.11	4.17	5.27	7.19	9.63
xxxiv)	2 150	0.49	0.90	1.40	2.01	3.04	4.08	5.14	7.02	9.40
xxxv)	2 200	0.48	0.88	1.37	1.96	2.97	3.98	5.03	6.86	9.19
xxxvi)	2 250	0.47	0.86	1.34	1.92	2.90	3.90	4.92	6.71	8.99
xxxvii)	2 300	0.46	0.84	1.31	1.88	2.84	3.81	4.81	6.56	8.79
xxxviii)	2 350	0.45	0.82	1.28	1.84	2.78	3.73	4.71	6.42	8.60
xxxix)	2 400	0.44	0.80	1.26	1.80	2.72	3.65	4.61	6.29	8.43
xl)	2 450	0.43	0.79	1.23	1.76	2.67	3.58	4.51	6.16	8.25
xli)	2 500	0.43	0.77	1.21	1.73	2.61	3.51	4.42	6.04	8.09
xlii)	2 550	0.42	0.76	1.18	1.69	2.56	3.44	4.34	5.92	7.93
xliii)	2 600	0.41	0.74	1.16	1.66	2.51	3.37	4.25	5.81	7.78
xliv)	2 650	0.40	0.73	1.14	1.63	2.46	3.31	4.17	5.70	7.63
xlv)	2 700	0.39	0.71	1.12	1.60	2.42	3.25	4.10	5.59	7.49
xlvi)	2 750	0.39	0.70	1.10	1.57	2.38	3.19	4.02	5.49	7.35
xlvii)	2 800	0.38	0.69	1.08	1.54	2.33	3.13	3.95	5.39	7.22

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Table 17 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
xlviii)	2 850	0.37	0.68	1.06	1.51	2.29	3.08	3.88	5.30	7.09
xlix)	2 900	0.37	0.66	1.04	1.49	2.25	3.02	3.81	5.21	6.97
l)	2 950	0.36	0.65	1.02	1.46	2.21	2.97	3.75	5.12	6.85
li)	3 000	0.35	0.64	1.00	1.44	2.18	2.92	3.69	5.03	6.74
lii)	3 050	0.35	0.63	0.99	1.42	2.14	2.87	3.63	4.95	6.63
liii)	3 100	0.34	0.62	0.97	1.39	2.11	2.83	3.57	4.87	6.52
liv)	3 150	0.34	0.61	0.96	1.37	2.07	2.78	3.51	4.79	6.42
lv)	3 200	0.33	0.60	0.94	1.35	2.04	2.74	3.46	4.72	6.32
lvi)	3 250	0.33	0.59	0.93	1.33	2.01	2.70	3.40	4.65	6.22
lvii)	3 300	0.32	0.58	0.91	1.31	1.98	2.66	3.35	4.58	6.13
lviii)	3 350	0.32	0.58	0.90	1.29	1.95	2.62	3.30	4.51	6.04
lix)	3 400	0.31	0.57	0.89	1.27	1.92	2.58	3.25	4.44	5.95
lx)	3 450	0.31	0.56	0.87	1.25	1.89	2.54	3.21	4.38	5.86
lxi)	3 500	0.30	0.55	0.86	1.23	1.87	2.50	3.16	4.31	5.78
lxii)	3 550	0.30	0.54	0.85	1.22	1.84	2.47	3.12	4.25	5.70
lxiii)	3 600	0.30	0.54	0.84	1.20	1.81	2.43	3.07	4.19	5.62
lxiv)	3 650	0.29	0.53	0.83	1.18	1.79	2.40	3.03	4.14	5.54
lxv)	3 700	0.29	0.52	0.81	1.17	1.77	2.37	2.99	4.08	5.47
lxvi)	3 750	0.28	0.51	0.80	1.15	1.74	2.34	2.95	4.03	5.39
lxvii)	3 800	0.28	0.51	0.79	1.14	1.72	2.31	2.91	3.97	5.32
lxviii)	3 850	0.28	0.50	0.78	1.12	1.70	2.28	2.87	3.92	5.25
lxix)	3 900	0.27	0.49	0.77	1.11	1.67	2.25	2.84	3.87	5.18
lxx)	3 950	0.27	0.49	0.76	1.09	1.65	2.22	2.80	3.82	5.12
lxxi)	4 000	0.27	0.48	0.75	1.08	1.63	2.19	2.77	3.77	5.06
lxxii)	4 050	0.26	0.48	0.74	1.07	1.61	2.16	2.73	3.73	4.99
lxxiii)	4 100	0.26	0.47	0.74	1.05	1.59	2.14	2.70	3.68	4.93
lxxiv)	4 150	0.26	0.46	0.73	1.04	1.57	2.11	2.67	3.64	4.87
lxxv)	4 200	0.25	0.46	0.72	1.03	1.56	2.09	2.63	3.59	4.81
lxxvi)	4 250	0.25	0.45	0.71	1.02	1.54	2.06	2.60	3.55	4.76
lxxvii)	4 300	0.25	0.45	0.70	1.00	1.52	2.04	2.57	3.51	4.70
lxxviii)	4 350	0.24	0.44	0.69	0.99	1.50	2.01	2.54	3.47	4.65
lxxix)	4 400	0.24	0.44	0.69	0.98	1.48	1.99	2.51	3.43	4.60
lxxx)	4 450	0.24	0.43	0.68	0.97	1.47	1.97	2.49	3.39	4.54

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Table 17 (*Concluded*)

Sl No.	Design Wind Pressure N/m²	Nominal Thickness of Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
lxxxi)	4 500	0.24	0.43	0.67	0.96	1.45	1.95	2.46	3.36	4.49
lxxxii)	4 550	0.23	0.42	0.66	0.95	1.44	1.93	2.43	3.32	4.44
lxxxiii)	4 600	0.23	0.42	0.66	0.94	1.42	1.91	2.40	3.28	4.40
lxxxiv)	4 650	0.23	0.41	0.65	0.93	1.40	1.88	2.38	3.25	4.35
lxxxv)	4 700	0.23	0.41	0.64	0.92	1.39	1.86	2.35	3.21	4.30
lxxxvi)	4 750	0.22	0.41	0.63	0.91	1.38	1.85	2.33	3.18	4.26
lxxxvii)	4 800	0.22	0.40	0.63	0.90	1.36	1.83	2.30	3.15	4.21
lxxxviii)	4 850	0.22	0.40	0.62	0.89	1.35	1.81	2.28	3.11	4.17
lxxxix)	4 900	0.22	0.39	0.62	0.88	1.33	1.79	2.26	3.08	4.13
xc)	4 950	0.21	0.39	0.61	0.87	1.32	1.77	2.23	3.05	4.08
xci)	5 000	0.21	0.39	0.60	0.86	1.31	1.75	2.21	3.02	4.04
xcii)	Maximum aspect ratio	7.3	6.8	6.5	6.3	5.9	4.9	4.3	3.8	3.3

Table 18 Maximum Areas for Laminated Glass Fixed on All Four Sides, m²

(*Clauses* 6.1.9, 6.3.5 *and* 6.3.6.1)

Sl No.	Design Wind Pressure N/m²	Nominal Thickness of Laminated Glass, *T* mm					
		5.38	6.38	8.38	10.38	12.38	16.38
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	500	4.47	6.28	10.45	14.02	15.00	15.00
ii)	550	4.06	5.71	9.50	12.75	15.00	15.00
iii)	600	3.72	5.23	8.71	11.69	15.00	15.00
iv)	650	3.44	4.83	8.04	10.79	13.93	15.00
v)	700	3.19	4.48	7.46	10.02	12.94	15.00
vi)	750	2.98	4.19	6.97	9.35	12.08	15.00
vii)	800	2.79	3.92	6.53	8.76	11.32	15.00
viii)	850	2.63	3.69	6.15	8.25	10.66	15.00
ix)	900	2.48	3.49	5.81	7.79	10.06	14.21

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Table 18 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Laminated Glass, *T* mm					
		5.38	6.38	8.38	10.38	12.38	16.38
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
x)	950	2.35	3.30	5.50	7.38	9.53	13.46
xi)	1 000	2.23	3.14	5.23	7.01	9.06	12.79
xii)	1 050	2.13	2.99	4.98	6.68	8.63	12.18
xiii)	1 100	2.03	2.85	4.75	6.37	8.23	11.63
xiv)	1 150	1.94	2.73	4.54	6.10	7.88	11.12
xv)	1 200	1.86	2.62	4.35	5.84	7.55	10.66
xvi)	1 250	1.79	2.51	4.18	5.61	7.25	10.23
xvii)	1 300	1.72	2.41	4.02	5.39	6.97	9.84
xviii)	1 350	1.66	2.33	3.87	5.19	6.71	9.47
xix)	1 400	1.60	2.24	3.73	5.01	6.47	9.14
xx)	1 450	1.54	2.16	3.60	4.84	6.25	8.82
xxi)	1 500	1.49	2.09	3.48	4.67	6.04	8.53
xxii)	1 550	1.44	2.03	3.37	4.52	5.84	8.25
xxiii)	1 600	1.40	1.96	3.27	4.38	5.66	7.99
xxiv)	1 650	1.35	1.90	3.17	4.25	5.49	7.75
xxv)	1 700	1.31	1.85	3.07	4.12	5.33	7.52
xxvi)	1 750	1.28	1.79	2.99	4.01	5.18	7.31
xxvii)	1 800	1.24	1.74	2.90	3.90	5.03	7.11
xxviii)	1 850	1.21	1.70	2.82	3.79	4.90	6.91
xxix)	1 900	1.18	1.65	2.75	3.69	4.77	6.73
xxx)	1 950	1.15	1.61	2.68	3.60	4.64	6.56
xxxi)	2 000	1.12	1.57	2.61	3.51	4.53	6.40
xxxii)	2 050	1.09	1.53	2.55	3.42	4.42	6.24
xxxiii)	2 100	1.06	1.49	2.49	3.34	4.31	6.09
xxxiv)	2 150	1.04	1.46	2.43	3.26	4.21	5.95
xxxv)	2 200	1.02	1.43	2.38	3.19	4.12	5.81
xxxvi)	2 250	0.99	1.40	2.32	3.12	4.03	5.68
xxxvii)	2 300	0.97	1.36	2.27	3.05	3.94	5.56

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Table 18 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Laminated Glass, *T* mm					
		5.38	6.38	8.38	10.38	12.38	16.38
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
xxxviii)	2 350	0.95	1.34	2.22	2.98	3.85	5.44
xxxix)	2 400	0.93	1.31	2.18	2.92	3.77	5.33
xl)	2 450	0.91	1.28	2.13	2.86	3.70	5.22
xli)	2 500	0.89	1.26	2.09	2.80	3.62	5.12
xlii)	2550	0.88	1.23	2.05	2.75	3.55	5.02
xliii)	2 600	0.86	1.21	2.01	2.70	3.48	4.92
xliv)	2 650	0.84	1.18	1.97	2.65	3.42	4.83
xlv)	2 700	0.83	1.16	1.94	2.60	3.35	4.74
xlvi)	2 750	0.81	1.14	1.90	2.55	3.29	4.65
xlvii)	2 800	0.80	1.12	1.87	2.50	3.23	4.57
xlviii)	2 850	0.78	1.10	1.83	2.46	3.18	4.49
xlix)	2 900	0.77	1.08	1.80	2.42	3.12	4.41
l)	2 950	0.76	1.06	1.77	2.38	3.07	4.34
li)	3 000	0.74	1.05	1.74	2.34	3.02	4.26
lii)	3 050	0.73	1.03	1.71	2.30	2.97	4.19
liii)	3 100	0.72	1.01	1.69	2.26	2.92	4.13
liv)	3 150	0.71	1.00	1.66	2.23	2.88	4.06
lv)	3 200	0.70	0.98	1.63	2.19	2.83	4.00
lvi)	3 250	0.69	0.97	1.61	2.16	2.79	3.94
lvii)	3 300	0.68	0.95	1.58	2.12	2.74	3.88
lviii)	3 350	0.67	0.94	1.56	2.09	2.70	3.82
lix)	3 400	0.66	0.92	1.54	2.06	2.66	3.76
lx)	3 450	0.65	0.91	1.51	2.03	2.63	3.71
lxi)	3 500	0.64	0.90	1.49	2.00	2.59	3.65
lxii)	3 550	0.63	0.88	1.47	1.98	2.55	3.60
lxiii)	3 600	0.62	0.87	1.45	1.95	2.52	3.55
lxiv)	3 650	0.61	0.86	1.43	1.92	2.48	3.50
lxv)	3 700	0.60	0.85	1.41	1.90	2.45	3.46

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Table 18 (*Concluded*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Laminated Glass, *T* mm					
		5.38	6.38	8.38	10.38	12.38	16.38
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
lxvi)	3 750	0.60	0.84	1.39	1.87	2.42	3.41
lxvii)	3 800	0.59	0.83	1.38	1.85	2.38	3.37
lxviii)	3 850	0.58	0.82	1.36	1.82	2.35	3.32
lxix)	3 900	0.57	0.80	1.34	1.80	2.32	3.28
lxx)	3 950	0.57	0.79	1.32	1.78	2.29	3.24
lxxi)	4 000	0.56	0.78	1.31	1.75	2.26	3.20
lxxii)	4 050	0.55	0.78	1.29	1.73	2.24	3.16
lxxiii)	4 100	0.55	0.77	1.27	1.71	2.21	3.12
lxxiv)	4 150	0.54	0.76	1.26	1.69	2.18	3.08
lxxv)	4 200	0.53	0.75	1.24	1.67	2.16	3.05
lxxvi)	4 250	0.53	0.74	1.23	1.65	2.13	3.01
lxxvii)	4 300	0.52	0.73	1.22	1.63	2.11	2.97
lxxviii)	4 350	0.51	0.72	1.20	1.61	2.08	2.94
lxxix)	4 400	0.51	0.71	1.19	1.59	2.06	2.91
lxxx)	4 450	0.50	0.71	1.17	1.58	2.04	2.87
lxxxi)	4 500	0.50	0.70	1.16	1.56	2.01	2.84
lxxxii)	4 550	0.49	0.69	1.15	1.54	1.99	2.81
lxxxiii)	4 600	0.49	0.68	1.14	1.52	1.97	2.78
lxxxiv)	4 650	0.48	0.68	1.12	1.51	1.95	2.75
lxxxv)	4 700	0.48	0.67	1.11	1.49	1.93	2.72
lxxxvi)	4 750	0.47	0.66	1.10	1.48	1.91	2.69
lxxxvii)	4 800	0.47	0.65	1.09	1.46	1.89	2.66
lxxxviii)	4 850	0.46	0.65	1.08	1.45	1.87	2.64
lxxxix)	4 900	0.46	0.64	1.07	1.43	1.85	2.61
xc)	4 950	0.45	0.63	1.06	1.42	1.83	2.58
xci)	5 000	0.45	0.63	1.05	1.40	1.81	2.56
xcii)	Maximum aspect ratio	6.5	6.2	5.7	4.8	4.3	3.6

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Table 19 Maximum Areas for Tempered Glass Fixed on All Four Sides, m^2

(*Clauses* 6.1.9 *and* 6.3.6.1)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Tempered Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
i)	500	5.32	9.64	15.00	15.00	15.00	15.00	15.00	15.00	15.00
ii)	550	4.84	8.76	13.70	15.00	15.00	15.00	15.00	15.00	15.00
iii)	600	4.43	8.03	12.56	15.00	15.00	15.00	15.00	15.00	15.00
iv)	650	4.09	7.42	11.59	15.00	15.00	15.00	15.00	15.00	15.00
v)	700	3.80	6.89	10.77	15.00	15.00	15.00	15.00	15.00	15.00
vi)	750	3.55	6.43	10.05	14.39	15.00	15.00	15.00	15.00	15.00
vii)	800	3.33	6.03	9.42	13.49	15.00	15.00	15.00	15.00	15.00
viii)	850	3.13	5.67	8.87	12.70	15.00	15.00	15.00	15.00	15.00
ix)	900	2.96	5.36	8.37	11.99	15.00	15.00	15.00	15.00	15.00
x)	950	2.80	5.07	7.93	11.36	15.00	15.00	15.00	15.00	15.00
xi)	1 000	2.66	4.82	7.54	10.79	15.00	15.00	15.00	15.00	15.00
xii)	1 050	2.53	4.59	7.18	10.28	15.00	15.00	15.00	15.00	15.00
xiii)	1 100	2.42	4.38	6.85	9.81	14.84	15.00	15.00	15.00	15.00
xiv)	1 150	2.31	4.19	6.55	9.38	14.20	15.00	15.00	15.00	15.00
xv)	1 200	2.22	4.02	6.28	8.99	13.61	15.00	15.00	15.00	15.00
xvi)	1 250	2.13	3.86	6.03	8.63	13.06	15.00	15.00	15.00	15.00
xvii)	1 300	2.05	3.71	5.80	8.30	12.56	15.00	15.00	15.00	15.00
xviii)	1 350	1.97	3.57	5.58	7.99	12.10	15.00	15.00	15.00	15.00
xix)	1 400	1.90	3.44	5.38	7.71	11.66	15.00	15.00	15.00	15.00
xx)	1 450	1.83	3.32	5.20	7.44	11.26	15.00	15.00	15.00	15.00
xxi)	1 500	1.77	3.21	5.02	7.19	10.89	14.61	15.00	15.00	15.00
xxii)	1 550	1.72	3.11	4.86	6.96	10.53	14.14	15.00	15.00	15.00
xxiii)	1 600	1.66	3.01	4.71	6.74	10.21	13.69	15.00	15.00	15.00
xxiv)	1 650	1.61	2.92	4.57	6.54	9.90	13.28	15.00	15.00	15.00
xxv)	1 700	1.56	2.84	4.43	6.35	9.61	12.89	15.00	15.00	15.00
xxvi)	1 750	1.52	2.75	4.31	6.17	9.33	12.52	15.00	15.00	15.00
xxvii)	1 800	1.48	2.68	4.19	6.00	9.07	12.17	15.00	15.00	15.00
xxviii)	1 850	1.44	2.61	4.07	5.83	8.83	11.84	14.95	15.00	15.00

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Table 19 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Tempered Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
xxix)	1 900	1.40	2.54	3.97	5.68	8.59	11.53	14.55	15.00	15.00
xxx)	1 950	1.36	2.47	3.86	5.53	8.37	11.24	14.18	15.00	15.00
xxxi)	2 000	1.33	2.41	3.77	5.40	8.16	10.96	13.83	15.00	15.00
xxxii)	2 050	1.30	2.35	3.68	5.26	7.97	10.69	13.49	15.00	15.00
xxxiii)	2 100	1.27	2.30	3.59	5.14	7.78	10.43	13.17	15.00	15.00
xxxiv)	2 150	1.24	2.24	3.51	5.02	7.59	10.19	12.86	15.00	15.00
xxxv)	2 200	1.21	2.19	3.43	4.91	7.42	9.96	12.57	15.00	15.00
xxxvi)	2 250	1.18	2.14	3.35	4.80	7.26	9.74	12.29	15.00	15.00
xxxvii)	2 300	1.16	2.10	3.28	4.69	7.10	9.53	12.02	15.00	15.00
xxxviii)	2 350	1.13	2.05	3.21	4.59	6.95	9.32	11.77	15.00	15.00
xxxix)	2 400	1.11	2.01	3.14	4.50	6.80	9.13	11.52	15.00	15.00
xl)	2 450	1.09	1.97	3.08	4.40	6.66	8.94	11.29	15.00	15.00
xli)	2 500	1.06	1.93	3.01	4.32	6.53	8.76	11.06	15.00	15.00
xlii)	2 550	1.04	1.89	2.96	4.23	6.40	8.59	10.84	14.80	15.00
xliii)	2 600	1.02	1.85	2.90	4.15	6.28	8.43	10.64	14.52	15.00
xliv)	2 650	1.00	1.82	2.84	4.07	6.16	8.27	10.43	14.24	15.00
xlv)	2 700	0.99	1.79	2.79	4.00	6.05	8.12	10.24	13.98	15.00
xlvi)	2 750	0.97	1.75	2.74	3.92	5.94	7.97	10.06	13.73	15.00
xlvii)	2 800	0.95	1.72	2.69	3.85	5.83	7.83	9.88	13.48	15.00
xlviii)	2 850	0.93	1.69	2.64	3.79	5.73	7.69	9.70	13.24	15.00
xlix)	2 900	0.92	1.66	2.60	3.72	5.63	7.56	9.54	13.02	15.00
l)	2 950	0.90	1.63	2.55	3.66	5.54	7.43	9.37	12.79	15.00
li)	3 000	0.89	1.61	2.51	3.60	5.44	7.30	9.22	12.58	15.00
lii)	3 050	0.87	1.58	2.47	3.54	5.35	7.18	9.07	12.38	15.00
liii)	3 100	0.86	1.56	2.43	3.48	5.27	7.07	8.92	12.18	15.00
liv)	3 150	0.84	1.53	2.39	3.43	5.18	6.96	8.78	11.98	15.00
lv)	3 200	0.83	1.51	2.36	3.37	5.10	6.85	8.64	11.79	15.00
lvi)	3 250	0.82	1.48	2.32	3.32	5.02	6.74	8.51	11.61	15.00
lvii)	3 300	0.81	1.46	2.28	3.27	4.95	6.64	8.38	11.44	15.00
lviii)	3 350	0.79	1.44	2.25	3.22	4.87	6.54	8.25	11.27	15.00

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Table 19 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Tempered Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
lix)	3 400	0.78	1.42	2.22	3.17	4.80	6.44	8.13	11.10	14.87
lx)	3 450	0.77	1.40	2.18	3.13	4.73	6.35	8.02	10.94	14.65
lxi)	3 500	0.76	1.38	2.15	3.08	4.67	6.26	7.90	10.78	14.44
lxii)	3 550	0.75	1.36	2.12	3.04	4.60	6.17	7.79	10.63	14.24
lxiii)	3 600	0.74	1.34	2.09	3.00	4.54	6.09	7.68	10.48	14.04
lxiv)	3 650	0.73	1.32	2.06	2.96	4.47	6.00	7.58	10.34	13.85
lxv)	3 700	0.72	1.30	2.04	2.92	4.41	5.92	7.47	10.20	13.66
lxvi)	3 750	0.71	1.29	2.01	2.88	4.35	5.84	7.37	10.07	13.48
lxvii)	3 800	0.70	1.27	1.98	2.84	4.30	5.77	7.28	9.93	13.30
lxviii)	3 850	0.69	1.25	1.96	2.80	4.24	5.69	7.18	9.80	13.13
lxix)	3 900	0.68	1.24	1.93	2.77	4.19	5.62	7.09	9.68	12.96
lxx)	3 950	0.67	1.22	1.91	2.73	4.13	5.55	7.00	9.56	12.80
lxxi)	4 000	0.67	1.21	1.88	2.70	4.08	5.48	6.91	9.44	12.64
lxxii)	4 050	0.66	1.19	1.86	2.66	4.03	5.41	6.83	9.32	12.48
lxxiii)	4 100	0.65	1.18	1.84	2.63	3.98	5.34	6.74	9.21	12.33
lxxiv)	4 150	0.64	1.16	1.82	2.60	3.93	5.28	6.66	9.09	12.18
lxxv)	4 200	0.63	1.15	1.79	2.57	3.89	5.22	6.58	8.99	12.04
lxxvi)	4 250	0.63	1.13	1.77	2.54	3.84	5.16	6.51	8.88	11.89
lxxvii)	4 300	0.62	1.12	1.75	2.51	3.80	5.10	6.43	8.78	11.76
lxxviii)	4 350	0.61	1.11	1.73	2.48	3.75	5.04	6.36	8.68	11.62
lxxix)	4 400	0.60	1.10	1.71	2.45	3.71	4.98	6.28	8.58	11.49
lxxx)	4 450	0.60	1.08	1.69	2.43	3.67	4.92	6.21	8.48	11.36
lxxxi)	4 500	0.59	1.07	1.67	2.40	3.63	4.87	6.14	8.39	11.23
lxxxii)	4 550	0.58	1.06	1.66	2.37	3.59	4.82	6.08	8.30	11.11
lxxxiii)	4 600	0.58	1.05	1.64	2.35	3.55	4.76	6.01	8.21	10.99
lxxxiv)	4 650	0.57	1.04	1.62	2.32	3.51	4.71	5.95	8.12	10.87
lxxxv)	4 700	0.57	1.03	1.60	2.30	3.47	4.66	5.88	8.03	10.76
lxxxvi)	4 750	0.56	1.01	1.59	2.27	3.44	4.61	5.82	7.95	10.64
lxxxvii)	4 800	0.55	1.00	1.57	2.25	3.40	4.56	5.76	7.86	10.53
lxxxviii)	4 850	0.55	0.99	1.55	2.23	3.37	4.52	5.70	7.78	10.42

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Table 19 (*Concluded*)

Sl No.	Design Wind Pressure N/m²	Nominal Thickness of Tempered Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
lxxxix)	4 900	0.54	0.98	1.54	2.20	3.33	4.47	5.64	7.70	10.32
xc)	4 950	0.54	0.97	1.52	2.18	3.30	4.43	5.59	7.63	10.21
xci)	5 000	0.53	0.96	1.51	2.16	3.27	4.38	5.53	7.55	10.11
xcii)	Maximum aspect ratio	7.3	6.8	6.5	6.3	5.9	4.9	4.3	3.8	3.3

Table 20 Maximum Areas for Insulating Glass Fixed on All Four Sides, m²

(*Clauses* 6.1.9 *and* 6.3.6.1)

Sl No.	Design Wind Pressure N/m²	Nominal Thickness of Glass, *T* mm						
		3 + 3	4 + 4	5 + 5	6 + 6	8 + 8	10 + 10	12 + 12
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	500	3.19	5.78	9.04	12.95	15.00	15.00	15.00
ii)	550	2.90	5.26	8.22	11.77	15.00	15.00	15.00
iii)	600	2.66	4.82	7.54	10.79	15.00	15.00	15.00
iv)	650	2.46	4.45	6.96	9.96	15.00	15.00	15.00
v)	700	2.28	4.13	6.46	9.25	14.00	15.00	15.00
vi)	750	2.13	3.86	6.03	8.63	13.06	15.00	15.00
vii)	800	2.00	3.62	5.65	8.09	12.25	15.00	15.00
viii)	850	1.88	3.40	5.32	7.62	11.53	15.00	15.00
ix)	900	1.77	3.21	5.02	7.19	10.89	14.61	15.00
x)	950	1.68	3.04	4.76	6.82	10.31	13.84	15.00
xi)	1 000	1.60	2.89	4.52	6.48	9.80	13.15	15.00
xii)	1 050	1.52	2.75	4.31	6.17	9.33	12.52	15.00
xiii)	1 100	1.45	2.63	4.11	5.89	8.91	11.95	15.00
xiv)	1 150	1.39	2.52	3.93	5.63	8.52	11.43	14.43
xv)	1 200	1.33	2.41	3.77	5.40	8.16	10.96	13.83
xvi)	1 250	1.28	2.31	3.62	5.18	7.84	10.52	13.27
xvii)	1 300	1.23	2.22	3.48	4.98	7.54	10.11	12.76
xviii)	1 350	1.18	2.14	3.35	4.80	7.26	9.74	12.29

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Table 20 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Glass, *T* mm						
		3 + 3	4 + 4	5 + 5	6 + 6	8 + 8	10 + 10	12 + 12
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
xix)	1 400	1.14	2.07	3.23	4.63	7.00	9.39	11.85
xx)	1 450	1.10	1.99	3.12	4.47	6.76	9.07	11.44
xxi)	1 500	1.06	1.93	3.01	4.32	6.53	8.76	11.06
xxii)	1 550	1.03	1.87	2.92	4.18	6.32	8.48	10.70
xxiii)	1 600	1.00	1.81	2.83	4.05	6.12	8.22	10.37
xxiv)	1 650	0.97	1.75	2.74	3.92	5.94	7.97	10.06
xxv)	1 700	0.94	1.70	2.66	3.81	5.76	7.73	9.76
xxvi)	1 750	0.91	1.65	2.58	3.70	5.60	7.51	9.48
xxvii)	1 800	0.89	1.61	2.51	3.60	5.44	7.30	9.22
xxviii)	1 850	0.86	1.56	2.44	3.50	5.30	7.11	8.97
xxix)	1 900	0.84	1.52	2.38	3.41	5.16	6.92	8.73
xxx)	1 950	0.82	1.48	2.32	3.32	5.02	6.74	8.51
xxxi)	2 000	0.80	1.45	2.26	3.24	4.90	6.57	8.30
xxxii)	2 050	0.78	1.41	2.21	3.16	4.78	6.41	8.09
xxxiii)	2 100	0.76	1.38	2.15	3.08	4.67	6.26	7.90
xxxiv)	2 150	0.74	1.35	2.10	3.01	4.56	6.11	7.72
xxxv)	2 200	0.73	1.31	2.06	2.94	4.45	5.98	7.54
xxxvi)	2 250	0.71	1.29	2.01	2.88	4.35	5.84	7.37
xxxvii)	2 300	0.69	1.26	1.97	2.82	4.26	5.72	7.21
xxxviii)	2 350	0.68	1.23	1.92	2.76	4.17	5.59	7.06
xxxix)	2 400	0.67	1.21	1.88	2.70	4.08	5.48	6.91
xl)	2 450	0.65	1.18	1.85	2.64	4.00	5.37	6.77
xli)	2 500	0.64	1.16	1.81	2.59	3.92	5.26	6.64
xlii)	2 550	0.63	1.13	1.77	2.54	3.84	5.16	6.51
xliii)	2 600	0.61	1.11	1.74	2.49	3.77	5.06	6.38
xliv)	2 650	0.60	1.09	1.71	2.44	3.70	4.96	6.26
xlv)	2 700	0.59	1.07	1.67	2.40	3.63	4.87	6.14
xlvi)	2 750	0.58	1.05	1.64	2.35	3.56	4.78	6.03
xlvii)	2 800	0.57	1.03	1.61	2.31	3.50	4.70	5.93
xlviii)	2 850	0.56	1.01	1.59	2.27	3.44	4.61	5.82

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Table 20 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Glass, *T* mm						
		3 + 3	4 + 4	5 + 5	6 + 6	8 + 8	10 + 10	12 + 12
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
xlix)	2 900	0.55	1.00	1.56	2.23	3.38	4.53	5.72
l)	2 950	0.54	0.98	1.53	2.19	3.32	4.46	5.62
li)	3 000	0.53	0.96	1.51	2.16	3.27	4.38	5.53
lii)	3 050	0.52	0.95	1.48	2.12	3.21	4.31	5.44
liii)	3 100	0.51	0.93	1.46	2.09	3.16	4.24	5.35
liv)	3 150	0.51	0.92	1.44	2.06	3.11	4.17	5.27
lv)	3 200	0.50	0.90	1.41	2.02	3.06	4.11	5.18
lvi)	3 250	0.49	0.89	1.39	1.99	3.01	4.05	5.11
lvii)	3 300	0.48	0.88	1.37	1.96	2.97	3.98	5.03
lviii)	3 350	0.48	0.86	1.35	1.93	2.92	3.92	4.95
lix)	3 400	0.47	0.85	1.33	1.90	2.88	3.87	4.88
lx)	3 450	0.46	0.84	1.31	1.88	2.84	3.81	4.81
lxi)	3 500	0.46	0.83	1.29	1.85	2.80	3.76	4.74
lxii)	3 550	0.45	0.81	1.27	1.82	2.76	3.70	4.67
lxiii)	3 600	0.44	0.80	1.26	1.80	2.72	3.65	4.61
lxiv)	3 650	0.44	0.79	1.24	1.77	2.68	3.60	4.55
lxv)	3 700	0.43	0.78	1.22	1.75	2.65	3.55	4.48
lxvi)	3 750	0.43	0.77	1.21	1.73	2.61	3.51	4.42
lxvii)	3 800	0.42	0.76	1.19	1.70	2.58	3.46	4.37
lxviii)	3 850	0.41	0.75	1.17	1.68	2.54	3.41	4.31
lxix)	3 900	0.41	0.74	1.16	1.66	2.51	3.37	4.25
lxx)	3 950	0.40	0.73	1.14	1.64	2.48	3.33	4.20
lxxi)	4 000	0.40	0.72	1.13	1.62	2.45	3.29	4.15
lxxii)	4 050	0.39	0.71	1.12	1.60	2.42	3.25	4.10
lxxiii)	4 100	0.39	0.71	1.10	1.58	2.39	3.21	4.05
lxxiv)	4 150	0.38	0.70	1.09	1.56	2.36	3.17	4.00
lxxv)	4 200	0.38	0.69	1.08	1.54	2.33	3.13	3.95
lxxvi)	4 250	0.38	0.68	1.06	1.52	2.31	3.09	3.90
lxxvii)	4 300	0.37	0.67	1.05	1.51	2.28	3.06	3.86
lxxviii)	4 350	0.37	0.66	1.04	1.49	2.25	3.02	3.81

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Table 20 (*Concluded*)

Sl No.	Design Wind Pressure N/m²	Nominal Thickness of Glass, *T* mm						
		3 + 3	4 + 4	5 + 5	6 + 6	8 + 8	10 + 10	12 + 12
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
lxxix)	4 400	0.36	0.66	1.03	1.47	2.23	2.99	3.77
lxxx)	4 450	0.36	0.65	1.02	1.46	2.20	2.95	3.73
lxxxi)	4 500	0.35	0.64	1.00	1.44	2.18	2.92	3.69
lxxxii)	4 550	0.35	0.64	0.99	1.42	2.15	2.89	3.65
lxxxiii)	4 600	0.35	0.63	0.98	1.41	2.13	2.86	3.61
lxxxiv)	4 650	0.34	0.62	0.97	1.39	2.11	2.83	3.57
lxxxv)	4 700	0.34	0.62	0.96	1.38	2.08	2.80	3.53
lxxxvi)	4 750	0.34	0.61	0.95	1.36	2.06	2.77	3.49
lxxxvii)	4 800	0.33	0.60	0.94	1.35	2.04	2.74	3.46
lxxxviii)	4 850	0.33	0.60	0.93	1.34	2.02	2.71	3.42
lxxxix)	4 900	0.33	0.59	0.92	1.32	2.00	2.68	3.39
xc)	4 950	0.32	0.58	0.91	1.31	1.98	2.66	3.35
xci)	5 000	0.32	0.58	0.90	1.30	1.96	2.63	3.32
xcii)	Maximum aspect ratio	7.3	6.8	6.5	6.3	5.9	4.9	4.3

Table 21 Maximum Area for Heat Strengthened Glass Fixed on Four Sides, m²

(*Clauses* 6.1.9, 6.3.5 *and* 6.3.6.1)

Sl No.	Design Wind Pressure N/m²	Nominal Thickness of Heat Strengthened Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
i)	500	3.41	6.17	9.65	13.81	15.00	15.00	15.00	15.00	15.00
ii)	550	3.10	5.61	8.77	12.56	15.00	15.00	15.00	15.00	15.00
iii)	600	2.84	5.14	8.04	11.51	15.00	15.00	15.00	15.00	15.00
iv)	650	2.62	4.75	7.42	10.63	15.00	15.00	15.00	15.00	15.00
v)	700	2.43	4.41	6.89	9.87	14.93	15.00	15.00	15.00	15.00
vi)	750	2.27	4.11	6.43	9.21	13.93	15.00	15.00	15.00	15.00
vii)	800	2.13	3.86	6.03	8.63	13.06	15.00	15.00	15.00	15.00

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Table 21 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Heat Strengthened Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
viii)	850	2.00	3.63	5.67	8.13	12.29	15.00	15.00	15.00	15.00
ix)	900	1.89	3.43	5.36	7.67	11.61	15.00	15.00	15.00	15.00
x)	950	1.79	3.25	5.08	7.27	11.00	14.76	15.00	15.00	15.00
xi)	1 000	1.70	3.09	4.82	6.91	10.45	14.02	15.00	15.00	15.00
xii)	1 050	1.62	2.94	4.59	6.58	9.95	13.36	15.00	15.00	15.00
xiii)	1 100	1.55	2.80	4.38	6.28	9.50	12.75	15.00	15.00	15.00
xiv)	1 150	1.48	2.68	4.19	6.01	9.09	12.19	15.00	15.00	15.00
xv)	1 200	1.42	2.57	4.02	5.76	8.71	11.69	14.75	15.00	15.00
xvi)	1 250	1.36	2.47	3.86	5.53	8.36	11.22	14.16	15.00	15.00
xvii)	1 300	1.31	2.37	3.71	5.31	8.04	10.79	13.61	15.00	15.00
xviii)	1 350	1.26	2.29	3.57	5.12	7.74	10.39	13.11	15.00	15.00
xix)	1 400	1.22	2.20	3.45	4.93	7.46	10.02	12.64	15.00	15.00
xx)	1 450	1.17	2.13	3.33	4.76	7.21	9.67	12.21	15.00	15.00
xxi)	1 500	1.14	2.06	3.22	4.60	6.97	9.35	11.80	15.00	15.00
xxii)	1 550	1.10	1.99	3.11	4.46	6.74	9.05	11.42	15.00	15.00
xxiii)	1 600	1.06	1.93	3.01	4.32	6.53	8.76	11.06	15.00	15.00
xxiv)	1 650	1.03	1.87	2.92	4.19	6.33	8.50	10.73	14.64	15.00
xxv)	1 700	1.00	1.81	2.84	4.06	6.15	8.25	10.41	14.21	15.00
xxvi)	1 750	0.97	1.76	2.76	3.95	5.97	8.01	10.11	13.80	15.00
xxvii)	1 800	0.95	1.71	2.68	3.84	5.81	7.79	9.83	13.42	15.00
xxviii)	1 850	0.92	1.67	2.61	3.73	5.65	7.58	9.57	13.06	15.00
xxix)	1 900	0.90	1.62	2.54	3.64	5.50	7.38	9.31	12.71	15.00
xxx)	1 950	0.87	1.58	2.47	3.54	5.36	7.19	9.08	12.39	15.00
xxxi)	2 000	0.85	1.54	2.41	3.45	5.23	7.01	8.85	12.08	15.00
xxxii)	2 050	0.83	1.50	2.35	3.37	5.10	6.84	8.63	11.78	15.00
xxxiii)	2 100	0.81	1.47	2.30	3.29	4.98	6.68	8.43	11.50	15.00
xxxiv)	2 150	0.79	1.43	2.24	3.21	4.86	6.52	8.23	11.24	15.00
xxxv)	2 200	0.77	1.40	2.19	3.14	4.75	6.37	8.04	10.98	14.71
xxxvi)	2 250	0.76	1.37	2.14	3.07	4.64	6.23	7.87	10.74	14.38
xxxvii)	2 300	0.74	1.34	2.10	3.00	4.54	6.10	7.69	10.50	14.07

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Table 21 (*Continued*)

Sl No.	Design Wind Pressure N/m²	Nominal Thickness of Heat Strengthened Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
xxxviii)	2 350	0.72	1.31	2.05	2.94	4.45	5.97	7.53	10.28	13.77
xxxix)	2 400	0.71	1.29	2.01	2.88	4.35	5.84	7.37	10.07	13.48
xl)	2 450	0.69	1.26	1.97	2.82	4.27	5.72	7.22	9.86	13.21
xli)	2 500	0.68	1.23	1.93	2.76	4.18	5.61	7.08	9.66	12.94
xlii)	2 550	0.67	1.21	1.89	2.71	4.10	5.50	6.94	9.47	12.69
xliii)	2 600	0.65	1.19	1.86	2.66	4.02	5.39	6.81	9.29	12.44
xliv)	2 650	0.64	1.16	1.82	2.61	3.94	5.29	6.68	9.12	12.21
xlv)	2 700	0.63	1.14	1.79	2.56	3.87	5.19	6.55	8.95	11.98
xlvi)	2 750	0.62	1.12	1.75	2.51	3.80	5.10	6.44	8.78	11.76
xlvii)	2 800	0.61	1.10	1.72	2.47	3.73	5.01	6.32	8.63	11.55
xlviii)	2 850	0.60	1.08	1.69	2.42	3.67	4.92	6.21	8.48	11.35
xlix)	2 900	0.59	1.06	1.66	2.38	3.60	4.84	6.10	8.33	11.16
l)	2 950	0.58	1.05	1.63	2.34	3.54	4.75	6.00	8.19	10.97
li)	3 000	0.57	1.03	1.61	2.30	3.48	4.67	5.90	8.05	10.78
lii)	3 050	0.56	1.01	1.58	2.26	3.43	4.60	5.80	7.92	10.61
liii)	3 100	0.55	1.00	1.56	2.23	3.37	4.52	5.71	7.79	10.44
liv)	3 150	0.54	0.98	1.53	2.19	3.32	4.45	5.62	7.67	10.27
lv)	3 200	0.53	0.96	1.51	2.16	3.27	4.38	5.53	7.55	10.11
lvi)	3 250	0.52	0.95	1.48	2.13	3.22	4.31	5.45	7.43	9.95
lvii)	3 300	0.52	0.93	1.46	2.09	3.17	4.25	5.36	7.32	9.80
lviii)	3 350	0.51	0.92	1.44	2.06	3.12	4.19	5.28	7.21	9.66
lix)	3 400	0.50	0.91	1.42	2.03	3.07	4.12	5.21	7.10	9.52
lx)	3 450	0.49	0.89	1.40	2.00	3.03	4.06	5.13	7.00	9.38
lxi)	3 500	0.49	0.88	1.38	1.97	2.99	4.01	5.06	6.90	9.24
lxii)	3 550	0.48	0.87	1.36	1.95	2.94	3.95	4.99	6.80	9.11
lxiii)	3 600	0.47	0.86	1.34	1.92	2.90	3.90	4.92	6.71	8.99
lxiv)	3 650	0.47	0.85	1.32	1.89	2.86	3.84	4.85	6.62	8.86
lxv)	3 700	0.46	0.83	1.30	1.87	2.82	3.79	4.78	6.53	8.74
lxvi)	3 750	0.45	0.82	1.29	1.84	2.79	3.74	4.72	6.44	8.63
lxvii)	3 800	0.45	0.81	1.27	1.82	2.75	3.69	4.66	6.36	8.51

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Table 21 (*Concluded*)

Sl No.	Design Wind Pressure N/m²	Nominal Thickness of Heat Strengthened Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
lxviii)	3 850	0.44	0.80	1.25	1.79	2.71	3.64	4.60	6.27	8.40
lxix)	3 900	0.44	0.79	1.24	1.77	2.68	3.60	4.54	6.19	8.30
lxx)	3 950	0.43	0.78	1.22	1.75	2.65	3.55	4.48	6.12	8.19
lxxi)	4 000	0.43	0.77	1.21	1.73	2.61	3.51	4.42	6.04	8.09
lxxii)	4 050	0.42	0.76	1.19	1.71	2.58	3.46	4.37	5.96	7.99
lxxiii)	4 100	0.42	0.75	1.18	1.68	2.55	3.42	4.32	5.89	7.89
lxxiv)	4 150	0.41	0.74	1.16	1.66	2.52	3.38	4.26	5.82	7.80
lxxv)	4 200	0.41	0.73	1.15	1.64	2.49	3.34	4.21	5.75	7.70
lxxvi)	4 250	0.40	0.73	1.13	1.63	2.46	3.30	4.16	5.68	7.61
lxxvii)	4 300	0.40	0.72	1.12	1.61	2.43	3.26	4.12	5.62	7.52
lxxviii)	4 350	0.39	0.71	1.11	1.59	2.40	3.22	4.07	5.55	7.44
lxxix)	4 400	0.39	0.70	1.10	1.57	2.38	3.19	4.02	5.49	7.35
lxxx)	4 450	0.38	0.69	1.08	1.55	2.35	3.15	3.98	5.43	7.27
lxxxi)	4 500	0.38	0.69	1.07	1.53	2.32	3.12	3.93	5.37	7.19
lxxxii)	4 550	0.37	0.68	1.06	1.52	2.30	3.08	3.89	5.31	7.11
lxxxiii)	4 600	0.37	0.67	1.05	1.50	2.27	3.05	3.85	5.25	7.03
lxxxiv)	4 650	0.37	0.66	1.04	1.49	2.25	3.02	3.81	5.19	6.96
lxxxv)	4 700	0.36	0.66	1.03	1.47	2.22	2.98	3.77	5.14	6.88
lxxxvi)	4 750	0.36	0.65	1.02	1.45	2.20	2.95	3.73	5.09	6.81
lxxxvii)	4 800	0.35	0.64	1.00	1.44	2.18	2.92	3.69	5.03	6.74
lxxxviii)	4 850	0.35	0.64	0.99	1.42	2.15	2.89	3.65	4.98	6.67
lxxxix)	4 900	0.35	0.63	0.98	1.41	2.13	2.86	3.61	4.93	6.60
xc)	4 950	0.34	0.62	0.97	1.40	2.11	2.83	3.58	4.88	6.54
xci)	5 000	0.34	0.62	0.96	1.38	2.09	2.80	3.54	4.83	6.47
xcii)	Maximum aspect ratio	7.3	6.8	6.5	6.3	5.9	4.9	4.3	3.8	3.3

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Table 22 Maximum Span for Annealed Glass Fixed on Two Opposite Sides, m

(*Clauses* 6.1.9 *and* 6.3.6.1)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Annealed Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
i)	500	0.53	0.73	0.92	1.12	1.40	1.77	2.11	2.68	3.36
ii)	550	0.51	0.69	0.88	1.07	1.33	1.69	2.02	2.56	3.20
iii)	600	0.48	0.66	0.84	1.02	1.27	1.62	1.93	2.45	3.07
iv)	650	0.46	0.64	0.81	0.98	1.22	1.56	1.85	2.35	2.95
v)	700	0.45	0.61	0.78	0.95	1.18	1.50	1.79	2.26	2.84
vi)	750	0.43	0.59	0.75	0.91	1.14	1.45	1.73	2.19	2.74
vii)	800	0.42	0.57	0.73	0.88	1.10	1.40	1.67	2.12	2.66
viii)	850	0.41	0.56	0.71	0.86	1.07	1.36	1.62	2.06	2.58
ix)	900	0.40	0.54	0.69	0.83	1.04	1.32	1.58	2.00	2.50
x)	950	0.38	0.53	0.67	0.81	1.01	1.29	1.53	1.94	2.44
xi)	1 000	0.37	0.51	0.65	0.79	0.99	1.25	1.49	1.89	2.38
xii)	1 050	0.37	0.50	0.64	0.77	0.96	1.22	1.46	1.85	2.32
xiii)	1 100	0.36	0.49	0.62	0.75	0.94	1.20	1.43	1.81	2.26
xiv)	1 150	0.35	0.48	0.61	0.74	0.92	1.17	1.39	1.77	2.22
xv)	1 200	0.34	0.47	0.60	0.72	0.90	1.15	1.36	1.73	2.17
xvi)	1 250	0.34	0.46	0.58	0.71	0.88	1.12	1.34	1.69	2.12
xvii)	1 300	0.33	0.45	0.57	0.69	0.87	1.10	1.31	1.66	2.08
xviii)	1 350	0.32	0.44	0.56	0.68	0.85	1.08	1.29	1.63	2.04
xix)	1 400	0.32	0.43	0.55	0.67	0.83	1.06	1.26	1.60	2.01
xx)	1 450	0.31	0.43	0.54	0.66	0.82	1.04	1.24	1.57	1.97
xxi)	1 500	0.31	0.42	0.53	0.65	0.81	1.02	1.22	1.55	1.94
xxii)	1 550	0.30	0.41	0.52	0.64	0.79	1.01	1.20	1.52	1.91
xxiii)	1 600	0.30	0.41	0.52	0.63	0.78	0.99	1.18	1.50	1.88
xxiv)	1 650	0.29	0.40	0.51	0.62	0.77	0.98	1.16	1.48	1.85
xxv)	1 700	0.29	0.39	0.50	0.61	0.76	0.96	1.15	1.45	1.82
xxvi)	1 750	0.28	0.39	0.49	0.60	0.75	0.95	1.13	1.43	1.80
xxvii)	1 800	0.28	0.38	0.49	0.59	0.74	0.93	1.11	1.41	1.77
xxviii)	1 850	0.28	0.38	0.48	0.58	0.73	0.92	1.10	1.39	1.75
xxix)	1 900	0.27	0.37	0.47	0.57	0.72	0.91	1.08	1.37	1.72
xxx)	1 950	0.27	0.37	0.47	0.57	0.71	0.90	1.07	1.36	1.70

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Table 22 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Annealed Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
xxxi)	2 000	0.27	0.36	0.46	0.56	0.70	0.89	1.06	1.34	1.68
xxxii)	2 050	0.26	0.36	0.46	0.55	0.69	0.88	1.04	1.32	1.66
xxxiii)	2 100	0.26	0.35	0.45	0.55	0.68	0.87	1.03	1.31	1.64
xxxiv)	2 150	0.26	0.35	0.44	0.54	0.67	0.86	1.02	1.29	1.62
xxxv)	2 200	0.25	0.35	0.44	0.53	0.67	0.85	1.01	1.28	1.60
xxxvi)	2 250	0.25	0.34	0.43	0.53	0.66	0.84	1.00	1.26	1.58
xxxvii)	2 300	0.25	0.34	0.43	0.52	0.65	0.83	0.99	1.25	1.57
xxxviii)	2 350	0.24	0.34	0.43	0.52	0.64	0.82	0.97	1.24	1.55
xxxix)	2 400	0.24	0.33	0.42	0.51	0.64	0.81	0.96	1.22	1.53
xl)	2 450	0.24	0.33	0.42	0.51	0.63	0.80	0.95	1.21	1.52
xli)	2 500	0.24	0.32	0.41	0.50	0.62	0.79	0.95	1.20	1.50
xlii)	2 550	0.23	0.32	0.41	0.50	0.62	0.79	0.94	1.19	1.49
xliii)	2 600	0.23	0.32	0.40	0.49	0.61	0.78	0.93	1.18	1.47
xliv)	2 650	0.23	0.32	0.40	0.49	0.61	0.77	0.92	1.16	1.46
xlv)	2 700	0.23	0.31	0.40	0.48	0.60	0.76	0.91	1.15	1.45
xlvi)	2 750	0.23	0.31	0.39	0.48	0.60	0.76	0.90	1.14	1.43
xlvii)	2 800	0.22	0.31	0.39	0.47	0.59	0.75	0.89	1.13	1.42
xlviii)	2 850	0.22	0.30	0.39	0.47	0.58	0.74	0.89	1.12	1.41
xlix)	2 900	0.22	0.30	0.38	0.46	0.58	0.74	0.88	1.11	1.39
l)	2 950	0.22	0.30	0.38	0.46	0.57	0.73	0.87	1.10	1.38
li)	3 000	0.22	0.30	0.38	0.46	0.57	0.72	0.86	1.09	1.37
lii)	3 050	0.21	0.29	0.37	0.45	0.57	0.72	0.86	1.09	1.36
liii)	3 100	0.21	0.29	0.37	0.45	0.56	0.71	0.85	1.08	1.35
liv)	3 150	0.21	0.29	0.37	0.45	0.56	0.71	0.84	1.07	1.34
lv)	3 200	0.21	0.29	0.36	0.44	0.55	0.70	0.84	1.06	1.33
lvi)	3 250	0.21	0.28	0.36	0.44	0.55	0.70	0.83	1.05	1.32
lvii)	3 300	0.21	0.28	0.36	0.44	0.54	0.69	0.82	1.04	1.31
lviii)	3 350	0.20	0.28	0.36	0.43	0.54	0.69	0.82	1.04	1.30
lix)	3 400	0.20	0.28	0.35	0.43	0.54	0.68	0.81	1.03	1.29
lx)	3 450	0.20	0.28	0.35	0.43	0.53	0.68	0.80	1.02	1.28

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Table 22 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Annealed Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
lxi)	3 500	0.20	0.27	0.35	0.42	0.53	0.67	0.80	1.01	1.27
lxii)	3 550	0.20	0.27	0.35	0.42	0.52	0.67	0.79	1.01	1.26
lxiii)	3 600	0.20	0.27	0.34	0.42	0.52	0.66	0.79	1.00	1.25
lxiv)	3 650	0.20	0.27	0.34	0.41	0.52	0.66	0.78	0.99	1.24
lxv)	3 700	0.19	0.27	0.34	0.41	0.51	0.65	0.78	0.99	1.23
lxvi)	3 750	0.19	0.27	0.34	0.41	0.51	0.65	0.77	0.98	1.23
lxvii)	3 800	0.19	0.26	0.33	0.41	0.51	0.64	0.77	0.97	1.22
lxviii)	3 850	0.19	0.26	0.33	0.40	0.50	0.64	0.76	0.97	1.21
lxix)	3 900	0.19	0.26	0.33	0.40	0.50	0.64	0.76	0.96	1.20
lxx)	3 950	0.19	0.26	0.33	0.40	0.50	0.63	0.75	0.95	1.20
lxxi)	4 000	0.19	0.26	0.33	0.40	0.49	0.63	0.75	0.95	1.19
lxxii)	4 050	0.19	0.26	0.32	0.39	0.49	0.62	0.74	0.94	1.18
lxxiii)	4 100	0.19	0.25	0.32	0.39	0.49	0.62	0.74	0.94	1.17
lxxiv)	4 150	0.18	0.25	0.32	0.39	0.48	0.62	0.73	0.93	1.17
lxxv)	4 200	0.18	0.25	0.32	0.39	0.48	0.61	0.73	0.92	1.16
lxxvi)	4 250	0.18	0.25	0.32	0.38	0.48	0.61	0.72	0.92	1.15
lxxvii)	4 300	0.18	0.25	0.31	0.38	0.48	0.60	0.72	0.91	1.15
lxxviii)	4 350	0.18	0.25	0.31	0.38	0.47	0.60	0.72	0.91	1.14
lxxix)	4 400	0.18	0.24	0.31	0.38	0.47	0.60	0.71	0.90	1.13
lxxx)	4 450	0.18	0.24	0.31	0.38	0.47	0.59	0.71	0.90	1.13
lxxxi)	4 500	0.18	0.24	0.31	0.37	0.47	0.59	0.70	0.89	1.12
lxxxii)	4 550	0.18	0.24	0.31	0.37	0.46	0.59	0.70	0.89	1.11
lxxxiii)	4 600	0.17	0.24	0.30	0.37	0.46	0.58	0.70	0.88	1.11
lxxxiv)	4 650	0.17	0.24	0.30	0.37	0.46	0.58	0.69	0.88	1.10
lxxxv)	4 700	0.17	0.24	0.30	0.36	0.46	0.58	0.69	0.87	1.10
lxxxvi)	4 750	0.17	0.24	0.30	0.36	0.45	0.58	0.69	0.87	1.09
lxxxvii)	4 800	0.17	0.23	0.30	0.36	0.45	0.57	0.68	0.86	1.08
lxxxviii)	4 850	0.17	0.23	0.30	0.36	0.45	0.57	0.68	0.86	1.08
lxxxix)	4 900	0.17	0.23	0.29	0.36	0.45	0.57	0.68	0.86	1.07
xc)	4 950	0.17	0.23	0.29	0.36	0.44	0.56	0.67	0.85	1.07

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Table 22 (*Concluded*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Annealed Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
xci)	5 000	0.17	0.23	0.29	0.35	0.44	0.56	0.67	0.85	1.06

Table 23 Maximum Span for Laminated Glass Fixed on Two Opposite Sides, m

(*Clauses* 6.1.9, 6.3.5 *and* 6.3.6.1)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Laminated Glass, *T* mm					
		5.38	6.38	8.38	10.38	12.38	16.38
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	500	0.79	0.95	1.25	1.59	1.92	2.50
ii)	550	0.75	0.90	1.19	1.51	1.83	2.38
iii)	600	0.72	0.87	1.14	1.45	1.76	2.28
iv)	650	0.69	0.83	1.10	1.39	1.69	2.19
v)	700	0.67	0.80	1.06	1.34	1.63	2.11
vi)	750	0.65	0.77	1.02	1.30	1.57	2.04
vii)	800	0.62	0.75	0.99	1.25	1.52	1.98
viii)	850	0.61	0.73	0.96	1.22	1.48	1.92
ix)	900	0.59	0.71	0.93	1.18	1.43	1.86
x)	950	0.57	0.69	0.91	1.15	1.40	1.81
xi)	1 000	0.56	0.67	0.88	1.12	1.36	1.77
xii)	1 050	0.55	0.65	0.86	1.09	1.33	1.72
xiii)	1 100	0.53	0.64	0.84	1.07	1.30	1.68
xiv)	1 150	0.52	0.63	0.82	1.05	1.27	1.65
xv)	1 200	0.51	0.61	0.81	1.02	1.24	1.61
xvi)	1 250	0.50	0.60	0.79	1.00	1.22	1.58
xvii)	1 300	0.49	0.59	0.77	0.98	1.19	1.55
xviii)	1 350	0.48	0.58	0.76	0.97	1.17	1.52
xix)	1 400	0.47	0.57	0.75	0.95	1.15	1.49
xx)	1 450	0.46	0.56	0.73	0.93	1.13	1.47
xxi)	1 500	0.46	0.55	0.72	0.92	1.11	1.44

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Table 23 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Laminated Glass, *T* mm					
		5.38	6.38	8.38	10.38	12.38	16.38
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
xxii)	1 550	0.45	0.54	0.71	0.90	1.09	1.42
xxiii)	1 600	0.44	0.53	0.70	0.89	1.08	1.40
xxiv)	1 650	0.43	0.52	0.69	0.87	1.06	1.38
xxv)	1 700	0.43	0.51	0.68	0.86	1.04	1.35
xxvi)	1 750	0.42	0.51	0.67	0.85	1.03	1.34
xxvii)	1 800	0.42	0.50	0.66	0.84	1.01	1.32
xxviii)	1 850	0.41	0.49	0.65	0.82	1.00	1.30
xxix)	1 900	0.41	0.49	0.64	0.81	0.99	1.28
xxx)	1 950	0.40	0.48	0.63	0.80	0.97	1.27
xxxi)	2 000	0.40	0.47	0.62	0.79	0.96	1.25
xxxii)	2 050	0.39	0.47	0.62	0.78	0.95	1.23
xxxiii)	2 100	0.39	0.46	0.61	0.77	0.94	1.22
xxxiv)	2 150	0.38	0.46	0.60	0.77	0.93	1.20
xxxv)	2 200	0.38	0.45	0.60	0.76	0.92	1.19
xxxvi)	2 250	0.37	0.45	0.59	0.75	0.91	1.18
xxxvii)	2 300	0.37	0.44	0.58	0.74	0.90	1.16
xxxviii)	2 350	0.36	0.44	0.58	0.73	0.89	1.15
xxxix)	2 400	0.36	0.43	0.57	0.72	0.88	1.14
xl)	2 450	0.36	0.43	0.56	0.72	0.87	1.13
xli)	2 500	0.35	0.42	0.56	0.71	0.86	1.12
xlii)	2 550	0.35	0.42	0.55	0.70	0.85	1.11
xliii)	2 600	0.35	0.42	0.55	0.70	0.84	1.10
xliv)	2 650	0.34	0.41	0.54	0.69	0.84	1.09
xlv)	2 700	0.34	0.41	0.54	0.68	0.83	1.08
xlvi)	2 750	0.34	0.40	0.53	0.68	0.82	1.07
xlvii)	2 800	0.33	0.40	0.53	0.67	0.81	1.06
xlviii)	2 850	0.33	0.40	0.52	0.66	0.81	1.05
xlix)	2 900	0.33	0.39	0.52	0.66	0.80	1.04
l)	2 950	0.33	0.39	0.51	0.65	0.79	1.03

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Table 23 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Laminated Glass, *T* mm					
		5.38	6.38	8.38	10.38	12.38	16.38
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
li)	3 000	0.32	0.39	0.51	0.65	0.79	1.02
lii)	3 050	0.32	0.38	0.51	0.64	0.78	1.01
liii)	3 100	0.32	0.38	0.50	0.64	0.77	1.00
liv)	3 150	0.31	0.38	0.50	0.63	0.77	1.00
lv)	3 200	0.31	0.37	0.49	0.63	0.76	0.99
lvi)	3 250	0.31	0.37	0.49	0.62	0.75	0.98
lvii)	3 300	0.31	0.37	0.49	0.62	0.75	0.97
lviii)	3 350	0.31	0.37	0.48	0.61	0.74	0.97
lix)	3 400	0.30	0.36	0.48	0.61	0.74	0.96
lx)	3 450	0.30	0.36	0.48	0.60	0.73	0.95
lxi)	3 500	0.30	0.36	0.47	0.60	0.73	0.94
lxii)	3 550	0.30	0.36	0.47	0.60	0.72	0.94
lxiii)	3 600	0.29	0.35	0.47	0.59	0.72	0.93
lxiv)	3 650	0.29	0.35	0.46	0.59	0.71	0.92
lxv)	3 700	0.29	0.35	0.46	0.58	0.71	0.92
lxvi)	3 750	0.29	0.35	0.46	0.58	0.70	0.91
lxvii)	3 800	0.29	0.34	0.45	0.58	0.70	0.91
lxviii)	3 850	0.28	0.34	0.45	0.57	0.69	0.90
lxix)	3 900	0.28	0.34	0.45	0.57	0.69	0.89
lxx)	3 950	0.28	0.34	0.44	0.56	0.68	0.89
lxxi)	4 000	0.28	0.34	0.44	0.56	0.68	0.88
lxxii)	4 050	0.28	0.33	0.44	0.56	0.68	0.88
lxxiii)	4 100	0.28	0.33	0.44	0.55	0.67	0.87
lxxiv)	4 150	0.27	0.33	0.43	0.55	0.67	0.87
lxxv)	4 200	0.27	0.33	0.43	0.55	0.66	0.86
lxxvi)	4 250	0.27	0.33	0.43	0.54	0.66	0.86
lxxvii)	4 300	0.27	0.32	0.43	0.54	0.66	0.85
lxxviii)	4 350	0.27	0.32	0.42	0.54	0.65	0.85
lxxix)	4 400	0.27	0.32	0.42	0.53	0.65	0.84

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Table 23 (*Concluded*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Laminated Glass, *T* mm					
		5.38	6.38	8.38	10.38	12.38	16.38
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
lxxx)	4 450	0.26	0.32	0.42	0.53	0.65	0.84
lxxxi)	4 500	0.26	0.32	0.42	0.53	0.64	0.83
lxxxii)	4 550	0.26	0.31	0.41	0.53	0.64	0.83
lxxxiii)	4 600	0.26	0.31	0.41	0.52	0.63	0.82
lxxxiv)	4 650	0.26	0.31	0.41	0.52	0.63	0.82
lxxxv)	4 700	0.26	0.31	0.41	0.52	0.63	0.81
lxxxvi)	4 750	0.26	0.31	0.41	0.51	0.62	0.81
lxxxvii)	4 800	0.26	0.31	0.40	0.51	0.62	0.81
lxxxviii)	4 850	0.25	0.30	0.40	0.51	0.62	0.80
lxxxix)	4 900	0.25	0.30	0.40	0.51	0.61	0.80
xc)	4 950	0.25	0.30	0.40	0.50	0.61	0.79
xci)	5 000	0.25	0.30	0.40	0.50	0.61	0.79

Table 24 Maximum Span for Tempered Glass Fixed on Two Opposite Sides, m

(*Clauses* 6.1.9 *and* 6.3.6.1)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Tempered Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
i)	500	0.62	0.86	1.09	1.32	1.52	1.93	2.30	2.92	3.66
ii)	550	0.60	0.82	1.04	1.26	1.45	1.84	2.20	2.78	3.49
iii)	600	0.57	0.78	0.99	1.20	1.39	1.76	2.10	2.66	3.34
iv)	650	0.55	0.75	0.95	1.16	1.33	1.69	2.02	2.56	3.21
v)	700	0.53	0.72	0.92	1.11	1.29	1.63	1.95	2.47	3.09
vi)	750	0.51	0.70	0.89	1.08	1.24	1.58	1.88	2.38	2.99
vii)	800	0.49	0.68	0.86	1.04	1.20	1.53	1.82	2.31	2.89
viii)	850	0.48	0.66	0.83	1.01	1.17	1.48	1.77	2.24	2.81
ix)	900	0.47	0.64	0.81	0.98	1.13	1.44	1.72	2.18	2.73

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Table 24 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Tempered Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
x)	950	0.45	0.62	0.79	0.96	1.10	1.40	1.67	2.12	2.65
xi)	1 000	0.44	0.60	0.77	0.93	1.08	1.37	1.63	2.06	2.59
xii)	1 050	0.43	0.59	0.75	0.91	1.05	1.33	1.59	2.01	2.52
xiii)	1 100	0.42	0.58	0.73	0.89	1.03	1.30	1.55	1.97	2.47
xiv)	1 150	0.41	0.56	0.72	0.87	1.00	1.27	1.52	1.92	2.41
xv)	1 200	0.40	0.55	0.70	0.85	0.98	1.25	1.49	1.88	2.36
xvi)	1 250	0.39	0.54	0.69	0.83	0.96	1.22	1.46	1.85	2.31
xvii)	1 300	0.39	0.53	0.67	0.82	0.94	1.20	1.43	1.81	2.27
xviii)	1 350	0.38	0.52	0.66	0.80	0.93	1.18	1.40	1.78	2.23
xix)	1 400	0.37	0.51	0.65	0.79	0.91	1.15	1.38	1.74	2.19
xx)	1 450	0.37	0.50	0.64	0.77	0.89	1.13	1.35	1.71	2.15
xxi)	1 500	0.36	0.49	0.63	0.76	0.88	1.12	1.33	1.69	2.11
xxii)	1 550	0.35	0.49	0.62	0.75	0.86	1.10	1.31	1.66	2.08
xxiii)	1 600	0.35	0.48	0.61	0.74	0.85	1.08	1.29	1.63	2.05
xxiv)	1 650	0.34	0.47	0.60	0.73	0.84	1.06	1.27	1.61	2.01
xxv)	1 700	0.34	0.46	0.59	0.71	0.82	1.05	1.25	1.58	1.98
xxvi)	1 750	0.33	0.46	0.58	0.70	0.81	1.03	1.23	1.56	1.96
xxvii)	1 800	0.33	0.45	0.57	0.69	0.80	1.02	1.21	1.54	1.93
xxviii)	1 850	0.32	0.44	0.56	0.68	0.79	1.00	1.20	1.52	1.90
xxix)	1 900	0.32	0.44	0.56	0.68	0.78	0.99	1.18	1.50	1.88
xxx)	1 950	0.32	0.43	0.55	0.67	0.77	0.98	1.17	1.48	1.85
xxxi)	2 000	0.31	0.43	0.54	0.66	0.76	0.97	1.15	1.46	1.83
xxxii)	2 050	0.31	0.42	0.54	0.65	0.75	0.95	1.14	1.44	1.81
xxxiii)	2 100	0.30	0.42	0.53	0.64	0.74	0.94	1.12	1.42	1.79
xxxiv)	2 150	0.30	0.41	0.52	0.64	0.73	0.93	1.11	1.41	1.76
xxxv)	2 200	0.30	0.41	0.52	0.63	0.73	0.92	1.10	1.39	1.74
xxxvi)	2 250	0.29	0.40	0.51	0.62	0.72	0.91	1.09	1.38	1.72
xxxvii)	2 300	0.29	0.40	0.51	0.61	0.71	0.90	1.07	1.36	1.71
xxxviii)	2 350	0.29	0.39	0.50	0.61	0.70	0.89	1.06	1.35	1.69
xxxix)	2 400	0.28	0.39	0.50	0.60	0.69	0.88	1.05	1.33	1.67

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Table 24 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Tempered Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
xl)	2 450	0.28	0.39	0.49	0.60	0.69	0.87	1.04	1.32	1.65
xli)	2 500	0.28	0.38	0.49	0.59	0.68	0.86	1.03	1.31	1.64
xlii)	2 550	0.28	0.38	0.48	0.58	0.67	0.86	1.02	1.29	1.62
xliii)	2 600	0.27	0.38	0.48	0.58	0.67	0.85	1.01	1.28	1.60
xliv)	2 650	0.27	0.37	0.47	0.57	0.66	0.84	1.00	1.27	1.59
xlv)	2 700	0.27	0.37	0.47	0.57	0.65	0.83	0.99	1.26	1.57
xlvi)	2 750	0.27	0.36	0.46	0.56	0.65	0.82	0.98	1.24	1.56
xlvii)	2 800	0.26	0.36	0.46	0.56	0.64	0.82	0.97	1.23	1.55
xlviii)	2 850	0.26	0.36	0.46	0.55	0.64	0.81	0.96	1.22	1.53
xlix)	2 900	0.26	0.36	0.45	0.55	0.63	0.80	0.96	1.21	1.52
l)	2 950	0.26	0.35	0.45	0.54	0.63	0.80	0.95	1.20	1.51
li)	3 000	0.25	0.35	0.44	0.54	0.62	0.79	0.94	1.19	1.49
lii)	3 050	0.25	0.35	0.44	0.53	0.62	0.78	0.93	1.18	1.48
liii)	3 100	0.25	0.34	0.44	0.53	0.61	0.78	0.92	1.17	1.47
liv)	3 150	0.25	0.34	0.43	0.52	0.61	0.77	0.92	1.16	1.46
lv)	3 200	0.25	0.34	0.43	0.52	0.60	0.76	0.91	1.15	1.45
lvi)	3 250	0.24	0.34	0.43	0.52	0.60	0.76	0.90	1.14	1.44
lvii)	3 300	0.24	0.33	0.42	0.51	0.59	0.75	0.90	1.14	1.42
lviii)	3 350	0.24	0.33	0.42	0.51	0.59	0.75	0.89	1.13	1.41
lix)	3 400	0.24	0.33	0.42	0.51	0.58	0.74	0.88	1.12	1.40
lx)	3 450	0.24	0.33	0.41	0.50	0.58	0.74	0.88	1.11	1.39
lxi)	3 500	0.24	0.32	0.41	0.50	0.57	0.73	0.87	1.10	1.38
lxii)	3 550	0.23	0.32	0.41	0.49	0.57	0.73	0.86	1.10	1.37
lxiii)	3 600	0.23	0.32	0.40	0.49	0.57	0.72	0.86	1.09	1.36
lxiv)	3 650	0.23	0.32	0.40	0.49	0.56	0.72	0.85	1.08	1.35
lxv)	3 700	0.23	0.31	0.40	0.48	0.56	0.71	0.85	1.07	1.34
lxvi)	3 750	0.23	0.31	0.40	0.48	0.56	0.71	0.84	1.07	1.34
lxvii)	3 800	0.23	0.31	0.39	0.48	0.55	0.70	0.84	1.06	1.33
lxviii)	3 850	0.22	0.31	0.39	0.47	0.55	0.70	0.83	1.05	1.32
lxix)	3 900	0.22	0.31	0.39	0.47	0.54	0.69	0.82	1.05	1.31

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Table 24 (*Concluded*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Tempered Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
lxx)	3 950	0.22	0.30	0.39	0.47	0.54	0.69	0.82	1.04	1.30
lxxi)	4 000	0.22	0.30	0.38	0.47	0.54	0.68	0.81	1.03	1.29
lxxii)	4 050	0.22	0.30	0.38	0.46	0.53	0.68	0.81	1.03	1.29
lxxiii)	4 100	0.22	0.30	0.38	0.46	0.53	0.67	0.80	1.02	1.28
lxxiv)	4 150	0.22	0.30	0.38	0.46	0.53	0.67	0.80	1.01	1.27
lxxv)	4 200	0.22	0.30	0.37	0.45	0.52	0.67	0.79	1.01	1.26
lxxvi)	4 250	0.21	0.29	0.37	0.45	0.52	0.66	0.79	1.00	1.25
lxxvii)	4 300	0.21	0.29	0.37	0.45	0.52	0.66	0.79	1.00	1.25
lxxviii)	4 350	0.21	0.29	0.37	0.45	0.52	0.66	0.78	0.99	1.24
lxxix)	4 400	0.21	0.29	0.37	0.44	0.51	0.65	0.78	0.98	1.23
lxxx)	4 450	0.21	0.29	0.36	0.44	0.51	0.65	0.77	0.98	1.23
lxxxi)	4 500	0.21	0.29	0.36	0.44	0.51	0.64	0.77	0.97	1.22
lxxxii)	4 550	0.21	0.28	0.36	0.44	0.50	0.64	0.76	0.97	1.21
lxxxiii)	4 600	0.21	0.28	0.36	0.43	0.50	0.64	0.76	0.96	1.21
lxxxiv)	4 650	0.20	0.28	0.36	0.43	0.50	0.63	0.75	0.96	1.20
lxxxv)	4 700	0.20	0.28	0.35	0.43	0.50	0.63	0.75	0.95	1.19
lxxxvi)	4 750	0.20	0.28	0.35	0.43	0.49	0.63	0.75	0.95	1.19
lxxxvii)	4 800	0.20	0.28	0.35	0.43	0.49	0.62	0.74	0.94	1.18
lxxxviii)	4 850	0.20	0.27	0.35	0.42	0.49	0.62	0.74	0.94	1.17
lxxxix)	4 900	0.20	0.27	0.35	0.42	0.49	0.62	0.74	0.93	1.17
xc)	4 950	0.20	0.27	0.35	0.42	0.48	0.61	0.73	0.93	1.16
xci)	5 000	0.20	0.27	0.34	0.42	0.48	0.61	0.73	0.92	1.16

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Table 25 Maximum Span for Heat Strengthened Glass Fixed on Two Opposite Sides, m

(*Clauses* 6.1.9, 6.3.5 *and* 6.3.6.1)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Heat Strengthened Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
i)	500	0.50	0.68	0.87	1.05	1.22	1.55	1.84	2.34	2.93
ii)	550	0.48	0.65	0.83	1.00	1.16	1.47	1.76	2.23	2.79
iii)	600	0.46	0.62	0.79	0.96	1.11	1.41	1.68	2.13	2.67
iv)	650	0.44	0.60	0.76	0.92	1.07	1.36	1.62	2.05	2.57
v)	700	0.42	0.58	0.73	0.89	1.03	1.31	1.56	1.97	2.47
vi)	750	0.41	0.56	0.71	0.86	0.99	1.26	1.50	1.91	2.39
vii)	800	0.39	0.54	0.69	0.83	0.96	1.22	1.46	1.85	2.31
viii)	850	0.38	0.52	0.67	0.81	0.93	1.19	1.41	1.79	2.24
ix)	900	0.37	0.51	0.65	0.79	0.91	1.15	1.37	1.74	2.18
x)	950	0.36	0.50	0.63	0.76	0.88	1.12	1.34	1.69	2.12
xi)	1 000	0.35	0.48	0.61	0.75	0.86	1.09	1.30	1.65	2.07
xii)	1 050	0.34	0.47	0.60	0.73	0.84	1.07	1.27	1.61	2.02
xiii)	1 100	0.34	0.46	0.59	0.71	0.82	1.04	1.24	1.57	1.97
xiv)	1 150	0.33	0.45	0.57	0.69	0.80	1.02	1.21	1.54	1.93
xv)	1 200	0.32	0.44	0.56	0.68	0.79	1.00	1.19	1.51	1.89
xvi)	1 250	0.32	0.43	0.55	0.67	0.77	0.98	1.16	1.48	1.85
xvii)	1 300	0.31	0.42	0.54	0.65	0.75	0.96	1.14	1.45	1.82
xviii)	1 350	0.30	0.42	0.53	0.64	0.74	0.94	1.12	1.42	1.78
xix)	1 400	0.30	0.41	0.52	0.63	0.73	0.92	1.10	1.40	1.75
xx)	1 450	0.29	0.40	0.51	0.62	0.71	0.91	1.08	1.37	1.72
xxi)	1 500	0.29	0.40	0.50	0.61	0.70	0.89	1.06	1.35	1.69
xxii)	1 550	0.28	0.39	0.49	0.60	0.69	0.88	1.05	1.33	1.66
xxiii)	1 600	0.28	0.38	0.49	0.59	0.68	0.86	1.03	1.31	1.64
xxiv)	1 650	0.27	0.38	0.48	0.58	0.67	0.85	1.01	1.29	1.61
xxv)	1 700	0.27	0.37	0.47	0.57	0.66	0.84	1.00	1.27	1.59
xxvi)	1 750	0.27	0.37	0.46	0.56	0.65	0.83	0.98	1.25	1.56
xxvii)	1 800	0.26	0.36	0.46	0.56	0.64	0.81	0.97	1.23	1.54
xxviii)	1 850	0.26	0.36	0.45	0.55	0.63	0.80	0.96	1.21	1.52
xxix)	1 900	0.26	0.35	0.45	0.54	0.62	0.79	0.94	1.20	1.50
xxx)	1 950	0.25	0.35	0.44	0.53	0.62	0.78	0.93	1.18	1.48

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Table 25 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Heat Strengthened Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
xxxi)	2 000	0.25	0.34	0.43	0.53	0.61	0.77	0.92	1.17	1.46
xxxii)	2 050	0.25	0.34	0.43	0.52	0.60	0.76	0.91	1.15	1.45
xxxiii)	2 100	0.24	0.33	0.42	0.51	0.59	0.75	0.90	1.14	1.43
xxxiv)	2 150	0.24	0.33	0.42	0.51	0.59	0.75	0.89	1.13	1.41
xxxv)	2 200	0.24	0.33	0.41	0.50	0.58	0.74	0.88	1.11	1.40
xxxvi)	2 250	0.24	0.32	0.41	0.50	0.57	0.73	0.87	1.10	1.38
xxxvii)	2 300	0.23	0.32	0.41	0.49	0.57	0.72	0.86	1.09	1.36
xxxviii)	2 350	0.23	0.32	0.40	0.49	0.56	0.71	0.85	1.08	1.35
xxxix)	2 400	0.23	0.31	0.40	0.48	0.56	0.71	0.84	1.07	1.34
xl)	2 450	0.23	0.31	0.39	0.48	0.55	0.70	0.83	1.05	1.32
xli)	2 500	0.22	0.31	0.39	0.47	0.54	0.69	0.82	1.04	1.31
xlii)	2 550	0.22	0.30	0.38	0.47	0.54	0.68	0.82	1.03	1.30
xliii)	2 600	0.22	0.30	0.38	0.46	0.53	0.68	0.81	1.02	1.28
xliv)	2 650	0.22	0.30	0.38	0.46	0.53	0.67	0.80	1.01	1.27
xlv)	2 700	0.21	0.29	0.37	0.45	0.52	0.67	0.79	1.00	1.26
xlvi)	2 750	0.21	0.29	0.37	0.45	0.52	0.66	0.79	1.00	1.25
xlvii)	2 800	0.21	0.29	0.37	0.45	0.51	0.65	0.78	0.99	1.24
xlviii)	2 850	0.21	0.29	0.36	0.44	0.51	0.65	0.77	0.98	1.23
xlix)	2 900	0.21	0.28	0.36	0.44	0.51	0.64	0.76	0.97	1.22
l)	2 950	0.21	0.28	0.36	0.43	0.50	0.64	0.76	0.96	1.21
li)	3 000	0.20	0.28	0.35	0.43	0.50	0.63	0.75	0.95	1.19
lii)	3 050	0.20	0.28	0.35	0.43	0.49	0.63	0.75	0.95	1.19
liii)	3 100	0.20	0.27	0.35	0.42	0.49	0.62	0.74	0.94	1.18
liv)	3 150	0.20	0.27	0.35	0.42	0.48	0.62	0.73	0.93	1.17
lv)	3 200	0.20	0.27	0.34	0.42	0.48	0.61	0.73	0.92	1.16
lvi)	3 250	0.20	0.27	0.34	0.41	0.48	0.61	0.72	0.92	1.15
lvii)	3 300	0.19	0.27	0.34	0.41	0.47	0.60	0.72	0.91	1.14
lviii)	3 350	0.19	0.26	0.34	0.41	0.47	0.60	0.71	0.90	1.13
lix)	3 400	0.19	0.26	0.33	0.40	0.47	0.59	0.71	0.90	1.12
lx)	3 450	0.19	0.26	0.33	0.40	0.46	0.59	0.70	0.89	1.11

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Table 25 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Heat Strengthened Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
lxi)	3 500	0.19	0.26	0.33	0.40	0.46	0.58	0.70	0.88	1.11
lxii)	3 550	0.19	0.26	0.33	0.40	0.46	0.58	0.69	0.88	1.10
lxiii)	3 600	0.19	0.25	0.32	0.39	0.45	0.58	0.69	0.87	1.09
lxiv)	3 650	0.18	0.25	0.32	0.39	0.45	0.57	0.68	0.86	1.08
lxv)	3 700	0.18	0.25	0.32	0.39	0.45	0.57	0.68	0.86	1.08
lxvi)	3 750	0.18	0.25	0.32	0.38	0.44	0.56	0.67	0.85	1.07
lxvii)	3 800	0.18	0.25	0.32	0.38	0.44	0.56	0.67	0.85	1.06
lxviii)	3 850	0.18	0.25	0.31	0.38	0.44	0.56	0.66	0.84	1.05
lxix)	3 900	0.18	0.24	0.31	0.38	0.44	0.55	0.66	0.84	1.05
lxx)	3 950	0.18	0.24	0.31	0.37	0.43	0.55	0.66	0.83	1.04
lxxi)	4 000	0.18	0.24	0.31	0.37	0.43	0.55	0.65	0.83	1.03
lxxii)	4 050	0.18	0.24	0.31	0.37	0.43	0.54	0.65	0.82	1.03
lxxiii)	4 100	0.17	0.24	0.30	0.37	0.42	0.54	0.64	0.82	1.02
lxxiv)	4 150	0.17	0.24	0.30	0.37	0.42	0.54	0.64	0.81	1.02
lxxv)	4 200	0.17	0.24	0.30	0.36	0.42	0.53	0.64	0.81	1.01
lxxvi)	4 250	0.17	0.23	0.30	0.36	0.42	0.53	0.63	0.80	1.00
lxxvii)	4 300	0.17	0.23	0.30	0.36	0.41	0.53	0.63	0.80	1.00
lxxviii)	4 350	0.17	0.23	0.29	0.36	0.41	0.52	0.62	0.79	0.99
lxxix)	4 400	0.17	0.23	0.29	0.36	0.41	0.52	0.62	0.79	0.99
lxxx)	4 450	0.17	0.23	0.29	0.35	0.41	0.52	0.62	0.78	0.98
lxxxi)	4 500	0.17	0.23	0.29	0.35	0.41	0.52	0.61	0.78	0.98
lxxxii)	4 550	0.17	0.23	0.29	0.35	0.40	0.51	0.61	0.77	0.97
lxxxiii)	4 600	0.16	0.23	0.29	0.35	0.40	0.51	0.61	0.77	0.97
lxxxiv)	4 650	0.16	0.22	0.28	0.35	0.40	0.51	0.60	0.77	0.96
lxxxv)	4 700	0.16	0.22	0.28	0.34	0.40	0.50	0.60	0.76	0.95
lxxxvi)	4 750	0.16	0.22	0.28	0.34	0.39	0.50	0.60	0.76	0.95
lxxxvii)	4 800	0.16	0.22	0.28	0.34	0.39	0.50	0.59	0.75	0.94
lxxxviii)	4 850	0.16	0.22	0.28	0.34	0.39	0.50	0.59	0.75	0.94
lxxxix)	4 900	0.16	0.22	0.28	0.34	0.39	0.49	0.59	0.75	0.94
xc)	4 950	0.16	0.22	0.28	0.33	0.39	0.49	0.59	0.74	0.93

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Table 25 (*Continued*)

Sl No.	Design Wind Pressure N/m^2	Nominal Thickness of Heat Strengthened Glass, *T* mm								
		3	4	5	6	8	10	12	15	19
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
xci)	5 000	0.16	0.22	0.27	0.33	0.38	0.49	0.58	0.74	0.93

6.2 Installation

6.2.1 Basic installation requirements and materials used for glass are given hereunder:

NOTES

1 The provisions under this item do not include the use of other methods or systems for glazing, provided the alternate method or system can be demonstrated to satisfy the requirements for correctly supporting the glass within the frame, or glazing system.

2 Patent and other proprietary systems are not described/ covered here.

3 Installation technique for frameless glazing system is not covered here.

6.2.2 *Site Working and Glass Machining Operation*

6.2.2.1 Heat-strengthened and toughened glass shall not be cut or worked after heat treatment. All necessary cutting, drilling, notching and edge-working shall be carried out to correct dimensions prior to value addition to the glass, such as toughening/tempering/heat strengthening. The edges and surfaces of all glass types shall not be damaged during fixing.

6.2.3 *Dimensional Requirement*

The dimensions for edge clearance, edge cover, front and back clearance, rebate depth is selected in accordance with the requirements in **4.5.2**.

6.2.4 *Glazing Materials*

6.2.4.1 *Compatibility of materials*

A glazing material shall be used only when compatible with contiguous materials, including the rebate surface finish, setting or location blocks, distance pieces and glass type.

6.2.4.2 *Application of materials*

The application of glazing materials shall be in accordance with relevant/available Indian Standard or manufacturer's recommendations.

6.2.4.3 *Life expectancy of materials*

A glazing material shall only be used where its life expectancy (durability) has been established. The manufacturers' advice/guarantee should be sought for information regarding life expectancy.

6.2.5 *Setting Blocks*

The number and location of setting blocks shall be as shown in Fig. 14 and Fig 16. Generally, setting blocks shall be,

a) positioned at quarter points or not less than 30 mm from the corner, whichever is less;

b) the minimum width of each setting block shall be not less than the glass thickness; and

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c) if a glazing bridge is used then the thickness of the glazing bridge can be added to the thickness of the setting block to achieve the minimum gap of 6 mm between the edge of glass and the framing member.

Setting blocks shall be located to equally support all panes of glass, and shall be fixed to prevent displacement during installation and service. The minimum length of each setting block (or two blocks side by side) shall be 25 mm in length for every square metre of glass area, with a minimum length of 50 mm. When wood is used as the material for setting blocks, only the seasoned ones should be used as green wood may shrink laterally (in the process of attaining equilibrium moisture content over a period of time) and result in loosened installations.

For example, for a 3.0 m^2 glass area, 3.0 mm × 25 mm = 75 mm long, that is, 75 mm is the length of each setting block.

Setting blocks shall be of resilient, load-bearing, non-absorbent, rot-proof, and material that is compatible with all other glazing materials conforming to available standards (if any) that may come into contact with the blocks.

NOTES

1 Setting blocks are used between the bottom edge of the unit and the frame to centralize and equally support both panes of glass.

2 Setting block width and location should not restrict water drainage.

3 Extruded rubber material with 80 to 90 shore A hardness is recommended.

4 Shaped setting blocks will be required for a glazing platform.

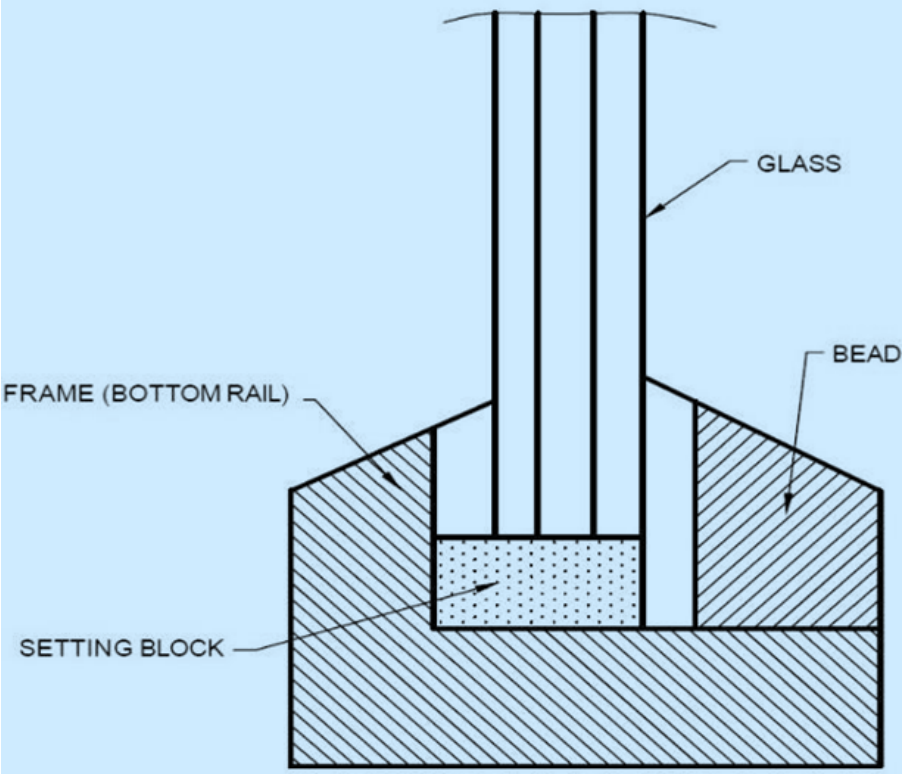

FIG. 14 POSITION OF SETTING BLOCK

6.2.6 *Location Blocks*

The number and position of location blocks shall be as shown in Fig. 15 and Fig 16.

Location blocks shall be,

a) of minimum of 25 mm in length;

b) at least as wide as the glass thickness;

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c) positively located to prevent displacement in service; and

d) sufficiently resilient to accommodate movement within the frame, without imposing stress on the glass, and of resilient, non-absorbent material.

NOTES

1 Location blocks are used between the edges of the glass and the frame to prevent movement of the glass within the frame by thermal expansion or when the window or door is opened or closed. They are required to prevent the weight of the glass from causing the frame to become out of square.

2 Extruded rubber material with 55 to 65 shore A hardness is recommended.

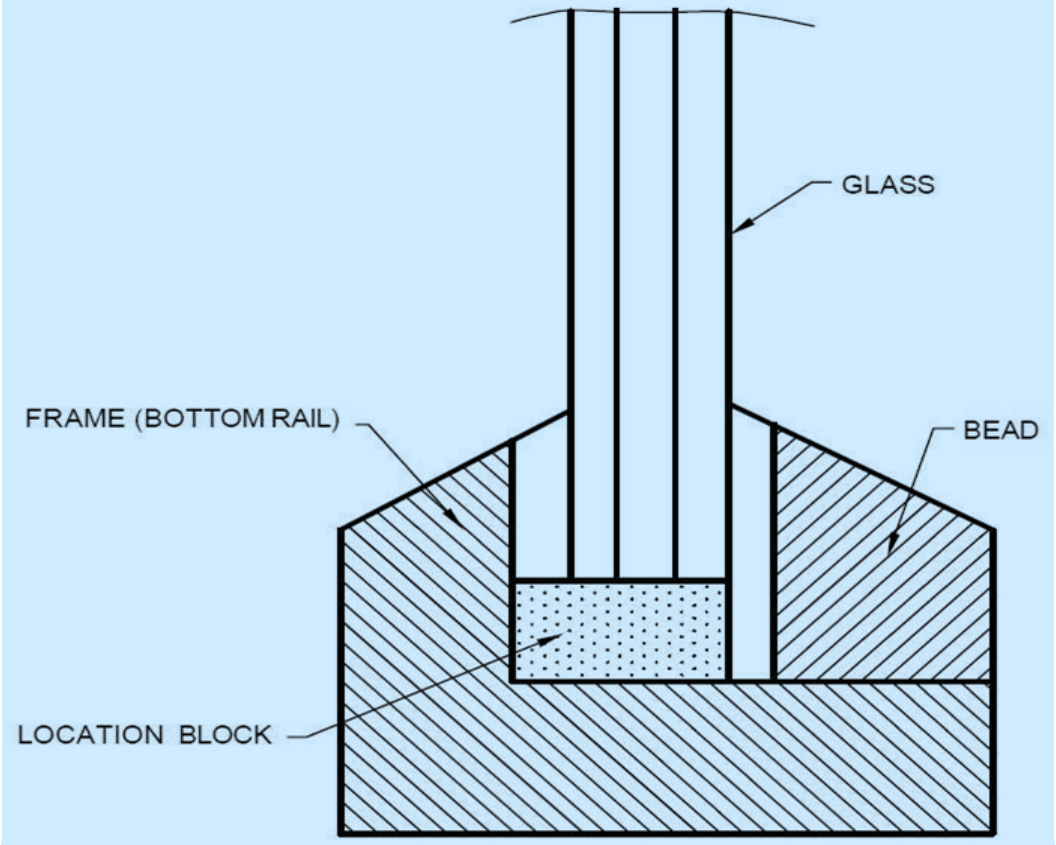

FIG. 15 POSITION OF LOCATION BLOCK

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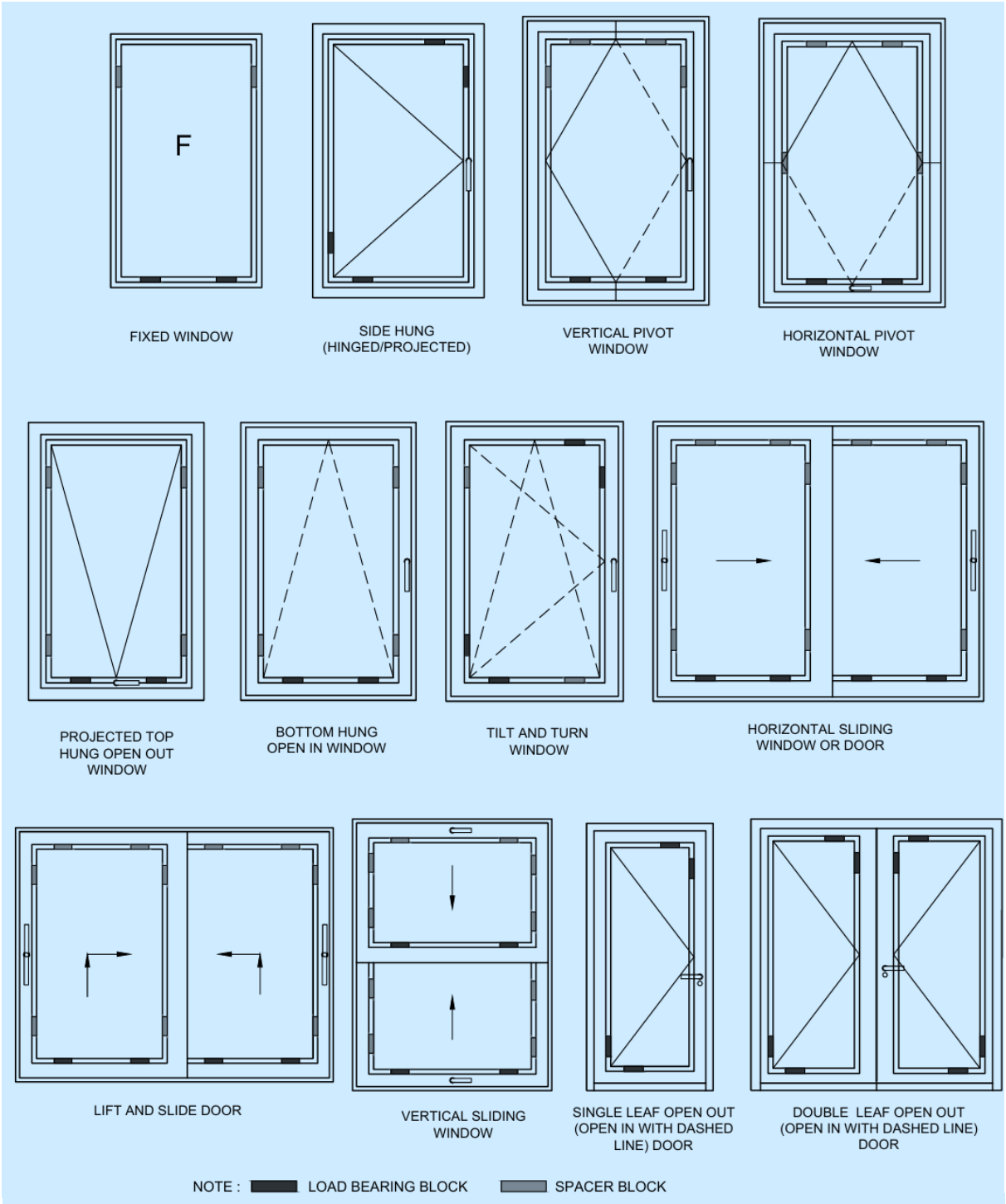

FIG. 16 NUMBER AND POSITION OF SETTING AND LOCATION/GLAZING BLOCK

6.2.7 *Distance Pieces*

Distance pieces, as shown in Fig. 17, used where required, shall be,

a) of resilient, non-absorbent material;

b) 25 mm long and of a height to suit the depth of the rebate and the method of glazing; and

c) spaced opposite each other, approximately 50 mm from each corner at intervals of not more than 300 mm.

The thickness shall be equal to the front and back clearance, to retain the glass firmly in the frame.

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NOTES

1 Distance pieces are required to prevent displacement of glazing compounds or sealant by external loading, such as wind pressure.

2 Extruded rubber material with 55 to 65 shore A hardness is required.

6.2.8 *Rebates and Grooves for Installation*

Rebates, grooves and beads shall be cleaned and free from grease, moisture and other contaminants. All sealant surfaces shall be primed or sealed in accordance with the recommendations of the manufacturer of the glazing material.

The number and location of setting blocks and location blocks shall be as shown in Fig. 14 to Fig. 16, wherever necessary to maintain the requisite edge clearance. Each block shall support the full thickness of the glass.

6.2.9 *Glazing Beads*

Where used, glazing beads shall be capable of restraining the glass under all design forces.

6.2.10 *Structural Sealants*

Structural silicone shall be installed such that a full adhesive bond to the substrate is achieved. Application of structural silicone may require prior cleaning and or priming of the substrate.

Applied structural silicones shall not be installed adjacent to other materials that may be chemically incompatible with the structural sealant and cause a loss of adhesion or adverse chemical changes within the structural silicone that could lead to a loss of strength in the structural silicone. For guidance on the installation of structural silicone, reference to available standards/literatures may be made.

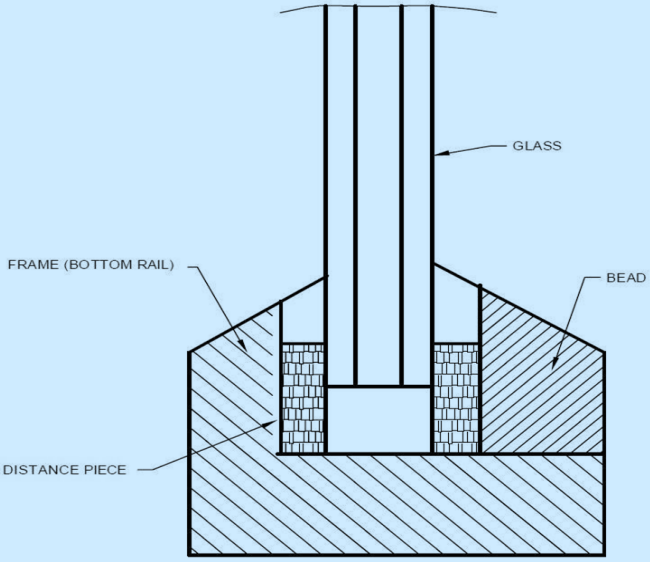

FIG. 17 POSITION OF DISTANCE PIECES

6.3 Design Criteria

6.3.1 *Frame Support Condition*

6.3.1.1 *Maximum area of glass panels subjected to wind loading*

The maximum span for a given standard nominal thickness of ordinary annealed, laminated, heat-strengthened and toughened glass for a given panel size shall be determined in accordance with **6.3.2**, **6.3.3** and **6.3.4** as applicable for the relevant support conditions. For heat strengthened laminated and toughened laminated glass, maximum allowable span shall be determined in accordance with **6.3.5**. For 3 mm monolithic annealed glass, the maximum area shall not exceed 0.85 m^2.

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The minimum thickness of the nominal glass size shall be as per the accepted practice in accordance with Indian Standard specifications.

For laminated glass composites, the combined minimum thickness of the glass sheets may be used excluding the interlayer thickness. Alternatively, reference to specialist literature may be drawn for the same purpose.

6.3.2 *Rectangular Glass Supported on All Four Edges*

For rectangular glass supported on all four edges, the maximum area for the design wind pressure shall be determined using the corresponding tables provided for each type of glass. Linear interpolation may be used for any value in between those given therein while maintaining the aspect ratio.

6.3.3 *Rectangles of Glass Supported on Two Opposite Edges*

For rectangles of glass supported on two opposite edges, the maximum glass area for the design wind pressure shall be determined from Table 22 to Table 25 for ordinary annealed, laminated, toughened and heat-strengthened glass, respectively.

6.3.4 *Rectangles of Glass Supported on Three Edges*

For rectangles of glass supported on three edges, the maximum glass area shall be determined as for two-edge support spanning along the unsupported edge. Alternatively, basic engineering principles shall be adopted in determining the glass thickness for all applications where only three edges of the panel are supported.

NOTE — Butt glazing of adjacent panels in the same plane should not be considered as a support.

6.3.5 *Heat-Strengthened Laminated and Toughened Laminated Glass*

For heat-strengthened laminated and toughened laminated glass, the maximum span for a given standard nominal thickness for a given panel size shall be determined from Table 21 and Table 25, and Table 19 and Table 24, respectively with the design wind pressure being divided by the appropriate glass type factor, P_f (*see* Table 14), as applicable for the relevant support conditions.

6.3.6 *Serviceability Checks*

6.3.6.1 Glass complying to Table 17 to Table 25 are deemed to meet, the serviceability deflection limits specified. In the absence of any other specified values, the following values may be used for guidance.

6.3.6.2 *Maximum allowable deflection of framing members*

a) *For mullions*:

   1) Single height glazing — Span/175 or 19 mm, whichever is less.

   2) Double height glazing — For spans up to 4 110 mm, same as single height glazing; and for spans above 4 110 mm, the same shall be (Span/240) + 6.35 mm.

b) *For transoms*:

   1) Span/500 or 3 mm, whichever is the least for dead load.
   2) Span/175 or 19 mm, whichever is the least for wind load.

c) *Deflection at the centre of the glass*:

   1) Monolithic glass −Shortest span/60 or 19 mm, whichever is the least.

   2) Double glazed unit −Shortest span/90 or 19 mm, whichever is the least.

d) *Deflection at edge of the glass*:

   1) Shall be limited to 15 mm.

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6.3.6.2.1 *Dielectric separation of dissimilar metals*

When the framing member for the glazing is aluminium and reinforcement is required to meet the deflection criteria, the reinforcement may be of aluminium or galvanized steel (red oxide coating being not recommended) with an insulating material between them at contact points. It is recommended to choose materials as close to each other in the galvanic series as possible.

6.3.7 *Ultimate Stresses*

The ultimate stresses for various types of glass are given in Table 26.

Table 26 Ultimate Limit State Design Stresses for Glass Subjected to Wind Loading

(*Clause* 6.3.7)

Sl No.	Glass Type	Standard Nominal Thickness '*T*'	Ultimate Limit State Design Stress at Given Location	
		mm	Away from Edge MPa	At Edge MPa
(1)	(2)	(3)	(4)	(5)
i)	Annealed	3	41.00	32.8
		4	38.99	31.19
		5	37.45	29.96
		6	36.2	28.96
		8	34.33	27.46
		10	32.8	26.24
		12	31.57	25.25
		15	30.15	24.12
		19	28.72	22.98
		25	26.96	21.57
ii)	Toughened	4	97.47	77.97
		5	93.61	74.97
		6	90.49	72.39
		8	85.82	68.65
		10	82.01	65.61
		12	78.91	63.13
		15	75.37	60.3
		19	71.81	57.45
		25	67.41	53.93
iii)	Heat strengthened	3	65.6	52.48
		4	62.38	49.9
		5	59.91	47.93
		6	57.91	46.33
		8	54.92	43.94
		10	52.48	41.99
		12	50.51	40.4

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Table 26 Ultimate Limit State Design Stresses for Glass Subjected to Wind Loading

(*Clause* 6.3.7)

Sl No.	Glass Type	Standard Nominal Thickness '*T*'	Ultimate Limit State Design Stress at Given Location	
		mm	Away from Edge MPa	At Edge MPa
(1)	(2)	(3)	(4)	(5)
iv)	Annealed laminated	5	37.73	30.18
		6	36.43	29.14
		8	34.41	27.53
		10	32.87	26.3
		12	31.62	25.3
		16	29.75	23.8
		20	28.23	22.58
		24	26.99	21.59

6.3.8 *Insulating Glass Units*

6.3.8.1 *Determining bite of structural silicone*

The structural bite requirement is directly proportional to the wind load and the dimension of glass. Higher the wind load and larger the dimensions of the glass are, the greater the amount of structural bite required. The controlling variables which affect the structural bite requirement are the maximum short span dimension of glass and the design wind load which the structural glazing system shall be designed to accommodate.

a) *Structural bite calculation for wind load and glass dimension*

$$\text{Minimum Structural Bite} = \frac{Glass\ Short\ span\ dimension \times wind\ load \times 0.5}{Maximum\ allowable\ design\ stress}$$

NOTES

1 Glass' short span dimension (SSD) is the shorter of the two dimensions (in m) of the rectangular glass panel.

2 Wind load is the maximum wind pressure, in Pascal, for a return period of 10 years based on local regulations.

3 The maximum allowable design stress for type of the structural sealant is selected as per manufacturer's recommendations.

4 The minimum structural bite shall be, not less than 6 mm for single glass and for double-glazed units (DGU) with equal glass thickness.

b) *Determination of thickness of structural silicone* — With a sealed IGU, there may be load sharing between the two panes of glass. If so, and if both panes are of the same thickness, the lateral load (*P*) is shared almost equally; thus the secondary seal bite is calculated as one half of that of the structural sealant bite, but should meet the minimum requirement. If of unequal thickness, the load shared by each pane will vary, depending on the difference in thicknesses. Further details are to be obtained from the manufacturer.

c) *Structural bite calculation for dead load*

$$Minimum\ Bite\ (m) = \frac{Weight\ of\ glass\ (kg)}{Perimeter\ of\ the\ glass\ (m) \times Seleant\ dead\ load\ design\ strength\ (kg/m^2)}$$

$$= \frac{\rho g T A}{Ps}$$

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where

ρ = 2 500 kg/m^3 is the specific mass of flat glass corresponding to approximately of specific weight;

g = 9.81 m/s^2;

s = Allowable design stress for dead load (DL) for selected sealant considered for design, Pa;

T = thickness of the bite, m;

P = perimeter (if the horizontal frame members do not support the glass or deflect under the dead load of the glass, consider 2 times of its height only); and

A = area of glass panel, m^2.

6.3.8.2 *Maximum allowable span for glass and type*

Maximum allowable span is dependent on the thickness of the glass selected and type of glass used (for example annealed, toughened or heat strengthened glass). The maximum span is calculated in accordance with **6.1.9** (*see* Fig. 18 for details).

6.3.9 *Safety During Installation*

Glass and the coating layer (if any) should not be exposed to/in direct contact with aggressive materials such as acid, solvent, cement mortar, etc.

The fixing system of the panel should, in no case, cause shear stresses in the panel, and particularly the bonding interface between the glass sheet and the frame, either under the effect of external forces or due to differential expansion of components.

The protection of glass panel by means of a polyethylene sheet is recommended and shall provide ventilation for the glass in order to prevent thermal breakage.

Workers involved in the installation process shall be provided with adequate training and awareness about safety practices related to the handling and installation of glass panels. They shall be equipped with appropriate personal protective equipment (PPE), including but not limited to safety gloves, helmets, safety goggles, and footwear designed for the work environment, to minimize risks of injury during installation activities.

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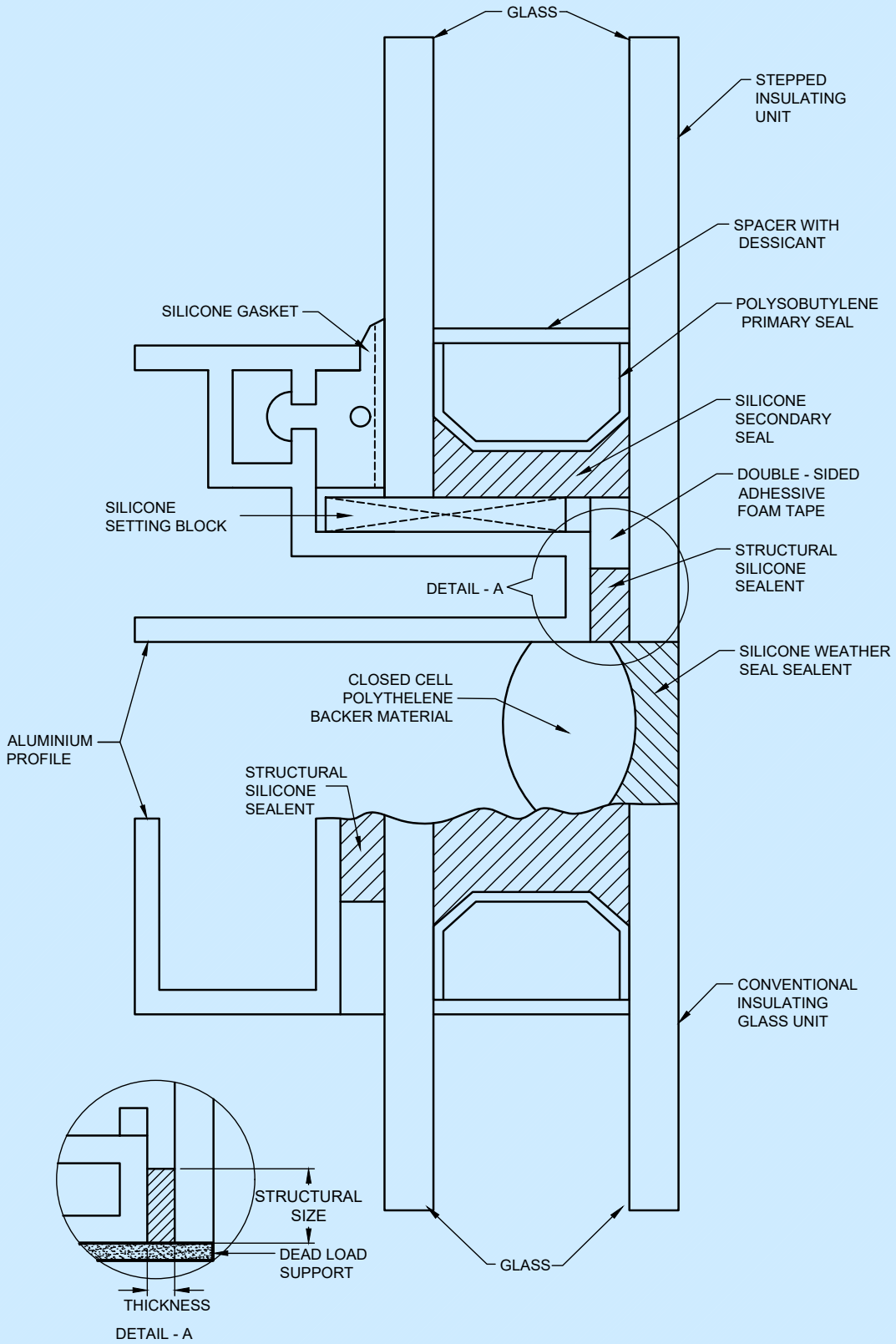

FIG. 18 TYPICAL STRUCTURAL GLAZING DETAIL

NOTE — Dead load support for the glass or infill shall be determined based on 6.3.8.1 (c). Minimum 50 percent of thickness of external glass pane shall be supported at the bottom, in the case of structural glazed systems.

FIG. 18 TYPICAL STRUCTURAL GLAZING DETAIL

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6.3.10 *Handling and Transportation*

6.3.10.1 *Transportation*

During the transportation of the flat glass (clear, tinted), coated glass (solar control and mirror), lacquered or painted glass the following methods (if other standard packaging specifications are not available) of storage, packing and interleaving shall be followed:

a) Flat glass shall be transported vertically;

b) Glass panes should not come into direct contact with each other by using appropriate interleaving such as powder or foam or paper. The lacquer opacified/enamel coated/screen printed glasses shall be separated by paper or foam; and

c) The packing and packing material shall be protected from water and if the glass is wrapped and sealed, the seal should remain closed until the product is used and precautions taken to not damage the packing while handling with a hoisting apparatus.

Workers involved in the transportation of glass shall be provided with adequate training on transportation safety practices. They shall be equipped with appropriate personal protective equipment (PPE), including but not limited to safety gloves, helmets, and safety footwear to minimize risks of injury during transportation activities.

6.3.10.2 *Handling*

During the handling of the flat glass (clear, tinted), coated glass (solar control and mirror), lacquered or painted glass, the following guidelines shall be followed:

a) Glass shall be handled with dry, clean gloves, specifically after washing to prevent stain marks on them;

b) Care shall be taken to avoid contact or friction of the coating or the opacified/ enamel/screen printed side with any rough surface or hard objects;

c) better practice is to handle glass with vacuum cups, and ensure that the vacuum cups and the glass are clean; and

d) It is recommended to keep the glass sheets clean if these are to be stored again before their processing or installation in the facade.

6.3.11 *Storage at Site*

To prevent staining, the storage area at site shall be,

a) a dry, well ventilated location at a sufficiently constant temperature;

b) protected from rain and running water;

c) protected from wide changes in temperature or humidity;

d) protected from alkaline material;

e) protected from direct contact of aggressive products such as corrosive vapours, acids, etc; and

f) glass panes are stored vertically on a soft surface such as wooden boards/battens.

6.3.12 *Storage after Cutting*

6.3.12.1 While stacking of coated glass/ mirror in the unpacked condition, the glass surface shall be facing towards the front (paint/coating backing towards the back).

6.3.12.2 Direct stacking of the mirror/coated glass one above the other should be prohibited and suitable inter-leaving material as mentioned in **6.3.10.1** should be used between two glass panes.

6.3.12.3 The support material for glass storage should not have affinity towards moisture absorption.

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6.4 Fire Safety

6.4.1 This Section covers the requirement and selection of fire safety glass for buildings. General fire safety provisions for buildings may be drawn from Part F 'Fire and Life Safety' of this NBCS. It is important for any fire rated glass to provide sufficient amount of human impact safety feature even during fire or in general situations. The level of human impact safety of glass in buildings shall be in accordance with 7.1. It is necessary that the fire rated system shall withstand the standard time-temperature curve during a fire test and shall give adequate time to the occupants for safe evacuation. This Section does not deal with the safety and security of people or goods in relation to risks of,

a) vandalism, riots, burglary or break in protection;

b) protection from explosion (terrorist attack);

c) natural disasters like earthquakes, cyclone, etc; and

d) plastic glazing, safety and security glazing, etc.

The requirements given here are minimum fire resistance requirements. In circumstances wherein the requirements of other Indian Standards exceed the requirements of this Section, the higher requirements shall be applicable.

6.4.2 *Glass for Fire Resistance*

6.4.2.1 Glazing system shall be so constructed, treated, or combined with other materials as to reduce loss of life or property during an event of fire, in comparison with ordinary sheet or flat glass, thereby preventing the passage of flame, hot gases, and smoke.

6.4.2.2 The following are the products in the scope of this Section that may be used for fire resistance provided they meet the requirements of accepted standard [C-8(18)]:

a) Wired fire resistant glass;

b) Glass-ceramic fire resistant glass;

c) Borosilicate fire resistant glass;

d) Thermally super toughened soda lime silicate fire resistant glass;

e) Laminated fire resistant glass;

f) Intumescent interlayer laminated fire resistant glass;

g) Gel-filled fire resistant glass; and

h) Low-E coated fire resistant glass.

6.4.2.3 *Testing procedures*

Any fire rated glazed building element needs to be tested as per accepted standard [C-8(19)]. The fire rating criteria of any building element is given in Part F 'Fire and Life Safety' of this NBCS.

6.4.2.4 *Identification of glass used for fire resistance*

a) All glass used for resisting fire shall conform to and preferably certified against available Indian Standards;

b) Since a fire resistant glass is not easily distinguishable from normal types of glass, it is imperative that every fire rated glass panel/unit should preferably have an indelible mark on all the panels/units of fire resistant glass showing the name of the manufacturer and the name of the product; and

c) Rating of the glass panel may also be provided, although the rating of glazing system is of primary concern.

The glass used for fire resistance should be in accordance with the requirements of Part F 'Fire and Life Safety' of this NBCS.

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6.4.3 *Precautions*

6.4.3.1 Fire resistant glass alone shall not be enough to protect occupants from fire. Glass is only one component of the complete system.

6.4.3.2 Fire test report conducted for one application shall not be substituted for any other application. For example, a report for a partition shall not be used for a door application although the glass used and the fire ratings are the same. Similarly, approval in vertical orientation cannot be considered for acceptance in horizontal or inclined orientation.

6.4.3.3 The maximum glass size as mentioned in the test report should not be exceeded in practice with respect to the bigger linear dimension or area, unless otherwise supported with valid third-party tested evidence. The aspect ratio may be modified provided that none of the linear sides has a dimension greater than that of the largest dimension of originally tested glass.

6.4.3.4 The dimensions of the profile need to be the same as that tested. The sheet thickness of the profile can be increased with no change in the tested fire rating.

6.4.3.5 In case of any change in hardware, the new assembly should be retested as per the same standard and/or specification requirements. In the case of replacement of hardware, the same specification and make of the hardware can be used without retesting of the whole assembly.

6.4.3.6 Factors like edge cover, types of gaskets and type of fixtures also play a very vital role in achieving the fire performance of the building element. These have to be used and installed as per the test report/available standards.

6.4.3.7 Buildings having non-structural joints between floor slabs and external perimeter walls, pose an associated risk of smoke and fire traversing between adjacent floors is not uncommon. The use of suitable materials to seal such joints needs careful consideration towards fire safety as the same is part of the compartment/fire barrier.

All gaps between floor-slabs and facade assembly shall be sealed at all levels by tested system of equal fire rating as that of floor slab to prevent fire and smoke propagation from one floor to another. Accepted standard [C-8(20)] specifies the test method to determine the fire resistance performance of perimeter fire barrier joint system based on the length of time the system being tested to resist the fire before development of through openings or flaming and transmission of heat on the unexposed surface.

6.4.4 *Classifications of Fire Resistant Glass*

See **4** of accepted standard [C-8(18)].

6.4.5 *Cleaning and Maintenance of Fire-Rated Glazing System*

Routine maintenance and inspection are essential for fire rated glazing to ensure their optimal performance during a fire emergency. For more details in this regard, Part F ‘Fire and Life Safety’ of this NBCS may be referred.

7 SAFETY RELATED TO HUMAN IMPACT, ANTI-FALLOUT, WINDOW RESTRICTOR FOR CHILDREN AND DYNAMIC WIND RESTRAINT

7.1 Safety Related to Human Impact

7.1.1 *General*

The provisions herein are limited to the behaviour of glass when subjected to various kinds of human impact, precautions against risk of fall and falling glass. Conditions outside of ‘human impact’ are not covered under the scope of this Section.

This Section does not assume that the glass used in accordance to this Section will not break under all human impact conditions, rather,

a) it will not break under most likely forms of human impact; and

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b) even if it breaks, the likelihood of cutting or piercing injuries will be minimised by virtue of the protection given to the glass, or by the limited size, or by the fracture characteristics of the glass.

Further, this Section does not deal with the safety and security of people or goods in relation to risks of,

a) vandalism, riots, burglary or break in protection;

b) fire arm protection;

c) protection from explosion (terrorist attack);

d) natural disasters like earthquakes, cyclone, fire, etc; and

e) plastic glazing material, safety and security glazing, etc.

In circumstances wherein the requirements of any referred standard(s) exceeds the requirements of this Section, the higher requirement shall be applicable.

The use of this Section shall also be in conformity with all other relevant standards on fire safety, structural stability, natural disasters, safety and security, etc in force.

7.1.2 *Safety Glass*

Glazing material that are constructed, treated, or combined with other materials shall reduce, in comparison with ordinary sheet/ float/ plate glass, the likelihood of injury to persons by objects from exterior sources or by these safety glasses when they may be cracked or broken.

The following are the products in the scope of this Section that may be used in safety glazing provided they meet the conformity requirements of accepted standard [C-8(7)]. Safety glass according to this Section shall be of following types:

a) Toughened safety (tempered) glass; and

b) Laminated safety glass.

7.1.2.1 All safety glasses shall meet the requirement of tests specified in accepted standard [C-8(7)].

7.1.2.2 In atria where there is no impact loads above 10 m, suitable toughened glass is adequate. *See* **4.2.9**, in case of use of glass fins as a structural member.

7.1.3 *Critical Location*

Where any glazing is within 1.5 m above the floor level of a building, it is considered likely to be subjected to human impact and hence, shall comply with the human impact safety requirements of this Section. Safety glazing should also be used,

a) where there is danger of falling infill glass material(s) from overhead glazing;

b) the danger of falling due to a change in floor level; and

c) in case of balustrades, stairs and floors.

Based on typical accidents in glazed buildings, certain locations in buildings are found to be more vulnerable to human impact than others. Some of such critical locations are given below and shown in Fig. 19, where necessary precautions have to be followed:

1) in-and-around doors, low windows;

2) door side panels;

3) panels mistaken for a doorway or opening;

4) panels at low levels in walls and partitions;

5) bathrooms;

6) building associated with special activities, for example, gymnasia, enclosed swimming pools, etc;

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7) schools and child care facilities; and

8) nursing homes and care facilities for the aged and infirmed.

Suitable precautions should be taken to reduce the injuries that can result from glass breakage by,

i) selecting glass of a suitable type, thickness and size;

ii) enhancing a person's awareness of presence of glass by making glass visible (manifestation of glass); and

iii) minimizing manual handling of large pieces of glass during installation.

Based on the above, a comprehensive suggestion on type of glass to be used at different critical location/cases is provided in Table 27 for information.

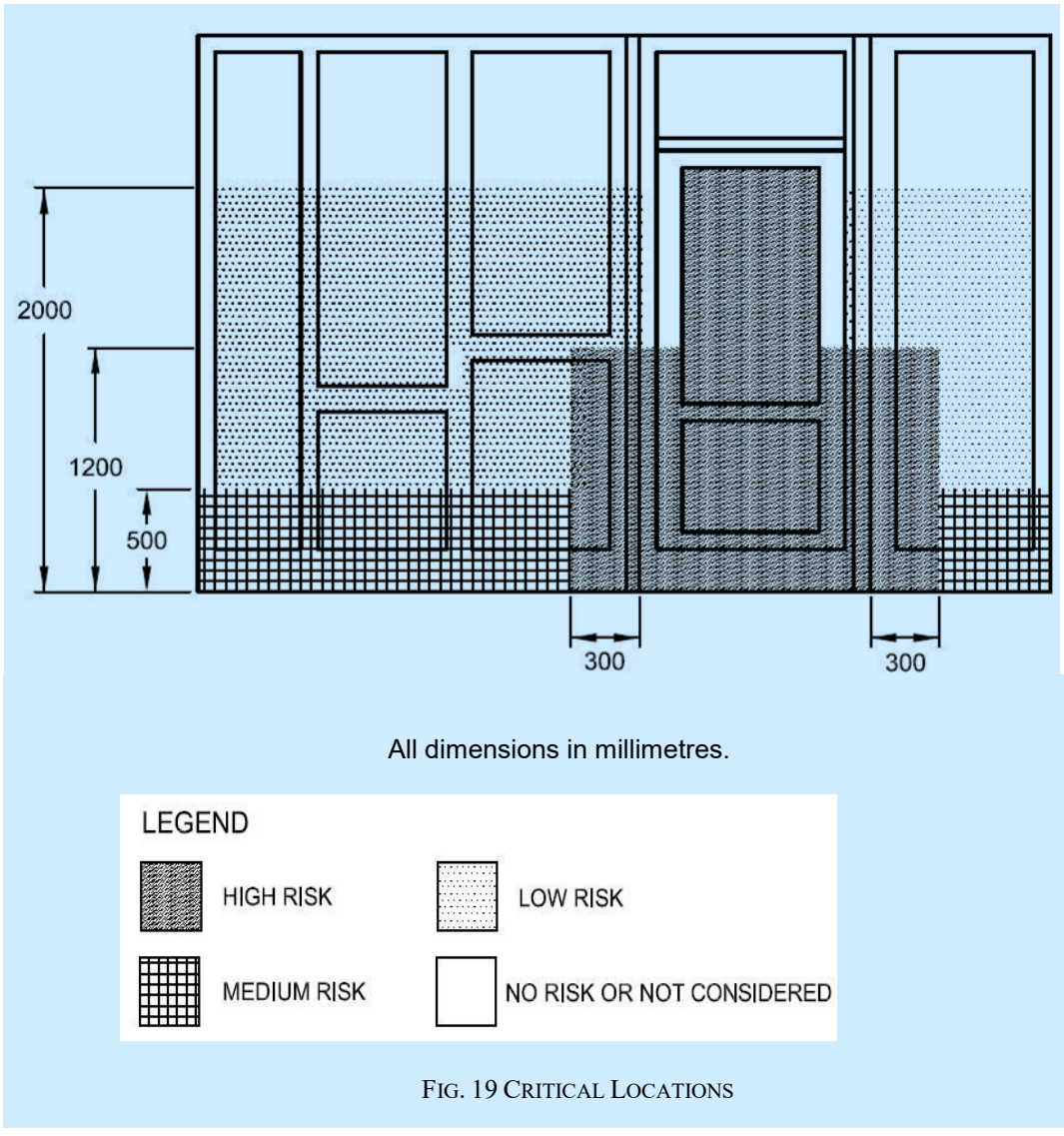

FIG. 19 CRITICAL LOCATIONS

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Table 27 Type of Glass Suggested for Use at Different Critical Locations/Cases in Buildings

(*Clause* 7.1.3)

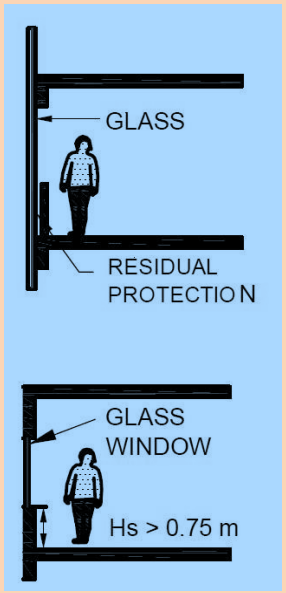
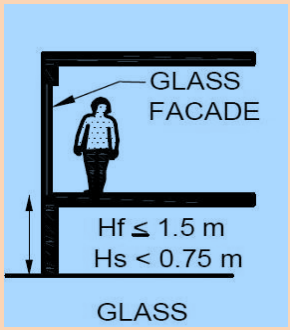
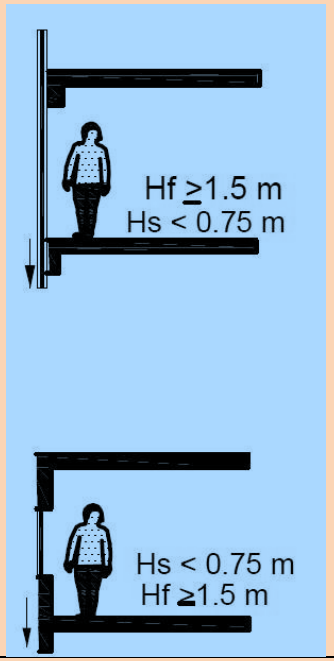
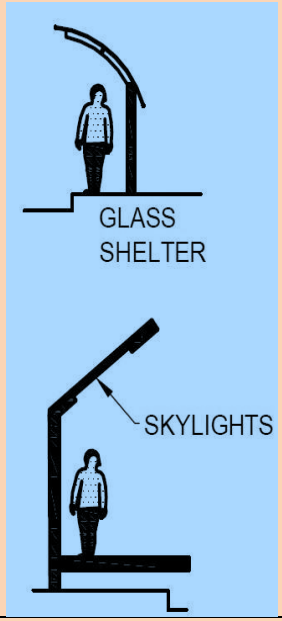
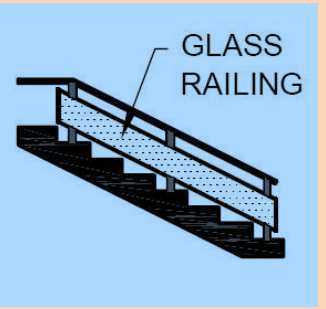
CASE 1		CASE 2	CASE 3	CASE 4	CASE 5
Vertical Walls with Residual Protection or $H_s \geq 0.75$ m (not likely to be Subjected to human impact)		**Vertical Walls $H_s < 0.75$m and $H_f \leq 1.5$ m (human impact but no risk of fall)**	**Vertical Walls $H_s < 0.75$m and $H_f > 1.5$m (human impact and risk of fall)**	**Horizontal or Sloped Glazing Glass (risk of fall)**	**Glass Acting as a Balustrade/Railing (human impact and risk of fall)**
(1)		(2)	(3)	(4)	(5)
					
Type of glass	Any glass[1)]	Safety glass[2)]	Safety glass[2)]	Laminated safety glass	Laminated safety glass

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Table 27 (*Continued*)

Examples	Residual protection is the safeguard provided to avoid the impact of human being on glass. It is provided on the side of the glass where there are chances of human impact. It can be achieved by providing protection in the form of a sill structure or transom, chair rail or grill work inside.	a) Doors b) Side panels c) Curtain walls d) Glazed area e) Doors in bathroom 1) Fully framed 2) Partially framed 3) Frameless f) Facade g) Windows h) Internal partitions and doors j) External facade and doors on ground floor, above floor with terrace outside.	a) Curtain walls b) Facade c) Spandrels d) High activity area e) High risk area f) To avoid risk	a) Roof (Skylights) b) Ceilings c) Bus shelters d) Floors e) Stairs f) Sloped facade	a) Balustrades b) Balcony c) Railings

[1] Safety glass is not mandatory.
[2] Laminated safety glass is preferred.

NOTES

1 'H_f' corresponds to height of fall of human being or glass in case of change in level and 'Hs' corresponds to the sill height with reference to floor height.

2 In case of mirror glazing, it should conform to the requirements of other safety glasses unless it is fully backed by a solid material.

3 In Case 2 and Case 3, if the smaller dimension of the pane is 250 mm or less and its area is 0.5 m^2 or less, glass other than safety glass may be used, provided that its nominal thickness is not less than 6 mm (applicable to vertical glazing).

4 Toughened or laminated safety glass should meet respective test requirements as given in respective Indian Standards.

5 The effective toughened safety glass thickness and/or laminated safety glass configuration shall be determined case by case with regard to,

a) other solicitations (wind load, snow load, dead load, and human load);
b) the overall dimension (length, width of surface);
c) the aspect ratio of the glass; and
d) the glazing fixing type (framing, bolted system, structural system, etc).

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Table 27 (*Concluded*)

6 Precautions against chances of injuries due to broken glass falling on people include:

a) Broken annealed glass falling on people can cause grievous or even fatal injuries; hence it is recommended to use safety glass in locations other than defined in Case 1 where the risk of people getting hurt by falling glass is high.

b) Toughened safety (tempered) glass has a safe breakage pattern, as it breaks and disintegrates into small and relatively harmless particles. However, thick toughened glass particles may stay interlocked, and fall as lumps of these multiple particles and can cause a minor or medium injury mainly due to the weight of the cluster.

c) Laminated safety glass shall generally not fall out of fixing. However, where laminated glass with both glasses toughened, used for horizontal or sloped glazing, in case of failure of both toughened glasses, it may crumble as a blanket and fall out of fixing. This factor needs to be considered while designing horizontal and sloped glazing. Further, when the slope is acute, inner pane facing the floor should be laminated; and when the slope is obtuse, the outer pane facing the ground/floor should be laminated and all obtuse angle sloped glazing shall be continuously capped for safety reasons.

d) Any broken glass in any glazing should be removed immediately on breakage.

e) Strength of the glazing system should be such that it has the ability to hold glass in place and prevent it from falling out as a whole.

f) For over-head applications like skylights and canopies, for the safety of maintenance workers and people underneath, the broken laminated glass shall be capable of sustaining a load of 100 kg for a minimum duration 30 min.

7 In case of external laminated glass facades, openable portions have to be left at regular distances for firefighting and smoke exhaust. This portion should be of toughened (safety tempered) glass and clearly indicated by suitable visible marking. *See also* Part F 'Fire and Life Safety' of this NBCS.

8 If insulating glass unit (IGU) is used in situations mentioned in this Section, then one of the following shall apply:

a) If IGU is installed in areas subjected to human impact on either side, then both the panes of the unit shall meet the requirements of this Section.

b) In situations where access is restricted to one side of the unit, then only the accessible side should meet the requirements of this Section.

9 Awareness of presence of glass may be by making visible manifestation.

10 For specific provisions relating to smoke evacuation, Part F 'Fire and Life Safety' of this NBCS may be referred.

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7.1.4 *Manifestation* (*Making Glass Visible*)

7.1.4.1 Presence of glass in a door, side panel or a panel capable of being mistaken for a doorway or opening, which is not made apparent by transoms, colonial bars, door frames, large door handles, stall or other components of glazing system, should be made apparent by some form of manifestation. Decorative treatment, such as being opaque, or patterned are the other common manifestation options.

The manifestation employed should be of a sufficient size to make it immediately obvious. The manifestation should preferably be permanent, for example etching of the glazing; alternatively, if applied materials are used, they should be durable and not easily removed.

7.1.4.2 A band or marking may not be required, where any one of the following apply:

a) Width of the glass is not greater than 500 mm at any part (this applies to overall panel assembly – not individual glass pieces as in faceted glazing).

b) Glass is provided with at least one fixed glazing bar, firmly attached to the styles to locate and protect each face of the glass. At least one transom (glazing bar) shall be located with its upper edge not less than 500 mm and its bottom edge not more than 1 000 mm above the floor level. The glazing bar shall have a face of width 40 ± 3 mm; and

c) Alternatively, patterns may be used as an acceptable form of marking provided it meets the other criteria in this Section.

7.1.5 *Identification*

All safety glasses shall be procured from certified manufacturers and the product shall conform to relevant Indian Standards and shall carry all relevant information through the approved label/permanent (indelible) markings on the glass surface.

7.1.6 *Test Requirements*

7.1.6.1 Safety glass shall confirm to all the test requirements mentioned in the accepted standard [C-8(7)].

7.1.6.2 *Mechanical strength*

The required mechanical strength values (*see* Table 28) apply to quasi-static loading over a short time like wind loading relate to 5 percent probability of breakage at the lower limit and 95 percent confidence intervals.

If the glass satisfies the relevant impact test performance requirements (or fragmentation test for toughened glass), in addition to all other appropriate tests mentioned in available Indian Standards or as listed in Table 28, these materials can be classified as safety glass.

Table 28 Minimum Mechanical Strength Required for Fully Toughened Glass

(*Clause* 7.1.6.2)

Sl No.	Type of Glass	Mechanical Strength N/mm^2
(1)	(2)	(3)
i)	Float (clear/ tinted/ coated/ mirror)	120
ii)	Enamelled flat (based on enamelled surface in tension)	75
iii)	Patterned glass	90

7.2 Anti-Fallout

The following shall apply:

a) For all open out windows in buildings up to 5 storey height (< 15 m) anti-fallout secondary safety device is optional; and

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b) For all open out windows in buildings more than 5 storey height (> 15 m) anti-fallout secondary safety device is mandatory.

7.2.1 In the case of a fall out due to the sash getting completely disengaged from the hinges/stay arms/restrictors, the anti-fallout device shall be capable of withstanding the self-weight of the sash. The drop height for the test shall be 100 mm. Design and testing for the mechanism to take into consideration jerk load with a factor of safety 2.0. Both static tensile test and a drop test with the mechanism installed in a window to be carried out to validate the compliance of the hardware to the design requirements.

7.3 Window Restrictor for Children

For windows which are non-emergency exits, projected top hung window, parallel opening window, vertical sliding hung window, side hung window and pivot window (except when used as an access door), the window restrictor shall be located such that the clear opening shall not exceed 100 mm (for human life safety especially for anti-fallout for children). When a force of 250 N is applied in opening condition to any part of the sash a sphere of 100 mm shall not pass through any part of the opening. Key releasable window restrictor can be used for the purpose of cleaning/maintenance. The side hung opening shall be restricted to be used in low rise buildings {< 15 m, *see* good practice [C-8(15)]}. For buildings above ≥ 15 m height, projected top hung or parallel opening with an opening restriction of 100 mm shall be used.

NOTES

1 For fire access panels/window, the local authority regulations will be applied and same will supersede the above.

2 The above requirement of window restrictor for children is mandatory for residential apartments and optional for commercial establishment. However, it is recommended to limit the opening to 100 mm for commercial establishments as well, especially for high rise buildings.

7.4 Dynamic Wind Restraint

This is applicable for open out windows in buildings more than 5 storey height (> 15m) and where the dynamic wind load is high due to the adjacent structures or terrain. Installed hardware shall be capable of self-locking the open window upon first time closure of the window due to wind conditions; in order to avoid fatigue failure on the hinges and stays. The mechanism can be releasable either using existing handle or additional release trigger. Design shall be to serviceability conditions.

8 GLAZING SYSTEM

8.1 The provisions relating to glazing system are covered hereunder. The details and procedure for selection, design, fabrication, installation, testing and maintenance of glazing system and its materials, considering the performance factors, including structural, seismic, acoustic insulation, fire resistance and energy efficiency are covered here under.

For classification of glazing system, reference may be made to **6** of accepted standard [C-8(14)].

8.2 Design and Selection

The design considerations and parameters for the design and selection of a glazing system and the components involved, along with its associated materials used in buildings are given below.

8.2.1 *Design Considerations*

Table 29 covers the various design considerations for glazing systems.

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Table 29 Design Considerations

(*Clause* 8.2.1)

Sl No.	Design Considerations/Criteria	Reference Standards
(1)	(2)	(3)
i)	**Structural aspects and parameters:**	
	a) Design load and wind load classification	As per good practice [C-8(15)] and accepted standard [C-8(14)]
	b) Other Loads — Live load, dead load, etc	As per good practice [C-8(21)]
	c) Air infiltration/Exfiltration and its classification	As per accepted standard [C-8(14)]
	d) Deflection	As per **6** and accepted standards [C-8(14)] and [C-8(12)]
	e) Seismic performance	As per **6.1.6** and good practice [C-8(16)]
	f) Water tightness and its classification	As per accepted standard [C-8(14)]
	g) Glass thickness	As per good practice [C-8(22)]
	h) Structural sealant	As per 4.4.1
	j) Frame/profile specifications:	
	1) Aluminium	As per accepted standard [C-8(12)]
	2) uPVC	As per accepted standard [C-8(23)]
	3) Steel	As per accepted standard [C-8(24)]
	k) Hardware and fixing	As per **5.4** of accepted standard [C-8(12)]
ii)	**Energy aspects and parameters:**	
	a) Window to wall ratio	As per relevant guidelines/best practices (Handbook on 'Sustainability in Buildings')
	b) Building orientation	
	c) Window to floor ratio	
	d) Thermal performance (roofs and fenestration):	
	1) U-Value of glazing	As per relevant guidelines/best practices (Handbook on 'Sustainability in Buildings')
	2) Solar heat gain Co-efficient (SHGC)/Solar factor of glazing	
	3) Visible light transmittance (VLT)	
	4) Glass – Thermo – Optical properties: U value, solar heat gain co-efficient, visible light transmittance, internal and external (heat and light) reflectance and transmittance, shading co-efficient	As per **5**

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Table 29 (*Concluded*)

Sl No.	Design Considerations/Criteria	Reference Standards
(1)	(2)	(3)
	e) Lighting	As per Part D 'Building Services, Section 1, Lighting and Natural Ventilation' of this NBCS
iii)	**Safety**	
	a) Safety related to human impact	As per 7.1
	b) Window restrictor for children	As per 7.3
	c) Anti-fallout	As per 7.2
	d) Fire safety	As per **6.4** and Part F 'Fire and Life Safety' of this NBCS
	e) Security glazing:	
	1) Bullet resistance	As per 9
	2) Ballistics resistance	As per 9
	3) Resistance to explosion	As per 9
	4) Burglar resistance	As per 9
iv)	**Acoustics parameters**	
	a) Airborne sound insulation	As per accepted standard [C-8(25)
	b) Impact sound insulation	As per accepted standard [C-8(26)]

8.2.2 *Design Parameters*

To achieve the performance criteria, care should be taken so that components in the system are selected and designed for the following parameters, such that the necessary performance criteria is met both individually and as a system:

a) Loading;

b) Deflections and allowable stress;

c) Movements;

d) Impact resistance;

e) Acoustic and sound insulation; and

f) Energy performance.

8.2.2.1 *Loading*

Wind load, live load, dead load, load combinations as per good practice [C-8(27)], seismic load as per good practice [C-8(16)], and other consequential loading may be considered.

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8.2.2.2 *Deflections and allowable stress*

The deflection of framing members, members supporting brittle materials, metal panels, glass panels and the allowable stress shall be in accordance with **6.3.6.2**. The allowable stress shall be in accordance with good practice [C-8(27)].

8.2.2.3 *Movements*

The glazing system (and its framing elements) shall be able to accommodate the movements caused due to wind loads, seismic loads, live loads, dead load deflection, thermal expansion and contraction of members, slab deflection, etc.

8.2.2.4 *Impact resistance*

Any glazing (in the glazing system) within 1 500 mm above the floor level of a building is considered likely to be subjected to human impact and hence, shall comply with the human impact safety requirements given in **7.1**.

8.2.2.5 *Acoustics and sound insulation*

The recommendations on acoustics and sound insulation given in Part D 'Building Services, Section 4 Acoustics, Sound Insulation and Noise Control' of this NBCS shall be applicable.

8.2.2.6 *Energy performance*

The recommendations on the principal energy performance characteristics of the glazing system shall meet the provisions given in **5**.

The primary characteristics required in general are reducing thermal losses and solar gains. For additional reference to visible light transmittance (VLT), solar heat gain coefficient (SHGC) and thermal transmittance (U-factor) features, relevant guidelines/best practices (Handbook on 'Sustainability in Buildings') may be referred.

8.2.3 *Types of Structural Frame Materials*

For the framed/semi-framed systems, materials such as aluminium, stainless steel, uPVC and timber, which are commonly employed are given hereunder.

8.2.3.1 *Aluminium alloy*

Extruded aluminium alloy used for structural framing members, shall be fabricated from the most appropriate grade of alloy complying with the relevant standard(s). The thickness of the aluminium sections should be in accordance with structural and hardware fastening requirements and shall be capable enough to meet the design requirements and satisfy the performance requirements.

Such extrusions in framing members and trims shall preferably be extruded aluminium grade 6063-T5, 6063-T6, 6060-T66, similar or stronger. Where 6063-T5 is to be used in shaped structures, supporting structural calculations particularly the stress criteria be ensured. Extrusion alloy 6061 or 6105 are commonly used for higher strength. Aluminium brackets shall be extruded aluminium grade 6061-T6 or approved equivalent. All aluminium alloys used as extruded rods/bars, tubes and profiles shall be compliant with accepted standard [C-8(28)].

Sheets and plates may be obtained from the following alloys of aluminium along with their primary characteristics:

a) 1100 — A low strength alloy suitable for applications requiring a high degree of formability.

b) 1135 — Brighter finish when anodized.

c) 3003 — General purpose sheet alloy with fair mechanical properties and excellent formability, but susceptible to structural streaking when anodized (noticeable in large flat areas).

d) 5005 — Excellent workability, weldability and corrosion resistance.

e) 5052 — Slightly better mechanical properties and excellent corrosion resistance and weldability.

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Other alloys can be used provided that they meet the required physical properties and the specified performance requirements.

8.2.3.2 *Stainless steel*

Mullions, panels, fascia, column covers, windows, doors, trim, roofing, gutters, flashing, hardware and other items where minimum maintenance is anticipated, shall be of austenitic stainless steels of Type 301, 302 and 304 complying with accepted standard [C-8(29)] and employed for their mechanical properties as they are highly corrosion resistant for all normal exposures.

For interior decorative applications, where regular maintenance will be provided, Type 430 may be used as it has lesser corrosion resistance properties. For external applications like in coastal regions where maximum resistance to corrosion is required, stainless steel shall be of Type 316. Particular attention shall be paid to the direction of the rolling grain on self-finished stainless steel components where the finish is aesthetically important. All screws, nuts, bolts, washers, self-tapping screws and other fastenings, including those used for the hardware, shall be of stainless steel Type 304, 316, 430 complying with accepted standard [C-8(29)]. Type 316 based steel fasteners, etc shall be preferable when they are used in coastal regions on the exposed side.

8.2.3.3 *uPVC*

A relatively flexible material, PVC which is internally reinforced with steel or aluminium to give it the required strength and stiffness may also be used. The inherent ability to make a variety of profiles, easier maintenance, corrosion resistance and heat resistance when used may be considered when choosing uPVC sections.

8.2.3.4 *Timber*

Timber frames are relatively stiff and particularly treated timber combined with good design and workmanship give an acceptable life. They are adopted for their inherent thermal insulation property, light weight and improved sound insulation. Tenderness to absorb moisture, if not carefully treated prior to use can lead to possible rotting or even warping resulting in air and water infiltration and regular painting and maintenance are the factors to be considered while choosing timber as the framing material.

8.2.4 *Surface Finishes*

The commonly employed surface finishes are listed hereunder:

a) *Aluminium frames* — The main types of aluminium frame coatings are as follows:
   1) Anodized finish,
   2) Powder paint finish, and
   3) PVDF coating.

b) *Steel frames* — Surface finish on stainless steel frames plays a major role in its resistance to corrosion and hence it is an important design aspect to be clearly specified. The types of surface finishes listed here shall be in accordance with applicable relevant standards:
   1) Mill,
   2) Ground,
   3) Brushed,
   4) Polished,
   5) Bead blasted,
   6) Electro polished,
   7) Coloured, and
   8) Patterned.

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Electro polishing of steel frames shall be done in accordance with available standards.

c) *Timber frames* — The function of finishes on timber is two-fold, first to improve the durability of the frame and second to add to the aesthetics. Unfinished, unprotected timber weathers as a result of gradual changes to its physio-chemical structure brought about by temperature and moisture content variations. The finishes may be of the following types:

1) *Conventional opaque systems* — Water and solvent borne;

2) *Natural finishes* — Semi-transparent and opaque; and

3) Water repellent preservatives, wood primers, undercoats and finishing coats (gloss, semi-gloss, flat and low sheen).

Based on the application, the appropriate type of finish shall be chosen.

8.2.5 *Types of Anchorages*

Post-installed anchors are used primarily to join the mullions and transoms to the beam or slab through an appropriate bracket. The most common types of post-installed anchors are torque controlled expansion anchors and adhesive anchors. The selection of the appropriate anchor depends on the load requirement. For details on the types of post-installed anchors, reference may be made to good practice [C-8(30)].

There are different factors which influence the selection of anchors and to ensure proper performance of the anchor during its service life. These include:

a) *Diameter* — The following parameters shall be calculated for the selection of the anchor diameter:

1) Dead load acting on the anchor;

2) Live load incident to the anchor (wind load- tensile or shear);

3) Weight of bracket, aluminium channels and other associated components; and

4) Performance against earthquake load.

b) *Length* — The overall length of the anchor is based on the effective embedment length into the concrete and the total thickness of the glazing components which have to be fastened to the base material. Insufficient embedment length shall lead to pull-out failure of the anchor;

c) *Material* — Stainless steel and galvanized iron are the two main types of materials used in the manufacturing of anchors. The selection of anchor material shall be based on the location of the anchors, considering factors such as interior or exterior application, exposure to weather, or enclosed conditions;

d) *Coating* — The following type of coating shall be used based on the specific environmental condition met with:

Sl No. (1)	*Environmental Condition* (2)	*Coating to be Used* (3)
i)	Particular influence of moisture	Zinc plated
ii)	Occasional exposure to condensation and in coastal areas/slightly corrosive	Hot-dip galvanized
iii)	Heavy condensation/corrosion and in coastal areas:	
	a) areas with high corrosion risk	Stainless steel (SS 316)
	b) risk of exposure to corrosion is medium	Stainless steel (SS 304)

e) *Load Direction* — Based on the orientation of the anchor, they are incident to shear loading, tensile loading and combined loading;

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f) *Base material* — The material characteristic of the base material (concrete) also plays a major role in avoiding failure in the anchorage. The material shall be of suitable grade, have minimum thickness, shall be properly cured and have the necessary compressive stress. Unless otherwise mentioned by the concerned structural engineer, the concrete shall be considered as cracked for all design purposes. Reference shall be made to good practice [C-8(30)] for more details in this regard;

g) *Anchor spacing and edge distance* — Anchor spacing is the centre to centre distance between the anchors. Edge distance is the distance from the centre of the anchor perpendicular to the respective edge of the concrete member. Reference shall be made to good practice [C-8(30)] for more details in this regard;

h) *Effective embedment depth* — It is the effective depth which actually transfers the forces passing through the anchor into the concrete. Reference shall be made to good practice [C-8(30)] for more details in this regard;

j) *Environmental condition* — Environmental conditions like temperature, humidity, salinity, etc will affect the performance of the anchors and may lead to pre-mature failure. Hence, careful selection of the material and coating should be done when selecting anchors which are to be installed in location subjected to adverse environmental impact such as coastal region; and

k) All the anchors shall be designed as per the recommendations of good practice [C-8(30)].

8.2.6 *Hardware Materials*

The material, geometry and finish of all the hardware materials shall be as per the requirement and should not affect the performance of the glazing system. All materials and finishes shall be as per relevant Indian Standards and tested before installing into the system in accordance to relevant Indian Standards.

8.3 Fabrication

8.3.1 *Factory*

The manufacturer shall ensure that proper internal control and documentation including records of test results performed be maintained about the production. The above shall enable quality assurance of the materials and process involved.

NOTE — For fabrication tolerance of aluminium systems, reference shall be made to **6** of accepted standard [C-8(12)].

8.3.2 *Handling of Materials on Site*

The following lists among others, the important features to be considered:

a) All materials shall be handled by workers who have been provided appropriate training and awareness on the techniques to handle the materials and shall operate with a full set of personal protection equipment (PPE);

b) Large glass panes or panels shall be handled with vacuum cups. Care shall be taken to ensure that the vacuum cups and the glass are perfectly clean. An additional rope or belt is recommended to support the dead load as a fall arrestor;

c) All materials shall be handled with dry, clean, gloves and shall not come in contact with rough surfaces or hard objects, in order to avoid scratches, dents or breakage; and

d) Labels and marking shall not be put on the coated side of glass, and any relative movement of the coating with the next glass pane shall be avoided.

8.3.3 *Transportation and Logistics*

While transporting materials from vendor to factory, within the factory, to and from surface finish coater, finished goods to site and moving finished goods at site, care should be taken to ensure the following:

a) Finished surface is adequately protected with a protective tape which can be easily removed without leaving any residue;

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NOTE — Most protective tapes either peel off or adhere to the substrate when exposed to UV for more than three months.

b) Suitable blocking, edge protection, crating, etc are done to avoid dents and gouges;

c) Hardware is secured and not damaged in transit;

d) Material is adequately secured in the truck/container especially when the goods are sent as loose container load (LCL) or partial shipment, avoiding any damages;

e) Lifting equipment shall be adequately engineered for capacity, safety and balance (especially when units are very heavy);

f) Suitable 'A' frames or customized transport support shall be used; and

g) Recommendations from glass supplier towards the use of protective film shall be followed.

8.4 Installation

8.4.1 *Site Survey of Building*

a) Measuring the elevation width and height, ascertaining the grid pattern including position of vision and spandrel frames, the levels of concrete and masonry in x, y and z axis, checking and recording the presence of any difference in plumbness of jambs, beam and column fascia and steel structures are the key activities necessary in deciding the final fabrication measurements;

b) Levelling (setting x-axis, common sill level for all elevation and adjustment in bracket depth) and surveying (measuring x-axis distance and setting the width and height for all elevation) are involved in the process of site survey of building. The key processes are listed below:

1) *Processes in levelling*:

i) The sill bottom level of all the glazing elevations are to be checked by fixing a common benchmark and the reading should be recorded at minimum one metre distance;

ii) After taking the reading the highest and lowest point can be located and a decision on the common level of all glazing elements made;

iii) The benchmark shall be a permanent mark, using oil paint mark in form of upside triangle;

iv) The levels of all beam bottoms and up stand, if any, coming across the glazing elevations at all floors are to be measured; and

v) Any other point that can affect fabrication drawings is to be noted.

2) *Processes in surveying*:

i) The total width shall be measured using a theodolite and recorded at one plane;

ii) It shall be cross checked by measuring existing column to column (at other plan) distance; and

iii) The two end point RCC shall be measured for plumbness. If any difference is observed, the readings of first two steps shall be adjusted.

8.4.2 *Installation Tolerance*

The effect of installation tolerance is vital for any type of glazed system to perform as per the requirement and the specified performance. For structural glazing, the installation tolerances shall be strictly followed in order to assure that the sealant joint opening meets the specified minimum and maximum joint opening dimensions.

NOTE — For installation tolerance of aluminium systems, reference shall be made to **5.2** of accepted standard [C-8(14)].

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8.4.3 *Good Installation Practices*

The good practices involved in installing a glazing system have been categorized hereunder.

8.4.3.1 *Anchorage*

The anchor installation shall be undertaken by installers who have been adequately trained by the manufacturer to install the specific type of anchor. The installer should be familiar with the correct installation procedures and ensure that the correct drilling and setting tools are available for the proper installation of the anchor.

For details with respect to the right installation procedure, reference shall be made to good practice [C-8(31)].

8.4.3.2 *Sub-structures*

a) In order to avoid physical contact with the panels and frames during sealant application, which might cause adhesion loss, powder-free latex gloves shall be used during panel placement;

b) Proper alignment and setting out of the sub-frame is critical in ensuring the ease of operation of the window/door;

c) Anchor/bolt heads and joints between external wall and sub-frame should be sealed with sealant for effective water tightness;

d) Prior to fixing the main frame, the sub frame should be checked for any physical damages and any damaged sub-frame should be repaired or replaced;

e) Setting out of the sub-frame should also be verified before the installation of the main frame; and

f) The sub-frame shall be properly cleaned before assembly as dust and debris accumulated in the sub-frame could affect the alignment and fixing of the main frame and may also cause blockage to the weep holes, resulting in overflow of any incidental water into the interior.

8.4.3.3 *Sealants*

a) All components (glass and framing member such as aluminium) receiving the structural glazing silicone sealant shall be wiped thoroughly with a clean cloth dampened with a cleaner recommended by the manufacturer, and immediately followed by a dry wipe, using clean cloth;

b) All surfaces once cleaned/primed shall be handled carefully so as not to contaminate the surface;

c) The substrates shall be kept free from moisture, condensation, dust and other impurities;

d) An application temperature range of + 10 °C to + 40 °C (or as specified by the manufacturer) is recommended and shall be adhered to;

e) Substrate temperatures in excess of + 50 °C will adversely affect curing process and also the adhesion of the sealant to the substrate;

f) Minimal time shall be taken for cleaning, placement and sealing of panels since environment conditions at job site cannot be controlled;

g) Temporary fasteners may be used to keep the joints stable during the curing process of the sealants and they shall be removed only after complete cure and adhesion has been achieved by the sealant;

h) Tooling of the sealant shall be done with light pressure, within 5 to 10 min, before a skin forms on the sealant;

j) The use of wet tooling aids such as soaps and solvents shall be avoided. Dry tooling is appropriate;

k) Masking tape shall be used in ensuring neat straight-line application and avoiding smears;

m) The masking tape shall be removed immediately after tooling has taken place and before the sealant starts to cure;

n) The general recommendation for the application of structural sealant required for a 4-sided system, bonding the glass to the aluminium frame should be done in a factory atmosphere;

p) Complete quality control process with respect to surface preparation, storage, sealant application and quality check are to be implemented in the factory with full traceability. In most cases sealant

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manufacturers have quality requirements and documentation process related to warranties which need to be followed; and

q) *In-situ* application may be required for 2-sided system, retrofit, repair and small areas in a facade. Sealant manufacturers should be contacted to get specific recommendation on sealant application for *in-situ* application.

8.4.3.4 *Installation*

The framing and fixing system of the panel should, in no case, cause shear stresses in the panel, either under the impact of external forces or due to differential expansion of components. The following shall be taken care of:

a) All framing shall be checked prior to glazing to assure that the opening is square, plumb and secure so that the uniform sealant bite, face and edge clearances are maintained;

b) Inspection of all butt and mitre joints shall be carried out and if found open, they shall be sealed using sealants prior to glazing;

c) Minimum edge clearances shall be maintained between glass and sash, as mentioned by the manufacturer;

d) Setting blocks shall be used to support the glass unit which shall prevent the addition of stress applied on the structural tensile bead due to the self-weight of glass; and

e) Adhesion tests shall be performed by the contractor/agency at the beginning and during the installation process.

8.5 Testing

8.5.1 *Laboratory Testing*

The glazing system is subjected to various adverse environmental conditions and impacts during its service life. A properly designed and installed glazing system may be able to resist any major damage to its stability or performance under any condition within its effective life time. The glazing system and its component materials shall be tested for the required parameters and performance.

8.5.2 *Field/On-Site Testing*

Testing of the glazing system as a whole unit in site (the field) subject to loads and effects such as water infiltration, wind (air) infiltration, acoustic insulation, fire insulation, thermal performance, structural performance, etc may be carried out for specific cases as mutually agreed to between the parties concerned, to ensure homogeneity.

The Table 30 lists the tests to be carried out for glazing systems including curtain walls, windows, skylights, sliders, doors, etc. The frequency of these tests shall be mutually agreed upon by the parties concerned.

Table 30 Testing of Glazing Systems including Curtain Walls, Windows, Skylights, Sliders, Doors, etc

(*Clause* 8.5.2)

Sl No.	Test Parameters	Testing Method
(1)	(2)	(3)
i)	Air permeability	As per accepted standard [C-8(32)]
ii)	Water penetration — Static air pressure difference	As per accepted standard [C-8(33)]
iii)	Water penetration — Dynamic air pressure difference	As per accepted standard [C-8(34)]
iv)	Standard operating forces	As per accepted standard [C-8(35)]
v)	Window and doors resistance to repeated opening and closing	As per accepted standard [C-8(36)]
vi)	Wind resistance for curtain walls, windows, sliders and skylights	As per accepted standard [C-8(37)]

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Table 30 (*Concluded*)

Sl No.	Test Parameters	Testing Method
(1)	(2)	(3)
vii)	Structural load test (100 percent design wind load)	As per accepted standard [C-8(37)]
viii)	Structural proof load test (150 percent design wind load and seismic force, if applicable)	As per accepted standard [C-8(37)]
ix)	Seismic racking test	As per Annex F
x)	Water penetration — Field	As per accepted standard [C-8(32)]
xi)	Thermal performance of windows, doors and shading devices — Detailed calculations	As per accepted standard [C-8(38)]
xii)	Calculation of thermal transmittance — Numerical method of frames	As per accepted standard [C-8(39)]
xiii)	Mechanical tests for standard windows and door height windows	As per accepted standard [C-8(40)]
xiv)	Fire resistance of perimeter fire barrier joint system	As per accepted standard [C-8(20)]
xv)	Fire resistance tests — Door and shutter assemblies	As per accepted standard [C-8(41)]
xvi)	Fire resistance test for glass walls	As per accepted standard [C-8(42)]
xvii)	Fire resistance test for doors with glass panes, openable glass windows and sliding glass doors	As per accepted standard [C-8(43)]
xviii)	Forced entry security glazing	As per accepted standard [C-8(44)]
xix)	Bullet resistant glass	As per accepted standard [C-8(45)]
xx)	Acoustics — Sound insulation	As per accepted standard [C-8(25)] and [C-8(26)]
xxi)	Blast resistant security glass	As per **9.4.3**

8.6 Maintenance

8.6.1 *Maintenance Infrastructure*

Regular maintenance of the building facades plays an important role in the holistic building maintenance. The access systems shall provide sufficient approach which enables all parts of the facade to be reached safely for the purpose of cleaning, inspection and maintenance and also offer high work efficiency to the maintenance workers, without also hindering the aesthetics of the building. The provisions of relevant guidelines/best practices (Handbook on 'Asset and Facility Management in Buildings') may also be referred.

The means of access and supporting infrastructure should be established at the design stage so that the loads are properly envisaged for.

Facade access requirements come in many varieties and may present challenges. Powered access cradles for facade access, cleaning of the glass directly from the ground, hoists based access and window based cleaning/access from inside the building are some of the common techniques employed. Recently, access techniques using ropes have been used for facade access and are generally considered to be safe, provided the workers have been trained and are using all the correct safety equipment and precautions.

In nearly all cases, facade access and maintenance requires a worker to be exposed to working at height with its associated risks. Therefore, use of height safety equipment, anchor point attachments (like the fall restraint or fall arrest system) and those given below should be employed:

a) *Roof guardrails* — When choosing equipment for working at heights, collective protection like roof guardrails may also be considered. Roof guardrails ensure that all users are protected from falling without the need for specialised training and/or equipment. They also eliminate the need for rescue

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procedures to be put in place. Roof guardrails are available as fixed, free standing, straight upright, cranked upright or collapsible; and

b) *Fall arrest systems and fall prevention cable systems* — Fall arrest systems allow safe access to roof areas for maintenance and inspections to be carried out by workers. Users of the fall arrest systems require wearing of full body harnesses and attachment to the cable/fall arrest system via a lanyard (rope) and shuttle device which allows them to walk the full length of the system without having to detach from the system. Fall arrest systems shall be of horizontal, overhead, sloping and vertical lifeline type and shall comply to the relevant/available standards.

Depending on the project requirement, the maintenance infrastructure shall consist of the following systems:

1) *Roof trolley/Cradles* — Fixed or adjustable jib, telescopic jib and cradle with pantograph.
2) *Rail systems* — Hooked, anchored, turntables and rotary plates.
3) *Other systems* — Additional winches for glass replacement and parking positions/lifting platforms.

8.6.2 *Maintenance of Materials*

The glazing system and the associated materials require a suitable degree of maintenance, if they are to fulfil their intended working life. Early detection of defects can mitigate expensive repairs or replacement later. The degree of inspection and maintenance required will depend on the type of glazing system and its intended design life.

8.6.3 *Common Practices for Maintenance*

The common principle maintenance practices to be followed shall be as given below:

a) When using the cleaning equipment or carrying out maintenance on the exterior of the building, the risk to personnel and property, prior to the start of work shall be assessed at first;
b) Where applicable, qualified personnel to carry out routine or complex maintenance shall be employed;
c) Power supplies may need to be isolated prior to maintaining electrically operated equipment such as motors to opening vents, roller blinds to atrium roof and main entrance pass doors;
d) While accessing the external facade, fall arrest systems shall always be used even while working in the house cradle;
e) Manufacturer's instructions shall be followed when using cleaning fluids and shall not be mixed up with drinking water or the like. To deal with instances of contact with eyes, skin, etc necessary eyewash arrangement and first aid equipment shall be in place;
f) Loosened fixing or bolts shall be identified, reported and attended to immediately, and shall never be tampered with;
g) Unauthorized loosening or removing of the fixings may prove dangerous and can result in structural damage and displacement of the system;
h) When lifting equipment such as hoists and sucker machines are used, it shall be ensured that they are capable of lifting the loads required. Further, the recommended safe working loads of any equipment shall never be exceeded;
j) When handling any insulation of firebreak material, it shall be ensured that gloves, goggles and masks are worn as fibrous materials (if any) may cause irritation;
k) Prior to the commencement of work involving glues or sealants (particularly in confined spaces), manufacturer's advice/special instructions shall be adhered to before exposing the product to air. Such materials shall preferably be worked with, in a well-ventilated area;
m) In the case of coated glass, the following precautions shall particularly be adhered to:
    1) Any scratching shall be avoided as this may penetrate the surface of the coating and cannot be repaired;
    2) Any excessive mechanical treatment shall be avoided as this might remove the coating in localised areas;

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3) Contact with metal objects shall be avoided; and

4) Use of chemicals that may attack the surface and damage it irreparably shall be avoided.

9 SECURITY GLASS

9.1 This clause covers the requirements and selection criteria for security glass in buildings. The general methodology for selecting types of security glass shall be in accordance with **4**. It is essential for any security glass in addition to its required function to provide adequate human impact safety features. The level of human impact safety for glass in buildings shall be in accordance with **7.1**. Furthermore, the security glass shall meet the classification standards to which it has been tested, ensuring the safety and security of occupants and goods. This clause deals with the safety and security of people or goods in relation to risks of,

a) vandalism, riots, burglary or break in protection;

b) protection from firearm shooting;

c) protection from explosion (terrorist attack); and

d) protection from ballistic attacks.

The requirements given below pertain to glass used in areas with high-security needs. In circumstances wherein the requirements of other Indian Standards exceed the requirements of this clause, the higher requirements shall become applicable.

9.2 Glass and Related Material for Security

Glass used for security purpose are laminated glass with different combination of glass panes including annealed glass, heat strengthened glass, toughened glass, and interlayers (including polyvinyl butyral, ethylene vinyl acetate, plastic sheets, polycarbonate sheets, etc) in accordance with **4**.

9.3 Selection Criteria

The selection criteria is based on the threat levels/risks (firearm shooting, ballistic attacks, explosion, burglary, etc) and stages of protection as per the requirements of the buildings and the user.

9.4 Testing procedures

9.4.1 *Burglary Resistance of Glazing Systems*

The framing member along with its hardware and glass should comply for the burglar resistance requirements as a whole. The glass to be used should comply with the testing requirements of accepted standard [C-8(44)]. The following Table 31 lists the minimum requirements for various resistance classes of glazing.

Table 31 Minimum Requirements for Glazing

(*Clause* 9.4.1)

Sl No.	Resistance Class	Minimum Resistance Class of Glazing as per Accepted Standard [C-8(46)]
(1)	(2)	(3)
i)	RC 1 N	Any glass (test is for hardware manipulation for entry)
ii)	RC 1	P4A
iii)	RC 2 N	Any glass (test is for hardware manipulation for entry)
iv)	RC 2	P4A
v)	RC 3	P5A
vi)	RC 4	P1B
vii)	RC 5	P2B
viii)	RC 6	P3B

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If a glass unit with a higher security level is used within the specimen used for the tests, it may not be possible to assess the use of glass units with a lower grade within those products without conducting further tests. This is because higher grades of glass can increase the rigidity of the product.

The following Table 32 lists the criteria for the selection of the resistance class (offender type, offender behavior, location, risk and recommendation for use).

Table 32 Criteria for the Selection of the Resistance Class

(*Clause* 9.4.1)

Sl No.	Resistance Class	Expected Offender Type, Suspected Offender Behaviour	Recommended Place of Use of the Burglar-Resistant Component		
			A Residential properties	B Commercial properties, public properties	C Commercial properties, public properties (High risk)
(1)	(2)	(3)	(4)	(5)	(6)
i)	RC 1 N/ RC 1	Components of resistance class RC 1 N/RC 1 provide basic protection against attempts to break in with physical force such as kicking in, jumping, shoulder slamming, pushing up and tearing out (mainly vandalism). Components of resistance class RC 1 N/RC 1 only have a low level of protection against the use of lever tools	If burglar resistance is required, the use of resistance class RC 1 N/RC 1 is only recommended for building components where no direct access (not ground level access) is possible		
ii)	RC 2 N/ RC 2	The occasional offender tries in addition, to break open the component with simple tools such as screwdrivers, pliers and wedges. (RC 2 N is only recommended for building components where no direct attack on the glazing used is to be expected)	Average risk attack on the glazing used is to be expected	Average risk	Not applicable

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Table 32 (*Concluded*)

Sl No.	Resistance Class	Expected Offender Type, Suspected Offender Behaviour	Recommended Place of Use of the Burglar-Resistant Component		
			A Residential properties	B Commercial properties, public properties	C Commercial properties, public properties (High risk)
(1)	(2)	(3)	(4)	(5)	(6)
iii)	RC 3		High risk	The offender tries in addition, to break open the component with a second screwdriver and a crow bar.	Not applicable
iv)	RC 4	The experienced burglar uses in addition, tools such as a hand saw, heavy hammer, axe, chisels, and a battery-powered drill.	Not applicable	Not applicable	Low risk
v)	RC 5	The experienced burglar uses in addition, electric tools such as an electric drill, jig saw, or sabre saw and an angle	Not applicable	Not applicable	Average risk
vi)	RC 6	The experienced burglar uses in addition, heavy-duty electric tools, such as a heavy-duty drill, jig saw or sabre saw and angle grinder	Not applicable	Not applicable	High risk

NOTES

1 This table is only an approximate guide. Expert advice, for example, from the local police advice centres, is essential. The assessment of the risk should be on one's own responsibility, taking into account the location of the building (protected/unprotected), use and the value of the goods inside. In case of high risk, tested and certified burglar alarm systems should be used in addition.

2 When selecting burglar-resistant elements of resistance classes RC 4 to RC 6, it should be noted that when selecting such elements in escape and rescue routes, the use of tools by the fire brigade is made more difficult and shall therefore be taken into account.

3 External sockets, for example, in the hallway, in the garden or in the patio (open space) area should be de-energized to prevent their use by the burglar.

In addition, parties to agreement may mutually decide on testing arrangement based on specialist literature.

NOTE — Specialist literature may be EN 1630 : 2021 'Pedestrian doorsets, windows, curtain walling, grilles and shutters - Burglar resistance - Test method for the determination of resistance to manual burglary attempts'.

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9.4.2 *Bullet Resistance Glazing*

The framing member along with its hardware and glass shall comply for the bullet resistance requirements as a whole. The glass to be used should comply with the testing requirements of accepted standard [C-8(45)].

The testing of framing member shall follow the same parameters of spall, no spall as well as threat levels (*see* Table 33) described in accepted standard [C-8(45)] but with penetration points in the framing member. Glazing system classification will hold good only if both the framing member and glass complies. In case, either glass or frame achieves a lower classification, then the whole glazing system shall be classified to the lower class that has been achieved.

Table 33 Threat Levels

(*Clause* 9.4.2)

Sl No.	Threat Level	Ammunition	Bullet weight	Bullet Type	Strike Velocity	Distance of Impact	No. of Strikes	Remarks
			g		m/s	m		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	1	9 mm × 19 mm	7.4 to 8.2	FMJ/Pb	430 ± 15	5 ± 0.5	5	–
ii)	2	7.62 mm × 39 mm	7.45 to 8.05	FMJ/ MSC	710 ± 15	10 ± 0.5	5	–
iii)	3	7.62 mm × 51 mm	9.4 to 9.6	FMJ/Pb	838 ± 15	10 ± 0.5	5	plus compliance with Threat Level 2
iv)	4	5.56 mm × 45 mm	3.5 to 4.0	FMJ/(SI + Pb)	890 ± 15	10 ± 0.5	5	plus compliance with Threat Level 3
v)	5	7.62 mm × 39 mm	7.45 to 8.05	HSC	700 ± 15	10 ± 0.5	5	plus compliance with Threat Level 3
vi)	6	7.62 mm × 54 mm	10.3 to 10.5	API	830 ± 15	10 ± 0.5	1	–
vii)	Special (*see* Notes)	Any other requirement by the user. Complete details of ammunition shall be stored for future upgradation of this NBCS						

NOTES

1 For special threat levels, the bullet velocity specified for the test should be representative of that developed by the weapon and ammunition, preferably at the higher end of the range of possible velocities. The range selected should be sufficient to enable the bullet flight to stabilize.

2 FMJ/Pb = Full metal jacket – Lead core, FMJ/MSC = Full metal jacket – Mild steel core, HSC = Hard steel core, API = Armour-piercing incendiary.

9.4.3 *Blast Resistance Glazing*

The framing member, along with its hardware and glass, shall collectively comply with the blast resistance requirements specified in Annex G. Glazing system classification will hold good only if both the framing member and glass complies. In case either glass or frame achieves a lower classification, then the whole glazing system shall be classified to the lower class that has been achieved.

The test ammunitions specified in this NBCS represent the most frequently faced threats and are classified into six threat levels given in Table 34.

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Table 34 Classification, Charge (TNT/Equivalent Charge) Mass, Distance (*d*) and Height of Burst (HOB)

(*Clauses* 9.4.3, G-4.7 *and* G-4.8)

Sl No.	Threat Level	TNT/Equivalent Charge Mass	Distance of Explosive Charge (TNT/Equivalent Charge) from the Test Specimen (*d*)	Height of Burst (HOB)
		kg	m	m
(1)	(2)	(3)	(4)	(5)
i)	1	2	4.5	0
ii)	2	3	5.0	0.5 ± 0.05
iii)	3	3	3.0	0.5 ± 0.05
iv)	4	12	5.5	0.8 ± 0.05
v)	5	12	4.0	0.8 ± 0.05
vi)	6	20	4.0	0.8 ± 0.05
vii)	Special	Any other requirement by the user.		

The structural integrity of glass system test specimen is evaluated by visual inspection in the following categories of outcome:

a) *Category A* — No damage to the glass and the frame of glass system;

b) *Category B* — The entire system is bent or deformed leading to the overall jamming or dis-functioning of the system;

c) *Category C* — Formation of cracks on the glass system and no damage to the frame of the glass system;

d) *Category D* — Cracking/shattering of different glass layers and damage to frame of the glass system;

e) *Category E* — Formation of splinters from the glass system or the frame within a distance of 3 m. The size of perforations in the glass or the frame shall not be more than 10 mm; and

f) *Category F* — Total collapse of the glass or frame of the glass system or both.

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ANNEX A

(*Clause* 2.15)

TEST METHOD FOR ASSESSING CURTAIN WALLS AND STOREFRONT SYSTEMS UNDER INTER-STOREY DRIFT CONDITIONS

A-1 OBJECTIVE

This test method provides a means of evaluating the performance of window wall, curtain walls and storefront systems when subjected to specified horizontal displacements in the plane of the wall. This test is applicable to window wall, curtain wall and/or storefront mock-ups consisting of supports and components, with glass panels fixed across more than one storey (hereinafter referred to as exterior wall systems). This test method is applicable to window systems only when included as part of a full-size wall mock-up. This test method describes the apparatus and the procedure to be used for displacing a specimen horizontally. The displacement cycle time is not limited and is intended to duplicate the inter-storey displacement rates associated with dynamic building sway or seismic or wind events. This test shall be applicable for seismic Zone IV and Zone V. However, depending upon application and for tall structures, it shall be performed for other seismic zones as well.

This annexure does not support to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this test method to establish appropriate safety, health, and environmental practices and determine the applicability of regulatory limitations prior to use.

A-2 TERMINOLOGY

A-2.1 Lateral Movement — A sideway displacement of a structure or part of the structure.

A-2.2 Test Specimen — The test element is referred to as the specimen. The test specimen for a wall shall be of sufficient size to determine the required performance of all typical parts of the wall system.

A-2.3 Specimen Area – The area calculated from the overall dimensions of the test specimen.

A-2.4 Floor Height – Height of one floor of the sample to be tested.

A-2.5 Horizontal Displacement – The relative displacement of the curtain wall in the horizontal direction when subjected to horizontal movement.

A-2.6 Window Wall – It sits between floor slabs and are anchored at the sill and head.

A-3 APPARATUS

A-3.1 The description of the apparatus in this section is general. Any suitable apparatus arrangement during the laboratory accreditation process is permitted.

A-3.2 The displacement(s) may be accomplished using hydraulic or pneumatic cylinders or jacks of sufficient load and stroke capacity, or mechanical devices, such as ratchet pulleys or a chain hoist.

A-3.3 Test chamber members that are representative of the building structure shall be designed to accomplish the specified displacement(s). This may be done through a system of sleeves, fixtures, pins, or any system capable of accomplishing the displacement(s) directed by the specifier.

A-3.4 The displacement measuring system shall be accurate to within 1.5 mm (0.06 inch).

A-4 CALIBRATION

All test equipment shall be calibrated periodically, at least once every six months. Calibration should be done from an accredited laboratory providing traceability of calibration to national standards established through a chain of calibration certificates.

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A-5 TEST SPECIMEN

A-5.1 The exterior wall system test specimen shall be of sufficient size and configuration to determine the performance of all typical parts of the exterior wall system. A minimum of three vertical bays in two adjoining floors of the exterior wall system shall be tested. If the wall has inside and/or outside corner, one set of corner bays shall also be included in the testing specimen. For glazing that is anchored within the floor/storey such as windows, doors, sliders and horizontal strip facade, the test specimen can be of a single floor.

A-5.2 All parts of the exterior wall system test specimen shall be full size, using the same materials, type of glass, detailing, methods of construction and anchorage as those used on the actual building.

A-5.3 The test chamber structure shall simulate the main structural supports of the actual building. However, the test chamber and/or support structure may differ from the actual building, as required, to perform the required displacement. For multi-storey mock-ups, the test chamber shall be constructed so that the anchorage at the simulated floor structure at an intermediate level of the test specimen is moveable in the horizontal direction(s). For single storey mock-ups, the test chamber shall be constructed so that the anchorage at the top or bottom is moveable in the horizontal direction(s).

A-5.4 The test laboratory shall minimize unintended torsional or flexural movements of chamber and/or support structure that could affect test results.

A-6 PROCEDURE

A-6.1 Prior to conducting the displacement tests, the test specimen shall be, at a minimum, characterized for serviceability by conducting air leakage and water penetration tests. The air leakage and water penetration tests shall be conducted at the differential pressures specified for the project.

A-6.2 Conduct Specified Displacement Test(s)

A-6.2.1 Test chamber elements representing the primary building structure shall be displaced to produce the specified movements. Each test shall consist of minimum three full cycles. A cycle is defined as a full displacement in one direction, back to the originating point, full displacement in the opposite direction, and back to the originating point (*see* Fig. 20).

A-6.2.2 Hold the specimen at displaced position not more than 60 s at each direction.

A-6.2.3 An inspection to be conducted during and after every cycle and recorded.

A-6.2.4 At the conclusion of the test, technicians and witnesses shall visually inspect the mock-up for evidences of failure.

A-6.2.5 The test agency shall record all areas of visual distress, such as disengagement, metal distortion, sealant or glazing failure, gap between framing members, or permanent deformation.

A-6.2.5.1 If glass breakage occurs during displacement tests, the test agency personnel shall carefully examine the test specimen and document the extent of this glass breakage.

A-6.2.6 Elements representing the building structure shall be displaced in a manner and method as determined by the test agency and may be subject to review by the design professional. Movement shall be parallel to the primary plane of the wall.

A-6.2.7 Changes in displacement direction(s) shall allow for movement of equipment, stops, measuring devices, etc. The test agency shall displace the chamber elements in a manner that is not intended to induce sudden acceleration and deceleration.

A-6.2.8 Additional cycles and/or displacement direction(s) may be specified by the design professional. The chamber elements shall be returned to their nominal position after completion of each cycling test, before further tests are conducted.

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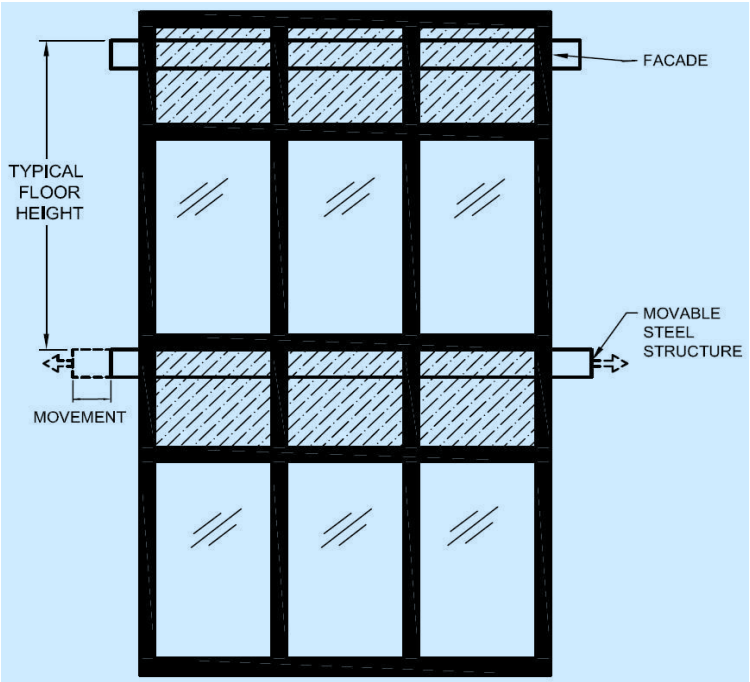

FIG. 20 ILLUSTRATION OF MOVABLE STRUCTURAL ELEMENTS IN CURTAIN WALL SYSTEMS UNDER INTER-STOREY DRIFT CONDITIONS

A-6.2.9 The elastic design displacement shall be as provided by the building engineer of record, determined by elastic analysis or other suitable method for a specific building. The displacement shall be measured at the movable floor element, not at the test specimen. For multi-storey mock-ups, the displacement between levels may vary due to different storey heights. If unspecified in contract documents, for testing, elastic design displacement in each direction shall be H/500, for curtainwall systems that span from floor-to-floor and primarily accommodate movement through racking or tipping.

A-6.3 Serviceability tests for air leakage as per accepted standard [C-8(47)] and static water penetration as per accepted standard [C-8(33)] may be conducted after the static elastic design displacement tests. The air leakage and static water penetration tests should be conducted at the pressures specified for the project.

A-6.4 The displacement shall be measured at the movable chamber member that represents the floor element, not at the test specimen or its anchorage.

A-7 CRITERIA

A-7.1 The project specifications shall state detailed performance criteria for the windows, window wall, curtain wall or storefront system. If detailed performance criteria are not included in the project specification, the criteria outlined in this test method shall be utilized.

A-7.2 Unless otherwise specified, an exterior wall system test specimen subjected to the elastic design displacement test shall be considered as passing the inter-storey drift provisions of this test method if the specimen meets the following criteria:

a) No visible damage to framing or trim components or assemblies is allowed.

b) No glass breakage or glass fallout is allowed.

c) Full disengagement of gaskets or weather seals is not allowed at any location.

d) Any gap between framing members shall be notified.

e) Air infiltration and static water penetration resistance shall remain within specified allowable limits without adjustments or repair for serviceability conditions.

NOTE — If any of air infiltration or static water penetration test fails, then seismic test has to repeated before proceeding with air infiltration or static water penetration retests.

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f) No wall components may fall off.

NOTE — If an inter-storey drift (1.5 × storey height) test is conducted, the results shall be recorded and mutually agreed upon with the specifier. In this case, compliance with the above failure criteria is not required.

A-8 REPORTING

The information to be duly reported after testing are given hereunder.

A-8.1 General

Testing agency, date and time of test, and date of report.

A-8.2 Specimen Details

A-8.2.1 *Specimen Description*

Manufacturer, model, operation type, materials, and other pertinent information; description of the locking and operating mechanisms if applicable; glass thickness, type and method of glazing; weather seal dimensions, type, weep holes, and material; and crack perimeter or specimen area, or both.

A-8.2.2 *Drawings of Specimen*

Detailed drawings of the specimen showing dimensioned section profiles, framing location, panel arrangement, installation and spacing of anchorage, weather stripping, locking arrangement, hardware, sealants, glazing details, weep holes, drainage path detail, and any other pertinent construction details. Any modifications made on the specimen to obtain the reported test values shall be noted.

A-8.3 For window and door components, a description of the locking and operating mechanism.

A-8.4 Identification of glass thickness and type and method of glazing.

A-8.5 Test Parameter

A-8.6 List of Equipment Used

List the equipment name, identification number and calibration validity of all the equipment used at the time of testing.

A-8.7 Test Observation

A visual record of the test observations.

A-8.8 Photographs or drawings of the test chamber and the wall system test specimen mock-up indicating location of measuring devices and movement devices.

H-8.9 Compliance Statement

A statement that the tests were conducted in accordance with this test method, or a complete description of any deviation from this test method. When the tests are conducted to check for conformity of the specimen to a particular performance specification, the specification shall be identified.

A-9 INFORMATION OF DESIGN LOADS

In order to design and detail exterior wall systems and their attachments to meet the inter storey drift requirements, the design load shall be as per good practice [C-8(48)].

Inter-storey drift can result from either seismic loads or by wind loads.

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ANNEX B

(*Clauses* 5.1, 5.2.11 *and* 5.3.3.6)

MINIMUM PERFORMANCE REQUIREMENT OF GLAZING FOR EXTERIORS

B-1 The panel size design of the exterior glass for the building is done in accordance with this Section considering the effect due to wind load, other imposed load and seismic design. The type of glazing to be installed is determined considering human safety in **7.1**.

In the determination of the energy and thermal performance of the glazing, the three compliance approaches prescriptive in nature, defines the minimum glazing/glazing system requirement to be met.

The prescriptive values are specified with respect to the geographic climatic zone and glazing area of the building.

B-2 BUILDING FENESTRATIONS

The fenestration may comply with the maximum glazing area weighted on *U*-value and solar factor as given in relevant guidelines/best practices (Handbook on 'Sustainability in Buildings').

The window wall ratio (WWR) is defined by the total fenestration area to the total gross area of the building envelope which is limited to 70 percent. The corresponding minimum visible light transmission is given below. For fenestration area greater than 70 percent, a reference may be made to other specialist literature.

Sl No.	*WWR*	*Minimum Visible Light Transmission* Percent
(1)	(2)	(3)
i)	0 to 0.3	27
ii)	0.31 to 0.4	20
iii)	0.41 to 0.5	16
iv)	0.51 to 0.6	13
v)	0.61 to 0.7	11

B-3 WEATHER PERFORMANCE

Air infiltration from the fix litres shall be permitted within 1.5 $m^3/h.m^2$ and from operable sash 2 $m^3/h.m$.

Water should be contained in the gutter and drained back to the exterior. Any uncontrolled water in excess of 15 ml or more on the top surface of any exposed interior shall be considered as leakage.

B-4 In glass facade buildings, it is recommended to have provision for smoke/heat exhaust ventilation that automatically or by manual trigger expels smoke, heat and gases during a fire or emergency. These vents should be integrated into the building's smoke control system to improve evacuation safety, firefighting efforts, and overall air quality. These vents should be part of building facade and may also be located at suitable locations for effective exhaust. For further details in this regard, Part F 'Fire and Life Safety' of this NBCS may be referred.

In glass facade buildings, where airtight construction can restrict natural airflow, it is recommended to have make-up air vent also as a part of ventilation system that automatically or by manual trigger, introduces fresh air into the building to compensate for the air exhausted by smoke vents, HVAC systems, or other ventilation processes. These vents help maintain air balance and indoor air quality which ideally should be 85 percent to 90 percent of vented volume of smoke/heat.

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ANNEX C

[*Clause* 5.3.2(b)]

TEST REQUIREMENTS FOR COATED GLASS

C-1 The coated glass that is used for the exterior of commercial and non-commercial (residential building) building should meet the test requirement listed in this Annex.

C-2 MEASUREMENT OF PERFORMANCE

The performance factors and spectral details of the coated glass shall be determined using standard accepted spectrophotometer equipment. The actual size shall be dependent on the type of equipment being used for the measurements. The transmittance of the measurement sample shall be measured with radiation of normal incidence at the following wavelengths:

a) 550 nm (representative wavelength for light and solar transmittance); and

b) 900 nm (representative wavelength for solar transmittance).

For glass claiming to have a low emissivity coating, a measurement of the reflectance shall be made at 8 μm using radiation of nearly normal incidence.

C-3 CONDENSATION RESISTANCE TEST

C-3.1 This test consists of subjecting the coated glass to a water saturated atmosphere at constant temperature. The samples have condensation continually forming on them and it is this condensation that may cause surface degradation.

C-3.2 Test apparatus should have a provision of four test pieces and the materials used for the inner walls shall be corrosion-resistant and shall not affect the test pieces.

C-3.3 Procedure

a) The water tank shall be filled with demineralized water, having conductivity lower than 30 μS and a *p*H higher than 5. The internal temperature of the cabinet shall be controlled by means of the reference thermocouple keeping a temperature of the reference glass piece of 40 ± 1.5 °C;

b) The test cabinet shall be in a room with an ambient temperature of 23 ± 3 °C. Care shall be taken to ensure that draughts, dust, moisture and solar radiation do not interfere with the test cabinet; and

c) The reference temperature shall be reached within 2 h of commencing heating. Condensation shall be seen to form on the glass pieces. The test is continued without interruption for the required time. Both the internal reference and the external air temperature shall be regularly checked.

C-3.4 Requirements

a) No defect, greater than 3 mm length;

b) Maximum one defect between 2 mm and 3 mm length;

c) Maximum five defects between 1 mm and 2 mm length;

d) No scratches, staining of the coating or clusters of pinholes greater than 1 mm;

e) When compared with the reference test piece, in both reflection and transmission, there shall be no significant colour change. This observation shall be made within 20 s; and

f) The transmittance measured at 550 nm and 900 nm shall differ by no more than ± 0.03 from the corresponding measured value on the reference test piece.

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C-4 ACID RESISTANCE TEST

C-4.1 This test consists of subjecting the coated glass to a sulphur dioxide saturated atmosphere at constant temperature. The samples shall have condensation continually forming on them. It is this condensation together with the quantity of sulphur dioxide that may cause surface degradation. Test apparatus should have a provision of four test pieces and the material used for the inner walls shall be corrosion-resistant and shall not affect the test pieces.

C-4.2 Procedure

a) The test cabinet contains 2 litre of demineralized water having conductivity lower than 30 µS. When the cabinet is closed 0.2 litre of SO_2 shall be added and the heating system switched on;

b) The test consists of a repetition of 24 h cycles (*see* Fig. 21);

c) Each cycle consists of high temperature plus condensation period and an ambient temperature without condensation period; and

d) The temperature shall increase to 40 °C ± 1.5 °C in less than 1.5 h. During the next 6.5 h the test pieces shall be subjected to condensation in the SO_2 atmosphere.

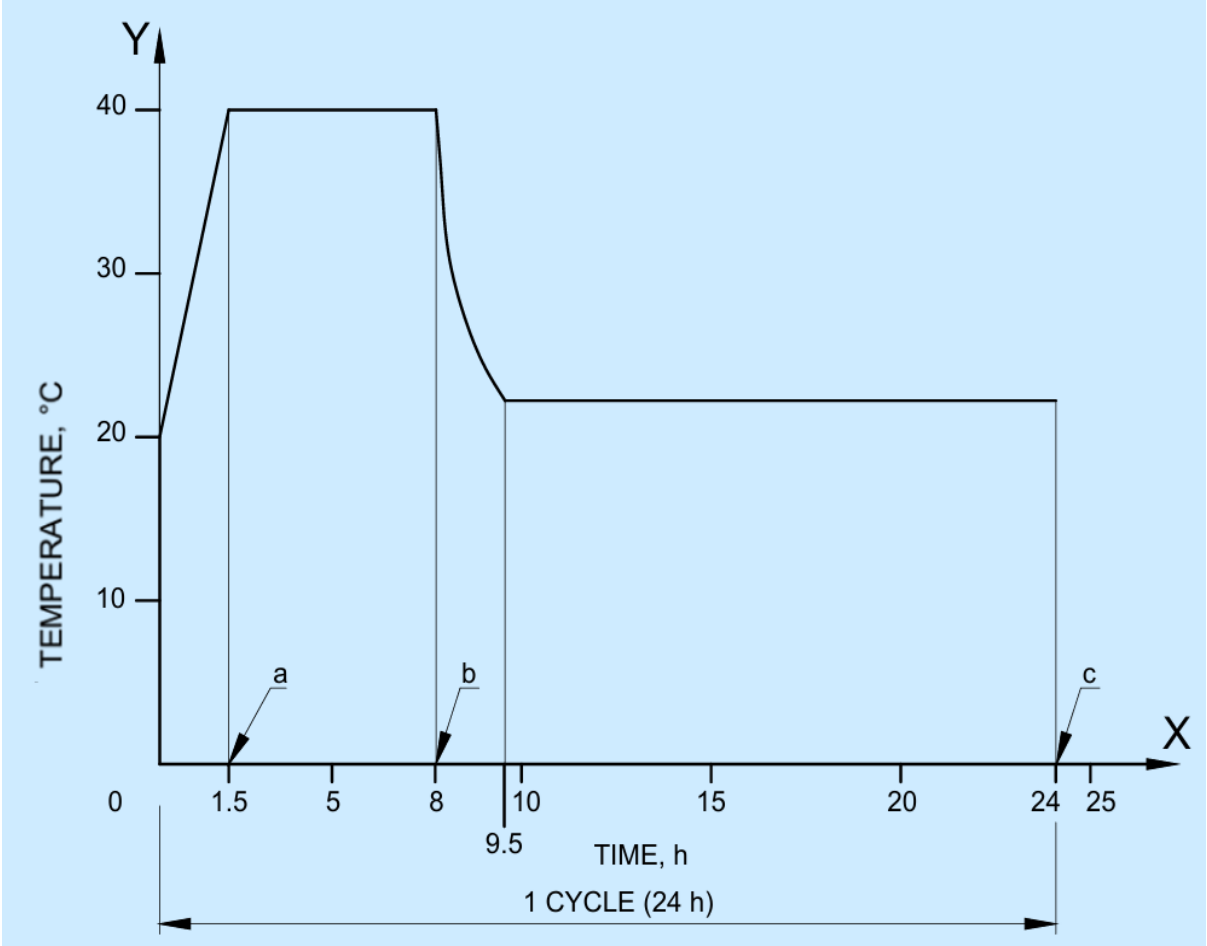

FIG. 21 24-HOUR CYCLE OF HEATING

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C-4.3 Requirement

The transmittance measured at 550 nm and 900 nm shall differ by no more than ± 0.03 nm from the corresponding measured value on the reference test piece.

C-5 NEUTRAL SALT SPRAY

C-5.1 This test consists of subjecting the coated glass to neutral, water saline atmosphere at constant temperature. It is the water saline spray that may cause surface degradation.

Test apparatus should have a provision of four test pieces and the material used for the inner walls shall be corrosion-resistant and shall not affect the test pieces.

C-5.2 Procedure

a) The test cabinet shall be prepared and run for a minimum period of 24 h before the test pieces are placed within it. The neutral salt solution is made up by dissolving NaCl in demineralized water having conductivity lower than 30 µS, to produce a concentration of 50 ± 5 g/l at 25 ± 2 °C; and

b) The compressed air supplied to the spray nozzle shall be passed through a filter to remove all traces of oil or solid matter and shall be at an absolute pressure of 70 kPa to 170 kPa through a saturator at 40 ± 1.5 °C. The spray nozzle shall be made of inert material, with baffles to prevent direct impact of spray on the test pieces.

C-5.3 Requirements

a) No defect, greater than 3 mm length;

b) Maximum one defect between 2 mm and 3 mm length;

c) Maximum five defects between 1 mm and 2 mm length;

d) No scratches, staining of the coating or clusters of pinholes greater than 1 mm;

e) When compared with the reference test piece, in both reflection and transmission, there shall be no significant colour change. This observation shall be made within 20 s; and

f) The transmittance measured at 550 nm and 900 nm shall differ by no more than ± 0.03 nm from the corresponding measured value on the reference test piece.

C-6 ABRASION RESISTANCE TEST

C-6.1 This test consists of subjecting the coated surface of the glass to rubbing with a felt pad in dry condition. It is the type of pad, the loading on it and the number of strokes that may cause surface degradation.

C-6.2 Procedure

a) The metal finger shall be approximately 15 mm to 20 mm in diameter and shall be driven so as to produce a frequency of 60 strokes/min + 6 strokes/min, alternating forwards and backwards. The stroke length shall be 120 mm ± 5 mm. The strokes shall be parallel and ensure a constant pressure over the zone to be tested;

b) A circle shape with a diameter of 14.5 mm ± 0.5 mm;

c) The test sequence shall commence within 30 min of the sample being cleaned; and

d) The metal finger containing the felt pad shall be lowered on to the glass surface and a load of 4 N applied perpendicular to the glass surface via the felt pad.

C-6.3 Requirements

a) Ensure that the abraded area is uniform; and

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b) The transmittance measured at 550 nm and 900 nm shall differ by no more than ± 0.03 nm from the corresponding measured value on the reference test piece.

C-7 DURATION OF TESTS

The duration of tests shall be as per the following:

Sl No.	*Test*	*Test Duration for Class*		
		A	B	S
(1)	(2)	(3)	(4)	(5)
i)	Condensation resistance	21 days	4 days	14 days
ii)	Acid resistance	5 cycles	1 cycle	5 cycle
iii)	Neutral salt spray	21 days	10 days	–
iv)	Abrasion resistance	500 stokes	50 stokes	500 stokes

NOTES

1 *Class A* − The coated surface of the glass can be positioned on the outer or the inner face of the building.

2 *Class B* − The coated glass can be used as monolithic glazing but the coated surface shall be on the inner face of the building.

3 *Class S* − The coated surface of the glass can be positioned on the outer or the inner face of the building but these types of coated glasses can only be used in specifically defined applications.

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ANNEX D

[*Clauses* 6.1.9 *and* E-1.1.1(4)]

PROCEDURE FOR CALCULATING THE APPROXIMATE DEFLECTION AT CENTRE OF GLASS PANEL

D-1 Maximum glass deflection as a function of plate geometry and load may be calculated using the following:

$$\text{Deflection, } w = t \,.\, \exp\,(r_0 + r_1 X + r_2 X^2)$$

where

$X = \ln\{\ln[q(a.b)^2/Et^4]\}$

$r_0 = 0.553 - 3.83(a/b) + 1.11(a/b)^2 - 0.0969(a/b)^3$

$r_1 = -2.29 + 5.83(a/b) - 2.17\,(a/b)^2 + 0.206\,7(a/b)^3$

$r_2 = 1.485 - 1.908(a/b) + 0.815(a/b)^2 - 0.082\,2(a/b)^3$

where

E = Young's modulus of glass, N/mm^2;

q = net pressure on the pane, N/mm^2;

a = longer dimension, mm;

b = shorter dimension, mm; and

t = thickness of the glass pane calculated.

Example:

Lateral Deflection Calculation — Determine the maximum lateral deflection (w) of a vertical 1 200 mm × 1 500 mm × 6 mm rectangular glass plate subjected to a uniform lateral load of 1.80 kPa. The actual thickness of the glass is 5.60 mm as determined through direct measurement.

a = 1 500;

b = 1 200;

$r_0 = -2.689$;

$r_1 = 2.011$;

$r_2 = 0.213$;

$q = 1.80 \times 10^{-3}$ N/mm^2;

$E = 71.73 \times 10^3$ N/mm^2;

t = 5.60; and

x = 1.490.

Therefore, maximum deflection at the centre of glass is:

$w = 5.6 \text{ x } \exp\,(-2.689 + 2.111 \times 1.490 + 0.213 \times 1.490^2)$

w = 12.2 mm

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ANNEX E

(*Clause* 6.1.9)

MODEL CALCULATION OF GLASS THICKNESS

E-1 MODEL CALCULATION

E-1.1 Glass panel dimension width 1 200 mm and height 1 800 mm for location Chennai for annealed glass type.

E-1.1.1 *General Checks have to be done for Safety Performance*

a) The maximum area of glass panel is restricted to 15 m^2;

b) The maximum span of window is restricted to 4 m;

c) Applicable to normal, reflective, laminated, tempered and insulating glass; and

d) Applicable to rectangular panels properly fixed.

The procedure for calculating glass thickness is described below:

1) *Step* 1 — Calculate aspect ratio (AR_{max}) from Table 15.

Aspect ratio (AR_{max}) = Longer side/shorter side of glass

Aspect ratio = 1 800/1 200 = 1.5

2) *Step* 2 — Wind load calculation

For wind load calculation, reference shall me made to good practice [C-8(15)].

$$p_z = 0.6\ V_z^2$$

$$V_z = V_b.k_1.k_2.k_3.k_4$$

where

V_b = basic wind speed based on location {*see* Fig. 1 of good practice [C-8(15)]}.

k_1 = probability factor (risk co-efficient) {*see* **6.3.1** of good practice [C-8(15)]}.

k_2 = terrain roughness and height factor {*see* **6.3.2** of good practice [C-8(15)]}.

k_3 = topography factor {see **6.3.3** of good practice [C-8(15)]}.

K_4 = importance factor for the cyclonic region {*see* **6.3.4** of good practice [C-8(15)]}.

V_z = design wind speed at height z, in m/s.

p_z = wind pressure at height z, in N/m^2.

The design wind pressure (p_d) can be obtained as:

$$p_d = K_d.\ K_a.\ K_c.\ p_z$$

where

K_d = wind directionality factor {*see* **7.2.1** of good practice [C-8(15)]}.

K_a = area averaging factor {*see* **7.2.2** of good practice [C-8(15)]}.

K_c = Combination factor {*see* **7.3.3.13** of good practice [C-8(15)]}.

The value of p_d, however shall not be taken less than 0.70 p_z.

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Design wind speed for Chennai location {Basic wind speed = 50 m/s taken from good practice [C-8(15)]} = 1 750 N/m^2.

3) *Step 3* — Strength factor

Based on the types of glass (laminated/tempered/insulating glass) pressure factor is calculated. To determine the thickness of laminated/ tempered/insulating glass, the design wind pressure P_{net} is modified as below:

$$P_{net} = p_d / P_f$$

where

P_f = pressure factor dependent on the type of glass.

The values of the pressure factor P_f can be taken from Table 14.

For annealed units, $P_f = 1.0$, therefore,

$$P_{net} = 1\ 750/1.0 = 1\ 750\ \text{N/m}^2$$

At this stage the glass thickness value shall be calculated from the table and then the deflection and stress check be done as described below:

4) *Deflection Check*

The deflection of the glass can be calculated using the formula in Annex D. Actual deflection for the pane shall be obtained as below:

Deflection, $w = t.\ \exp(r_0 + r_1 X + r_2 X^2)$

$$X = \ln\ \{\ln[q(a*b)^2/Et^4]\}$$

$$r_0 = 0.553 - 3.83(a/b) + 1.11(a/b)^2 - 0.0969(a/b)^3$$

$$r_1 = -2.29 + 5.83(a/b) - 2.17(a/b)^2 + 0.206\ 7(a/b)^3$$

$$r_2 = 1.485 - 1.908(a/b) + 0.815(a/b)^2 - 0.082\ 2(a/b)^3$$

where

E = Young's modulus (71.7 x 10^3 N/mm^2);

Q = net pressure on the pane (N/mm^2);

A = longer dimension (mm);

B = shorter dimension (mm); and

T = thickness of the glass pane calculated.

The actual deflection is, 4 mm < 19 mm maximum allowable.

As per **6.3.6.2**, deflection at the centre of the glass for monolithic glass shall be the lesser of shortest span/60 or 19 mm.

Result: As 1200/60 = 20 mm, hence, 19 mm is the maximum allowable deflection.

Hence the glass thickness calculated is safe against deflection.

Glass Thickness Calculation:

$P_{net}.A = 200.0\ T^k$ (let $T \leq 6$ mm)

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P_{net}= 1750

A = 2.16 m^2

K = 1.765

T = 5.28 mm, therefore choose the thickness of glass as 6 mm.

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ANNEX F

(*Table* 30)

PERFORMANCE TESTS FOR GLAZING SYSTEM

F-1 The following enlists the performance tests for glazing systems. The general test requirements shall be as per **7.1** of accepted standard [C-8(14)].

F-2 SEISMIC RACKING TEST

Criteria shall be the movement of the sub-structure in the horizontal (+/-) directions. Optional testing for vertical movements (+/-) shall be based on sub-structure movement to accommodate live load, long term creep, column shortening and thermal elongation or contraction. No glass breakage or fall out is allowed. No wall component fallout is allowed. The system shall comply with the requirements of air permeability and static water penetration tests as per the designed performance criteria. Post proof load testing, air permeability testing and static water penetration testing shall not be required unless specified otherwise.

F-3 BUILDING MAINTENANCE UNIT (BMU) PULL OUT TEST (WHEN APPLICABLE)

A tensile load of minimum 1.5 kN shall be applied on the BMU restraint in the following directions:

a) Outward and held for a period of 1 min;

b) Horizontally to the right and held for a period of 1 min;

c) Horizontally to the left and held for a period of 1 min;

d) Vertically upward and held for a period of 1 min; and

e) Vertically downward and held for a period of 1 min.

After each step the BMU restrain should not fail or have permanent deformation.

NOTE — If the load on the BMU restraints required by the BMU supplier is higher, the higher load shall apply.

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ANNEX G

(*Clause* 9.4.3)

TESTING REQUIREMENTS FOR BLAST RESISTANT GLAZING SYSTEMS

G-1 OBJECTIVE

G-1.1 The purpose of this clause is to provide general guidelines about equipment, range conditions and procedures used to conduct the blast evaluation of glazing systems against the designed blast load and to verify that it meets the defined blast pressure load.

G-1.2 The glazing system is evaluated for its structural integrity by visual inspection only.

G-1.3 The safety precautions and regulations undertaken during blast resistant tests of glazing systems shall be as specified by the test agency.

G-1.4 Blast resistant test of glazing systems shall be carried out only by the accredited institutions/laboratories authorized by the Government of India.

G-2 TERMINOLOGY

G-2.1 Test Sponsor — Party requesting and sponsoring the test. The test sponsor may be its manufacturer or user (or his representative).

G-2.2 Test Agency — Party performing the testing and documenting the test results.

G-2.3 Test Specimen — The test specimen is provided as a complete assembly by test sponsor to the test agency. It may include glazing systems like glass door, glass windows/frames, glass panels, etc.

G-2.4 Trial Officer — Individual identified by the test agency responsible to complete the specified tests as required and to document the results in accordance with this test method. The Test Director shall sign all of the test reports.

G-2.5 Trial Safety Officer — Individual identified by the test agency nominated by the means of trial plan approved by the competent authority. The Trial Safety Officer is responsible for all the safety aspects during the trial time and safe conduct of trials.

G-3 APPARATUS

G-3.1 High Explosive

The high explosive charge (prepared by using TNT/equivalent charge) as mentioned by the test sponsor is used to generate the desired blast parameters for the performance evaluation of test specimen.

G-3.2 Instrumentation

The instrumentation system used for measurement of blast pressure comprises of piezoelectric based pressure sensors, signal conditioner, low noise anti-microphonic cables and data acquisition systems. The details are as follows:

G-3.2.1 *Pressure Sensor*

Pressure sensor is used to record pressure-time history of blast wave. The sensor contains piezoelectric crystal as a sensing element which produces an electric charge proportional to impinging pressure. The pressure sensor has an in-built circuitry that converts pressure to voltage signal. It requires installation control panel (ICP) constant current power supply which is provided by compatible signal conditioner.

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G-3.2.2 *Low Noise Cables*

A special low noise anti-microphonic cable is used to transmit the signal from sensor to recording system. These cables are immune to tribo-electric effect and electromagnetic interference.

G-3.2.3 *Data Acquisition System* (*DAS*)

The DAS consists of an analog or digital recording system with a sufficient number of data channels to accommodate the pressure sensor(s) and any other electronic measuring devices. The DAS shall operate at a sufficiently high frequency to record reliably positive pressure and positive phase duration. The triggering of recording system is synchronized with explosion by using transistor-transistor logic system.

G-3.3 Photographic Equipment

Each test is documented with still photography. Video or high speed photography of the testing is deployed, if required.

G-4 PROCEDURE

G-4.1 The Trial Officer along with all team members will be responsible for the execution of the test. The Trial Safety Officer along with the range safety team will take care of the safety of personnel and property during the conduct of test.

G-4.2 The trial site location and range safety distance shall be decided by the test agency.

G-4.3 The parameters (identification number, shape, weight, dimensions, etc) are verified before trial. The documentary proof for the same shall be submitted by the test sponsor to the test agency.

G-4.4 The test specimen is held on to the wall or as per actual scenario like wall having duct with or without room with the help of supporting frame such that no additional pressure gets generated on the glazing system when it is fixed to the frame using nuts and bolts.

G-4.5 The attack face of the test specimen will be towards the explosive charge (TNT/equivalent charge).

G-4.6 Photographic records of details of the glazing assembly and the test configuration are taken prior to the test. If a video photographic record of the response of glazing assembly during the test is desired, video cameras should be deployed.

G-4.7 The explosive charge (TNT/equivalent charge) is prepared as per the threat levels based on weight of explosive and standoff (*see* Table 34) or as defined by the test sponsor.

G-4.8 The explosive charge (TNT/equivalent charge) is positioned at a certain height of burst (HOB) and at distance *d* from the test specimen [as shown in Fig. 22 (isometric view) and Fig. 23 (top view)] (*see* Table 34).

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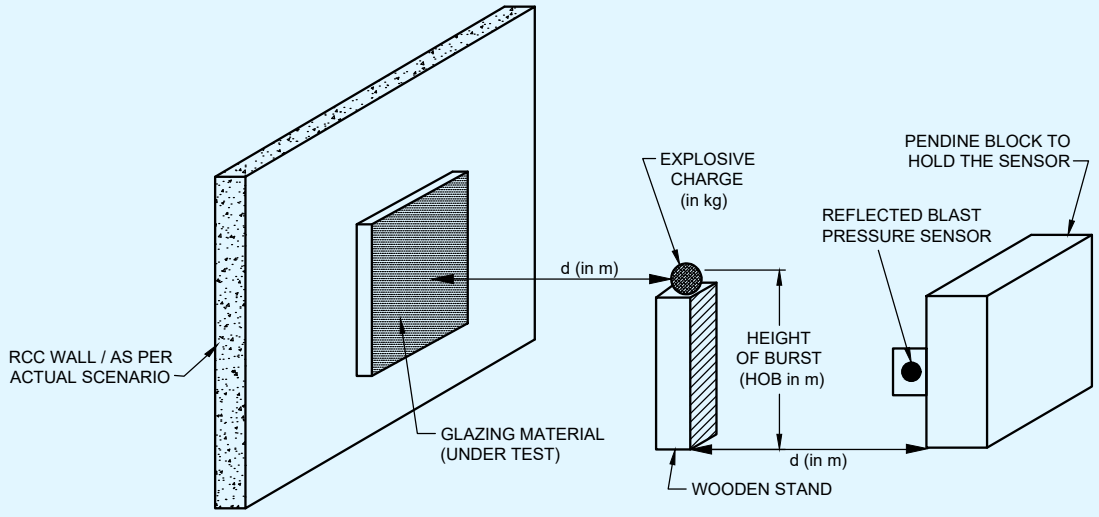

NOTE — The height of burst (HOB) shall be referenced from the hard stand on which the test specimen support (like RCC wall/test specimen support) stands.

FIG. 22 ISOMETRIC VIEW — SCHEMATIC OF TEST SETUP

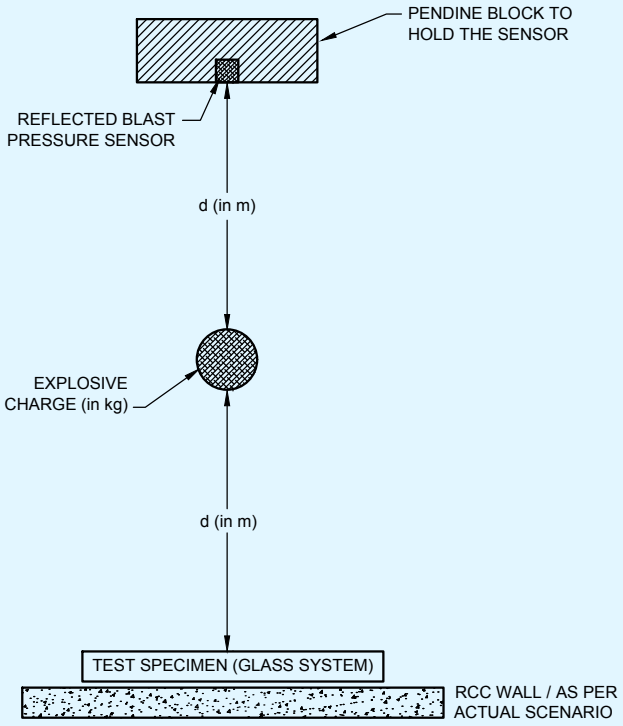

FIG. 23 TOP VIEW — SCHEMATIC OF TEST SETUP

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G-4.9 The pressure sensor to measure the reflected blast pressure is deployed at distance *d* (as shown in Fig. 22 and Fig. 23). The pressure sensor is deployed in such a way that its sensing element is directed towards the explosive charge (TNT/equivalent charge).

G-4.10 The distance *d* and height of burst (HOB) varies depending on the threat level.

G-4.11 The pressure sensor is then connected to the data acquisition system through signal conditioner via low noise cables and tested to verify proper functioning.

G-4.12 The explosive charge (prepared by using TNT/equivalent) is then detonated by following all safety precautions ensured by trial safety officer and range safety team.

G-4.13 After the test, the visual inspection of the glazing assembly is done to evaluate its structural integrity. Adequate photography is taken after each test.

G-4.14 In case of misfire, the second attempt is done as per misfire drill of the test agency.

G-5 OUTCOMES

The structural integrity of glass system test specimen after the trial is evaluated by visual inspection according to the outcome categories specified in **9.4.3**.

G-6 REPORT GENERATION

G-6.1 Upon completion of the testing, the Test Director assigns a test number and record the test measurement and observations obtained from all test specimen(s) for preparation of test report.

G-6.2 The Trial Officer and the Test Director signs the test report and gets it approved from competent authority of the test agency.

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ANNEX H

(*Foreword*)

INSTALLATION OF MIRRORS

H-1 This annex provides general guidelines, which is to be followed during the installation of mirror. Mirrors (Piece of glass silvered on one side, with a protective paint coating) are used in shops, gymnasiums, etc, where it is predominantly used as wall cladding. The mirror panels in such application use large panel size and it is recommended to follow the installation procedure in accordance with this Section.

H-1.1 The mirror should always be mounted on a perfectly flat, clean and dry surface free from acids and aggressive substances such as acid, solvent and cement mortar.

H-1.2 The mirror shall be mounted on wall or plywood and care is taken that the mirror is never stuck over unstable support (painted paper, paint or old plaster). It is recommended that the mirror installation is not done directly on the mirror supports. In order to have good ventilation, the following spacing behind the mirror is recommended depending on the height of the mirror panel:

a) A space of 5 mm for a mirror less than 1 m height; and

b) A space of 10 mm for a mirror greater than 1 m height.

Figure 24 gives typical details of the installation of mirrors. During the installation of mirror panels at several levels that is, one above the other, it is recommended to provide a space of 10 mm at the top and at the bottom for circulation of air. The side of the mirror is designed with a minimum 1 mm or 2 mm of interleaving space. When mirror is fixed in a profile (frame), care is to be taken to maintain perfectly dry and clean frame. It is recommended that mirror is supported on hard non-metallic/plastic wedges of at least 3 mm, to raise the mirror and thus avoid contact with the condensed water which could accumulate in the profile. If the mirror is screwed on, the screws of a suitable dimension is designed, with the interleaving and washers (*see* Fig. 25) made of plastic to avoid any direct glass-metal contact. To minimize the risk of breakage, cracks and splinters during screw-mounting the mirror, it is recommended to use synthetic protective collars.

H-1.3 For the adhesive used to mount mirrors, care should be taken to ensure that the adhesive is compatible with the protective coatings on the mirror. It is recommended to use only neutral binding agents such as alkoxy silicone, oxime silicone, etc. Avoid use of acid silicones, such as acetoxy silicones which contain acetic acid or water/rubber/polyurethane-based glues.

H-1.4 In the high humidity areas (for example bathrooms), it is recommended to seal the edges with neutral silicone to protect the backing paint from damage due to moisture and enhanced protection from moisture. It is recommended to use double sided adhesive tapes to fix the mirror on the mounting surface (*see* Fig. 26). The adhesive tapes used for installation shall be certified to be compatible with the mirror and the adhesive or the adhesive tapes shall always be fixed vertically.

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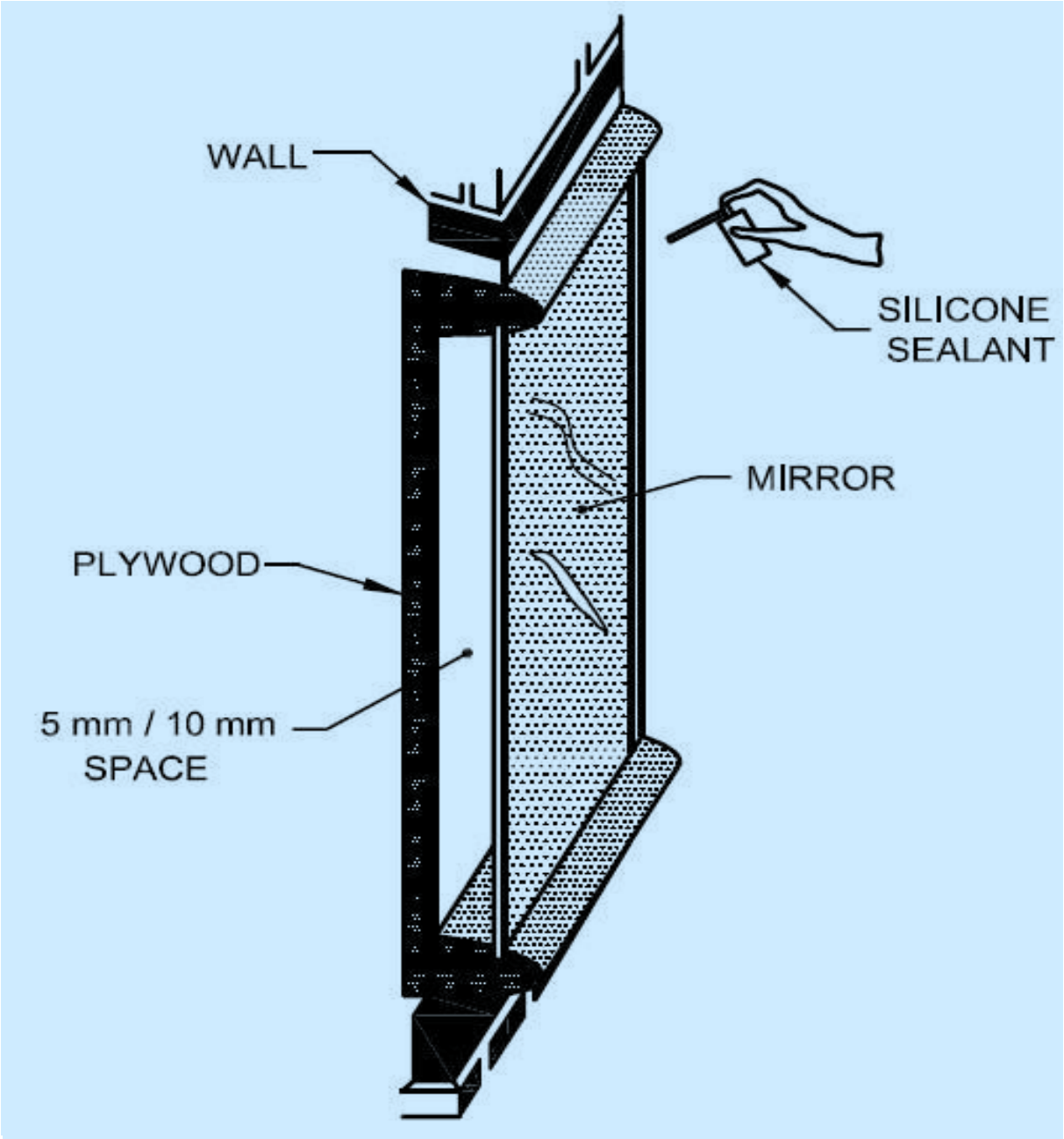

FIG. 24 INSTALLATION OF MIRRORS

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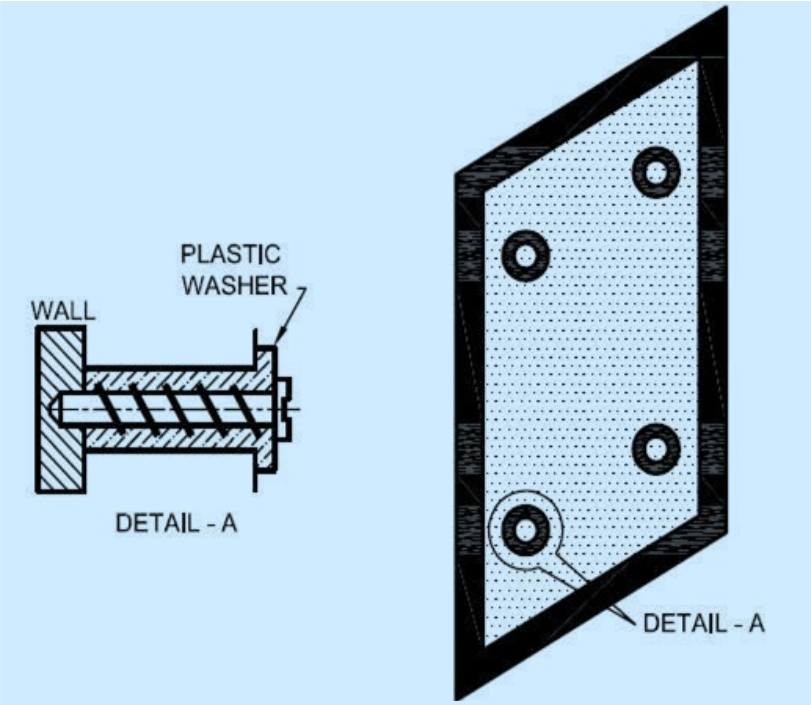

FIG. 25 DETAILS OF USE OF SCREWS AND INTERLEAVING WASHERS FOR INSTALLING MIRRORS

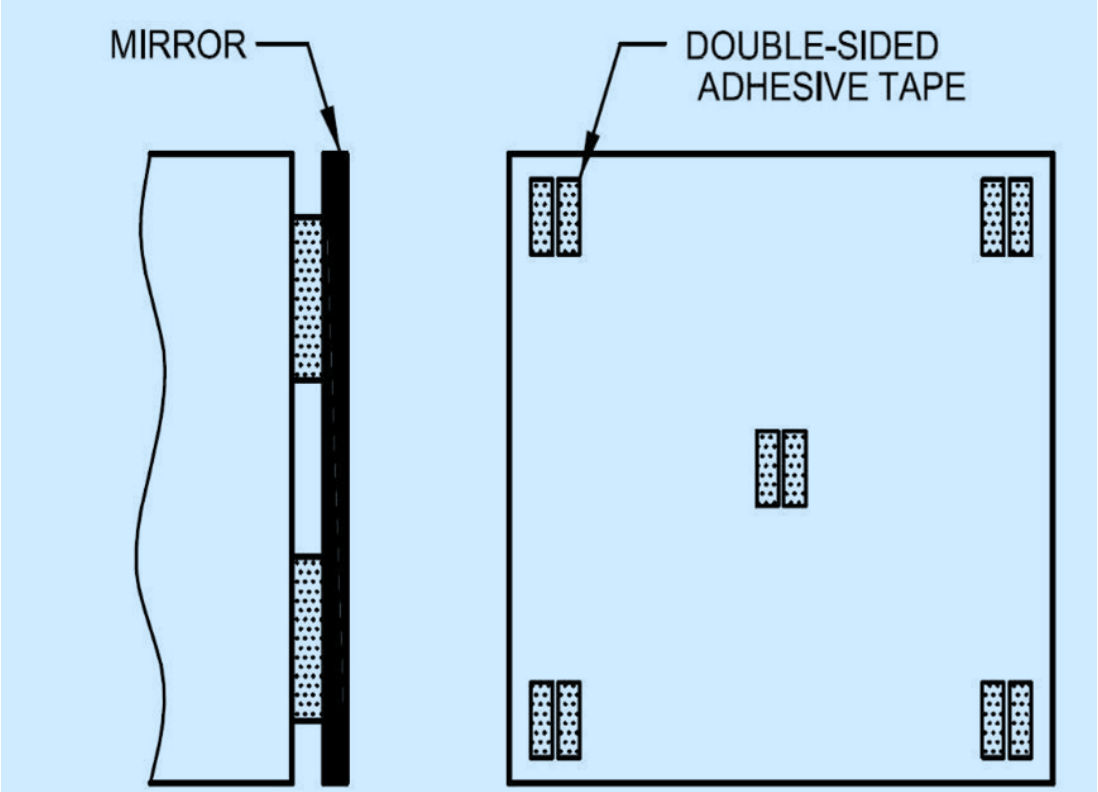

FIG. 26 DETAILS OF USE OF DOUBLE SIDED ADHESIVE TAPES FOR INSTALLING MIRRORS

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ANNEX J

(*Foreword*)

GLASS RAILINGS

J-1 GENERAL

Glass railings are generally sought in institutional, commercial, residential buildings and special structures like stadiums, foot over bridges, etc. Glass of appropriate thickness when used as a railing can provide transparency and when engineered properly can provide structural performance too. In public places such as stadiums, foot over bridges, shopping malls, etc, use of frameless glass railing shall be avoided and framed systems with vertical balustrades and glass infill panel may be used with proper design as per the design provisions.

The basic requirement of a parapet, which a glass railing (if so used) intends to substitute, shall be met with. This includes protecting people from falling and in some cases to control movement. Careful consideration, including in design, shall be made/determined where glass railings are used.

NOTE — Notwithstanding the above, the potential benefits including safety (through psychological aspects) of regular railings shall be borne in mind.

J-2 PRELIMINARY CONSIDERATIONS

In the assessment of the need for a railing and the type of railing to be provided, the designer should give consideration to the likely hazards, the building use and the risks to building users. Where, in a building, more than one use of the building is anticipated, either the railing design should be chosen to suit the worst case, or more than one type of railing should be provided, as appropriate, to the location. Generally, when there is a change in adjacent levels (600 mm in dwellings, 380 mm in other buildings) a railing is needed to restrict movement of people.

The railing adopted should be designed so as to minimize the risk of persons falling, rolling, sliding or slipping through gaps in the railing. In dwellings and other buildings which can be accessed by children, gaps in railing or infill should not be large enough to permit a sphere of 100 mm diameter to pass through, making due allowance for deflection under load.

The designer should ensure all the elements of the railing system satisfy the criteria given in available relevant standards.

For general safety, the finished barrier should have no sharp edges or projections that could cause injury to persons or clothing. Infill panels and balusters are intended to provide support and protection to users, and should be designed to restrain people without causing additional injury from sharp edges, thin sections, open-ended tubes, projecting details, etc.

The design shall be based on the horizontal uniformly distributed load (UDL) applied at a notional height of 1.1 m above finished floor level, even though the minimum permissible height of the railing may be less (the notional design height of 1.1 m is chosen as being representative of the height of the centre of gravity of a human being above the datum level).

J-3 GLASS RAILING TYPES

There are four common types of railings that are used to fix glass.

a) *Full Height Railing* — Where glass forms part or whole of a wall element, the railing should be carefully designed as per the relevant standards and as per this Section, if any part of the glass comes below the minimum railing height (*see* Table 35 and Fig. 27).

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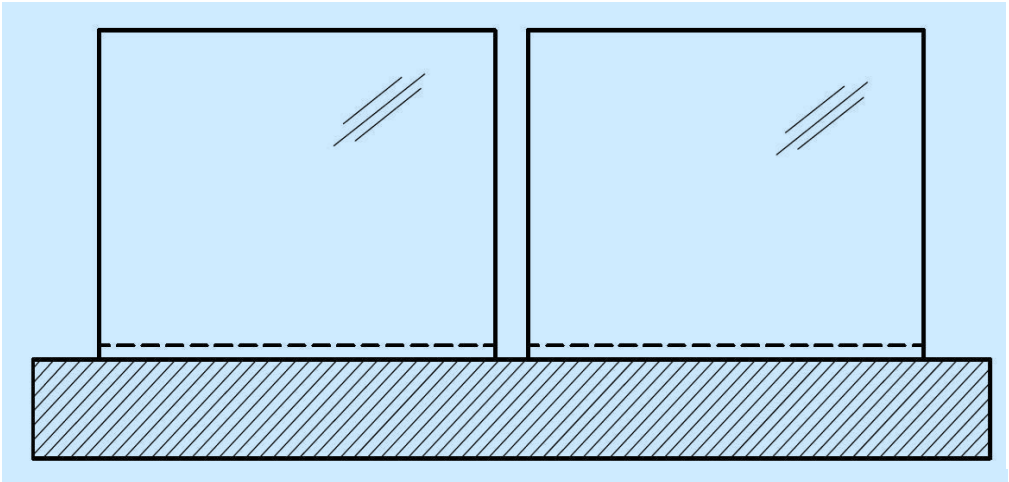

FIG. 27 FULL HEIGHT RAILING

b) *Railings with a Glass Infill Panel* — In this type of railing, the main frame of the railing (that is, top rail and baluster) should be designed to withstand the loads applied to the top rail and the glass should be used to form infill panels. It should be noted that the glass provides no support to the main frame. The designer shall ensure that, irrespective of the fixing type, in the event of glass breakage, the broken glass remains securely attached to its framing or supporting structure, thereby preventing safety hazards. This can be further sub-categorized into 4 types:

1) *Glass supported on vertical edges only* — This may involve the vertical edges of the glass being supported in a rebate into the side of each vertical post. The top and bottom edges of the glass are exposed. The glass may also be supported by patch fittings fixed to the posts. In all cases a handrail is required (*see* Fig. 28).

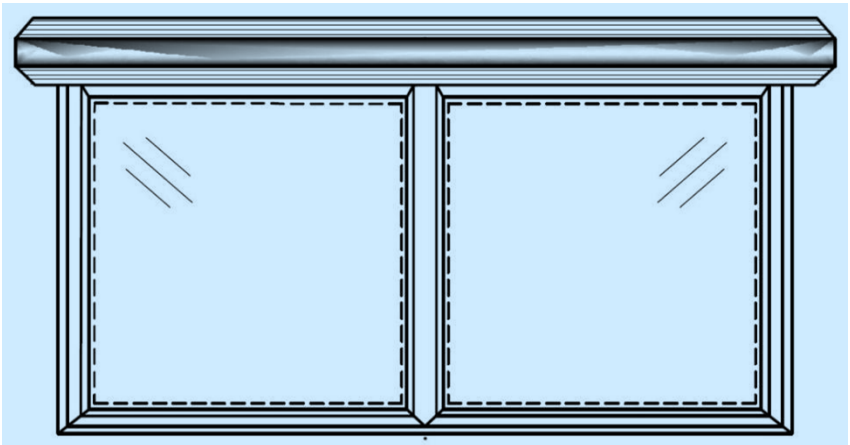

FIG. 28 GLASS SUPPORTED ON VERTICAL EDGES ONLY

2) *Glass supported on horizontal edges only* — This may involve a handrail at the top, and a channel or a rail at the base of the glass. In this case the handrail should be designed to carry the railing load with the glass designed to carry the infill load (*see* Fig. 29).

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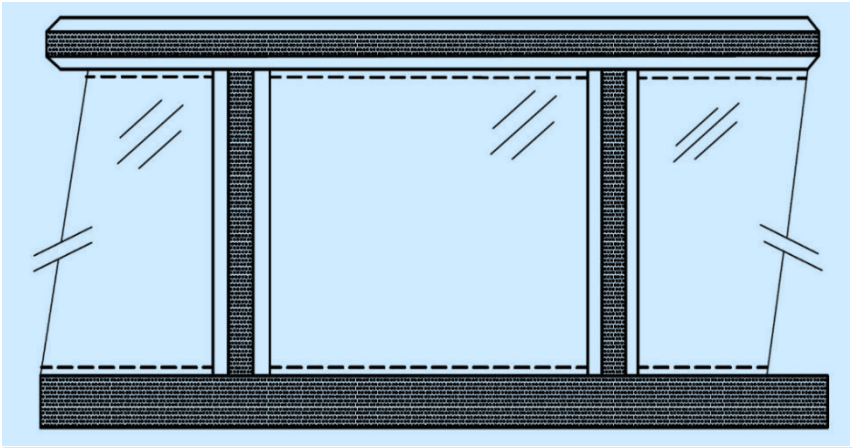

FIG. 29 GLASS SUPPORTED ON HORIZONTAL EDGES ONLY

3) *Glass supported on all four edges* — (*See* Fig. 30)

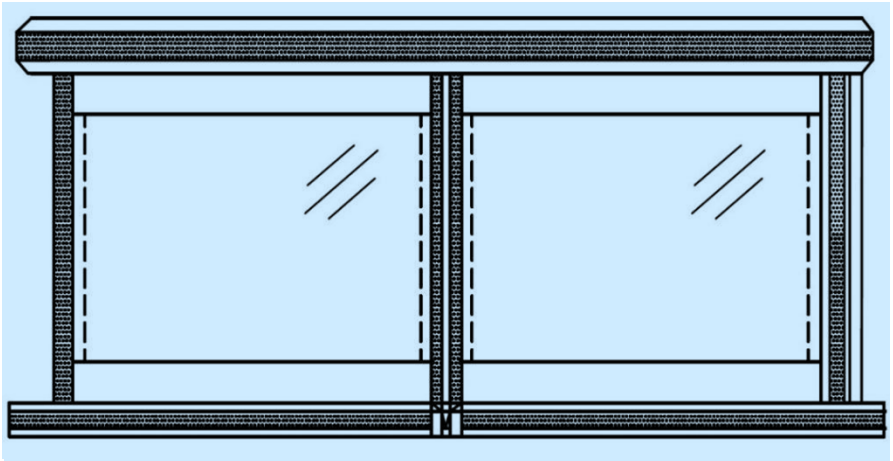

FIG. 30 GLASS SUPPORTED ON ALL FOUR EDGES

4) *Glass fixed by patch/bolted fittings* — This system requires the use of two rows of bolts (commonly) to provide a stiff fixing which minimizes panel deflection. There are a variety of bolt fixing types available in the market, so design and performance information should be sought for each type. The purpose of these fixings is to transfer the force applied to the railing into the support structure. These fixings occupy a small surface area on the glass which can produce high stresses in the glass, so careful analysis is required to determine suitable glass thickness, fixing spacing and size. Glass fixings shall be designed to avoid high local stresses and minimize the risk of failure. Minimum fixing sizes shall be used, and for large panels exceeding 2 m in dimension or under high wind pressure exceeding 2.5 kPa, fixings shall accommodate rotations by using articulated fixings (*see* Fig. 31).

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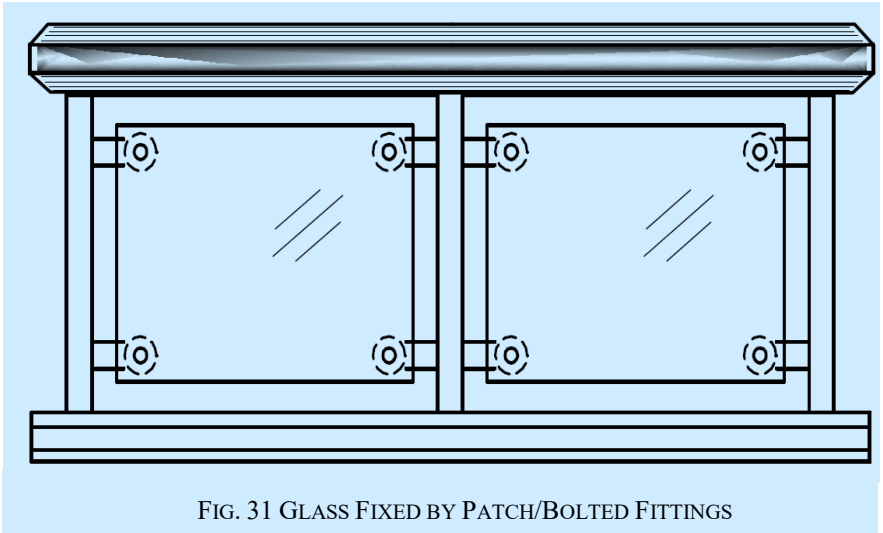

FIG. 31 GLASS FIXED BY PATCH/BOLTED FITTINGS

c) *Free Standing Glass Protective Railing* — In this type of railing, the glass should be designed to withstand the design loads. Each glass plate should be rigidly clamped to the structure along its bottom edge, the handrail attached to the top edge of the glass and there should be no balusters. In the event that a free standing railing is supplied without a handrail, each panel should be able to withstand the entire appropriate design load. This applies to vertical glazing and glass within 15º of vertical. Adequate provision for drainage of water should be made in the base structure to avoid any water ponding which may cause delamination issue. Laminated safety glass shall be used for this application. Clamping and brackets shall be designed with adequate anchorage, and the main supporting building elements (beam/slab) shall be verified to meet the required minimum thickness. The bottom glass clamping embedment shall be a minimum of 100 mm, regardless of railing size. The bottom clamping member shall provide continuous support along the glass bottom edge. In the event of glass breakage, the glass panel shall remain securely in place without risk of detachment or falling (*see* Fig. 32).

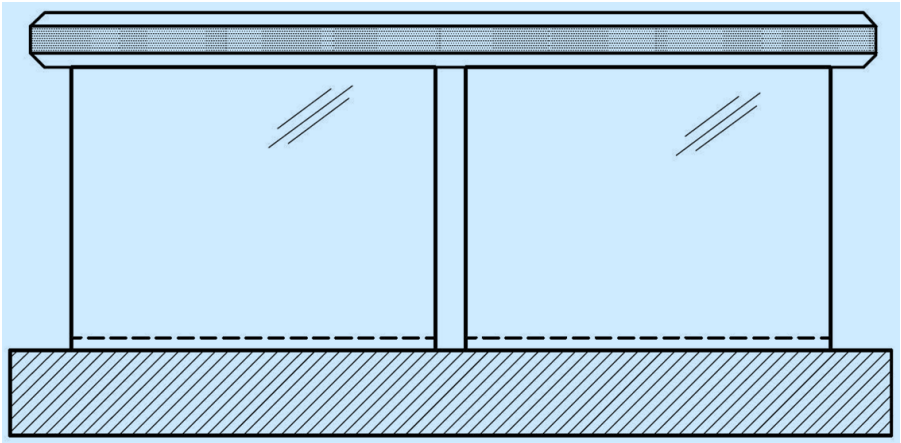

FIG. 32 FREE STANDING GLASS PROTECTIVE RAILING

d) *Bracket Systems* — These systems are an extension of the bolt fixed system where the bolts are part of a bracket and the bracket is fixed to the floor or the face of a concrete slab, steel or timber structure. The use of these systems shall be investigated carefully as they can develop very high stresses in the glass where the brackets are located. The design shall balance glass thickness and bracket location to produce an adequate system. Some systems may be designed to produce test data, to justify their use. This data shall be studied carefully as there have been some cases where such data has been shown to be inadequate. Proper interpretation of these test data shall also become necessary (*see* Fig. 33).

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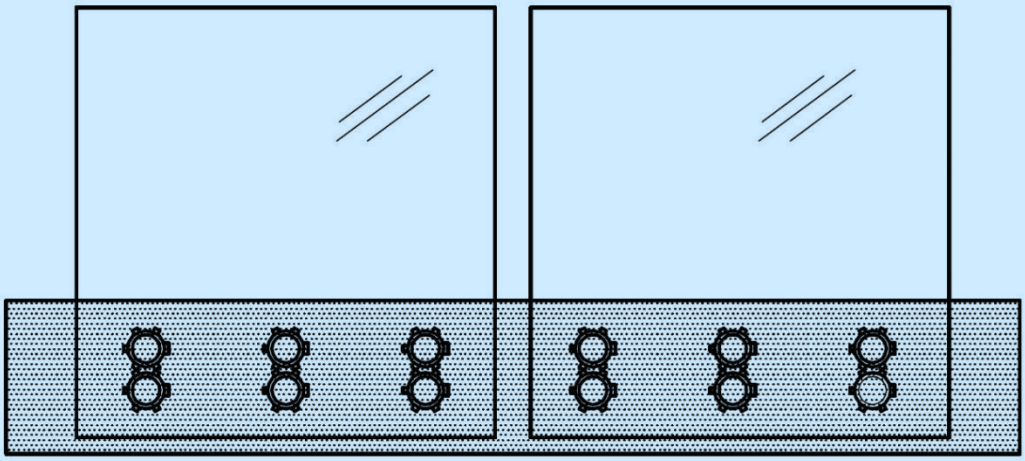

FIG. 33 BRACKET SYSTEMS

e) *Bracket Systems* — Bottom supported with surface attached/bolted Handrail — Glazing as structural member (*see* Fig. 34).

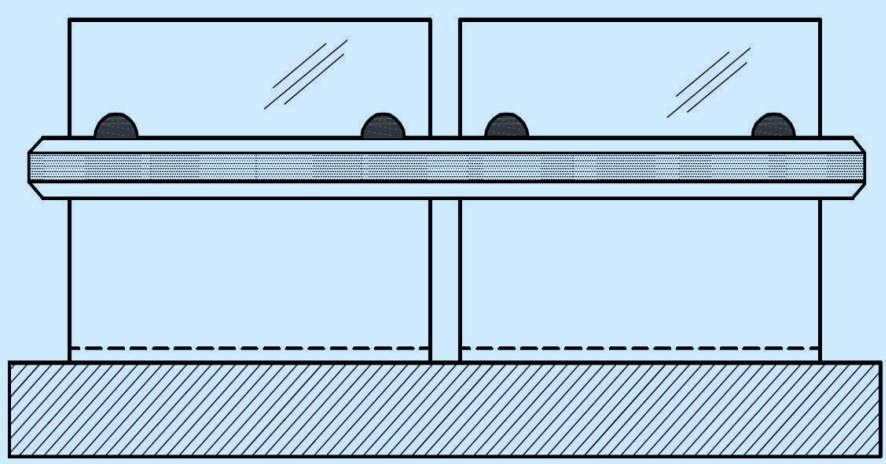

FIG. 34 BOTTOM SUPPORTED WITH SURFACE ATTACHED/BOLTED HANDRAIL — GLAZING AS STRUCTURAL MEMBER

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Table 35 Minimum Barrier Heights

[*Clauses* J-3 (a), J-4.1, J-5.4.1 *and* J-6.1]

Sl No.	Use	Position	Height mm
(1)	(2)	(3)	(4)
i)	Single-family dwelling	a) Barriers in front of a window up to 10 floors	800
		b) Barriers in front of a window from 11th floor to 20th floor	1 100
		c) Barriers in front of a window above 21st floor	1 200
		d) Stairs, landings, ramps, edges of internal floors	900
		e) External balconies including *balconet*, edges of roofs:	
		1) up to 20th floor	1 100
		2) 21st floor and above	1 200
ii)	All other uses	a) Barrier in front of a window	800
		b) Stairs	900
		c) Balconies and stands, etc having fixed seating within 530 mm of the barrier	800
		d) Balconies and stands, etc having fixed seating within 530 mm of the barrier, provided the sum of the barrier width and the barrier height is greater than 975 mm	750
		e) Other positions including balconets	1 100

NOTE — Height shall be measured from the top edge of the railing to the highest point of the floor where occupants can stand. This measurement shall be from the top of any ledges, up-stands, or toe walls, if present at the base.

J-4 DESIGN CONSIDERATIONS

The designer shall first determine the purpose of the railing. Once this is done, the appropriate load requirement can be derived. Typical values (minimum requirements) of railing loads to be chosen are given in Table 36 (If so desired by the designer, higher values than that in the Table 36 may also be chosen).

J-4.1 General

Railings should be designed to resist the most unfavourable likely imposed loads and wind loads separately without unacceptable deflections or distortions. Heights of railings shall be same as that for parapet walls as per other Parts of this NBCS. The railing height above datum is defined in Fig. 35. The minimum heights for each railing location shall be as given in Table 35.

The height of glass railings installed on top of low parapet walls should be measured from the top of the parapet and not at walk level.

For railings, either permissible stress or limit state design procedure should be used, according to the recommended procedures. When using limit state design, the partial safety factors for loads and materials should be those recommended by the appropriate standard for relevant materials. The deflection of the railing shall be determined as per serviceability limit state.

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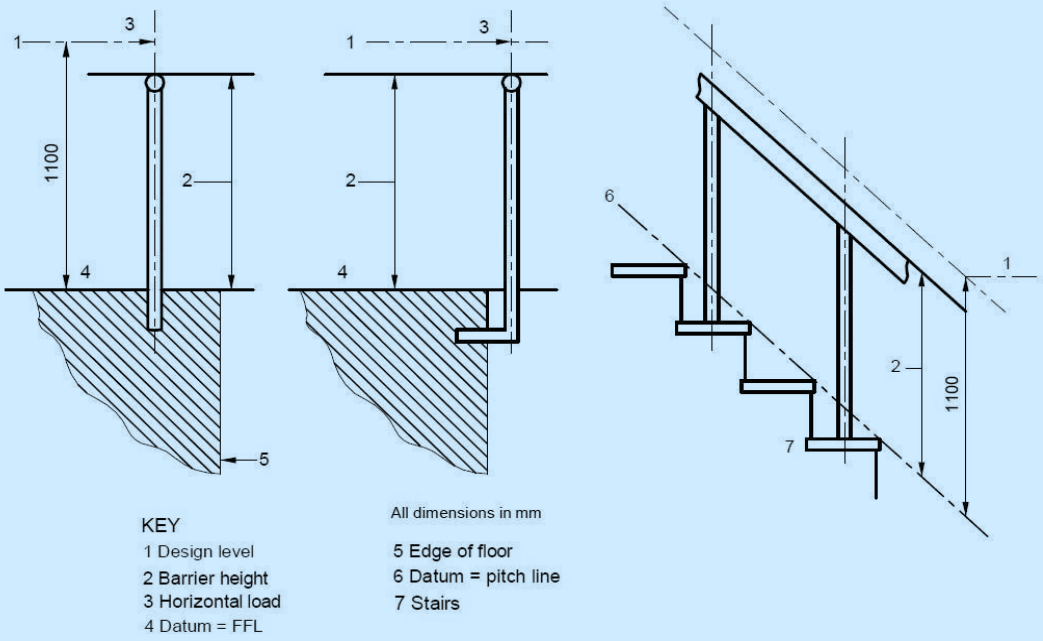


NOTE — In design, the horizontal uniformly distributed line load acts at a height of 1.1 m above datum level, irrespective of the actual height of the element.

FIG. 35 DATUM, BARRIER HEIGHT AND DESIGN HEIGHT

J-4.2 Loading

Minimum horizontal imposed loads appropriate to the design of parapets, railings, balustrades and other elements of structure intended to retain, stop or guide people, should be determined in accordance with Table 36, which recommends a uniformly distributed line load for the railing and a uniformly distributed and point load applied to the infill.

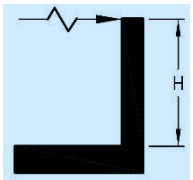
NOTE — These are not additive and should be considered as three separate load cases, all loads being determined according to the type of occupancy which reflects the possible in-service conditions.

Horizontal uniformly distributed line load should be applied at design height as presented in Table 36 or at design level (1 100 mm) for railings higher than the design height.

Uniformly distributed load should be applied to the area below the design height.

Point load should be applied at the most onerous point anywhere on the railing structure.

NOTE — For clarity, the design level (the level at which the horizontal uniformly distributed line load ought to be considered to act) is shown in Fig. 36.

Sl No.	*Building*	*Element*	*Strength*	*Height* (H)	
(1)	(2)	(3)	(4)	(5)	
i)	Single family dwelling	Stairs landings, ramps edges of internal floors	0.36 kN/m	900 mm for all elements	
		External balconies and edges of roofs	0.74 kN/m	1 100 mm	

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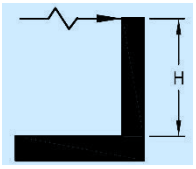
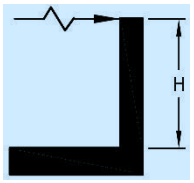
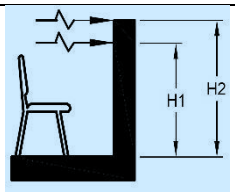
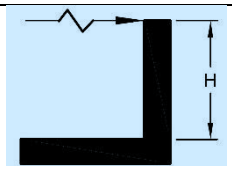
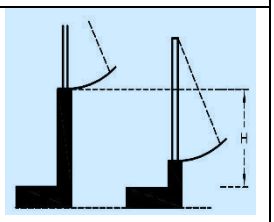
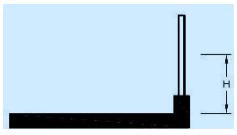
Sl No.	*Building*	*Element*	*Strength*	*Height* (H)	
(1)	(2)	(3)	(4)	(5)	(6)
ii)	Factories and warehouses (light traffic)	Stairs, ramps	0.36 kN/m	900 mm	
		Landing and edges of floor	0.36 kN/m	1 100 mm	
iii)	Residential institutional, educational, office and public building	All location	0.74 kN/m	900 mm for flights otherwise 1 100 mm	
iv)	Assembly	Within 530 mm in front of fixed seating	1.5 kN/m	800 mm (H1)	
		All other locations	3.0 kN/m	900 mm for flights elsewhere 1 100 mm (H2)	
v)	Retail	All locations	1.5 kN/m	900 mm for flights otherwise 1 100 mm	
vi)	All buildings	At opening windows		800 mm	
		At glazing to changes of level	To provide containment	Below 800 mm	

NOTE — *H* is the height at which the applied load (horizontal design load) is located and it need not be the height of the barrier itself.

FIG. 36 HEIGHT LIMIT FOR THE UNIFORM LOADING FOR VARIOUS CATEGORY OF BUILDINGS

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Table 36 Minimum Horizontal Imposed Loads for Parapets, Barriers and Balustrades
[*Clauses* J-4, J-4.2, J-4.4 *and* J-4.6]

Sl No.	Type of Occupancy for Part of the Building or Structure	Examples of Specific Use	Horizontal Uniformly Distributed Line load	Uniformly Distributed Load Applied to the Infill	A Point Load Applied to Part of the Infill
			kN/m	kN/m^2	kN
(1)	(2)	(3)	(4)	(5)	(6)
i)	Domestic and residential activities	a) All areas within or serving exclusively one single family dwelling including stairs, landings, etc but excluding external balconies and edges of roofs	0.36	0.5	0.25
		b) Other residential, that is houses of multiple occupancy and balconies, including Juliette balconies and edges of roofs in single family dwellings	0.74	1.0	0.5
ii)	Offices and work areas not included elsewhere, including storage areas	a) Light access stairs and gangways not more than 600 mm wide	0.22	—	—
		b) Light pedestrian traffic routes in industrial and storage buildings except designated escape routes	0.36	0.5	0.25
		c) Area not susceptible to overcrowding in office and institutional buildings, also industrial and storage buildings except as given above	0.74	1.0	0.5
iii)	Areas where people might congregate	Areas having fixed seating within 530 mm of the barrier, balustrade or parapet	1.5	1.5	1.5

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Table 36 (*Concluded*)

Table 36 Minimum Horizontal Imposed Loads for Parapets, Barriers and Balustrades
[*Clauses* J-4, J-4.2, J-4.4 *and* J-4.6]

Sl No.	Type of Occupancy for Part of the Building or Structure	Examples of Specific Use	Horizontal Uniformly Distributed Line load	Uniformly Distributed Load Applied to the Infill	A Point Load Applied to Part of the Infill
			kN/m	kN/m^2	kN
(1)	(2)	(3)	(4)	(5)	(6)
iv)	Area with tables or fixed seatings	Restaurants and bars	1.5	1.5	1.5
v)	Areas without obstacles for moving people and not susceptible to overcrowding	a) Stairs, landings, corridors, ramps	0.74	1.0	0.5
		b) External balconies including Juliette balconies and edges of roofs. Footways and pavements within building curtilage adjacent to basement/sunken areas	0.74	1.0	0.5
vi)	Areas susceptible to overcrowding	a) Footways or pavements less than 3 m wide adjacent to sunken areas	1.5	1.5	1.5
		b) Theatres, cinemas, discotheques, bars, auditoria, shopping malls, assembly areas, studio. Footways or pavements greater than 3 m wide adjacent to sunken areas.	3.0	1.5	1.5
vii)	Retail areas	All retail areas including public areas of banks/building societies or betting shops	1.5	1.5	1.5
viii)	Vehicular	Pedestrian areas in car parks, including stairs, landings, ramps, edges or internal floors, footways, edges of roofs	1.5	1.5	1.5

In general, the following load combinations as described in **J-4.2.1** to **J-4.2.3** should be applied.

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J-4.2.1 *Free Standing Glass Protective Railing* (*Without any Balusters*)

It requires four design loadings to be taken into account:

a) Infill load, in kN;

b) Uniform load, in kN/m;

c) Concentrated load, in kN; and

d) Shot bag test load, in Joules.

J-4.2.2 *Railing with Glass Infill Panel*

It requires four design loadings to be taken into account:

a) Infill load, in kN;

b) Uniform load, in kN/m;

c) Concentrated load, in kN; and

d) Shot bag test load, in Joules.

J-4.2.3 *Full Height railing*

It requires four design loadings to be taken into account:

a) Infill load, in kN;

b) Uniform load, in kN/m;

c) Concentrated load, in kN; and

d) Shot bag test load, in Joules.

J-4.3 Wind Loads

External railings should be designed to resist the wind loads given in good practice [C-8(15)].

J-4.4 Deflection

Railings for the protection of people should be of adequate strength and stiffness to sustain the applied loads given in Table 36. In addition, a railing that is structurally safe should not possess flexibility to such an extent so as to create a false alarm to building users when subject to normal service conditions. Therefore, for serviceability considerations, the limiting condition for deflection appropriate for a railing for the protection of people is that the total horizontal displacement of the railing at any point from its original unloaded position should not exceed the deflection limits determined from the relevant structural design code (where applicable) for the material used, or 25 mm, whichever is the smaller. Where the infill of a railing is subjected to the imposed loads given in Table 36, or if appropriate, other calculated design loads, the displacement of any point of the railing should not exceed H/65 or 25 mm, whichever is lower. These deflection limits shall apply to the deflection of the cantilevered top edge of the railing, measured perpendicular to the glass plane or the direction of the wind load.

J-4.5 Fixings, Attachments and Anchorage

The strength of fixings, attachments and anchorage securing the railing to a substrate should be adequate to sustain a loading greater than that to which the railing will be subjected. All joints should be designed to provide the full strength of the members being joined. To that end, where any uncertainty exists with regard to the strength of any component in the fixing, the design loading factors should be increased by 50 percent.

While designing fixings, particular care should be taken to account for the material into which the fixing is placed, the spacing between fixings, the edge distance, and where the substrate is concrete, the position of reinforcement. Building elements (beam/slab/up-stand) shall have adequate thickness and shall be composed of structural

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reinforced cement concrete (RCC) or steel. The designer shall exercise caution when the thickness of the concrete element is less than 100 mm, as adequate embedment may not be achievable. The designer shall ensure that the concrete element is cast with high quality and shall refrain from fixing to any loose concrete. Design of anchors shall conform to the requirements of accepted standard [C-8(30)].

NOTES

1 The recommendation of an additional load factor of 1.5 is intended to ensure that under an extreme load condition, railings give an indication of failure by deflection or distortion and not by total collapse, as may be brought about by a failure of the fixing, attachment or anchorage system. Where the design strength of a proposed system of fixing to an existing substrate cannot be determined with reasonable accuracy by theoretical consideration, load testing should be used to validate the design.

2 A factor greater than 1.5 on railing load design might also be chosen based on the importance desired by the designer/parties concerned.

J-4.6 Testing

It is recommended to conduct the tests as detailed in this clause, on railings provided for the type of occupancy classified as 'Areas susceptible to overcrowding', as specified in Table 36. For all other occupancy types, the tests to be conducted on railings for performance evaluation, shall be mutually decided between the parties concerned.

J-4.6.1 *Test Selection*

Load test methods for the uniform load test and the concentrated load test are selected based on the performance specification.

J-4.6.2 *Sequence of Test Method*

Specimens tested for two or more of the loading conditions are subjected first to the in-fill load test, followed by the uniform load test, and then the concentrated load test and shot bag glazing impact test, steel nose pendulum impact, and sphere penetration evaluation, in that order, unless directed otherwise by the performance specification.

J-4.6.2.1 *Apparatus and load application*

a) *Assembly support fixture* — An assembly support fixture shall ensure the rigidity normally provided to an assembly in a building by the ceiling, floor, and walls. The support fixture for the specimen shall consist of a vertical wall section constructed from nominal steel or 2 by 4 wood studs, 400 mm on centre, with a rough opening of sufficient size to support the test specimen in accordance with the manufacturer's specifications. The limiting deflection of the wall shall be L/175 (based on the anticipated loads);

b) *Load attachments* — Brackets, fasteners, or other devices used in performing these tests shall be designed and attached so as to minimize their influence on the test results;

c) *Shot bag, traction and release system* — The shot bag shall be of weight 45.4 kg as described in **6.4.2.5** and **6.4.6** of accepted standard [C-8(3)]. The suspension device shall conform to **6.4.2.6** of accepted standard [C-8(3)] and the Traction Release system shall conform to **6.4.2.7** of accepted standard [C-8(3)];

d) *Load measuring device* — Load and time-measuring devices with an accuracy of ± 2 percent of the full scale shall be incorporated in the test setups. The scale ranges used shall assure that the performance levels are within an accuracy of ± 5 percent; and

e) *Rate of loading* — The rate of loading shall be such that the maximum specified load is achieved in no less than 10 s and no more than 5 min**.** Maintain the maximum specified load for 1 min before releasing. [Warnings: 1) Possible injury to personnel and damage to the test equipment and instrumentation prior to, during, and after load application by any unexpected release of potential strain energy accumulated during testing can occur and shall be given consideration. 2) If tests are conducted in a structure and not in a testing laboratory, exercise caution against unwanted damage to the building, its components, and its finish].

For in-fill testing, use a bearing plate with appropriate cushioning between the plate and the glazing. The bearing plate should be 0.09 m^2.

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Each load test to be performed on a new specimen or the undamaged specimen from the previously applied test. Perform only one test per glazing in-fill type on each of the three specimens at the most critical point of the system.

J-4.6.3 *Sampling*

A minimum of three representative replicate specimens of each type of system shall be tested. Sequential testing of the same specimen shall be permissible provided no breakage of any component occurs during the previously performed tests.

J-4.6.4 *Test Specimens*

The specimen installation, including the post spacing, shall be the same as the actual field installation. The specimen shall have a minimum of three posts or minimum two largest glass panels. Each unique attachment scenario shall be tested.

Multiple types of infill panels shall be qualified within one test, provided each panel is individually glazed into the system prior to the start of the test. No substitution of infill product shall be permitted once testing has commenced.

J-4.6.5 *Installation*

The glass rail, guard, or balustrade system being tested shall be installed in accordance with the manufacturer's or designer's specifications.

J-4.6.6 *Test Procedure*

J-4.6.6.1 *In-fill load test*

The test specimen shall be capable of satisfactorily resisting the required test load applied over a 0.09 m^2 area normal to the in-fill. The test load shall be applied at the intersection of the horizontal and vertical centre lines of the in-fill, unless specified otherwise by the performance specification or the purposes of the test. If it is determined that another location will yield lesser results, that position should be used instead of the indicated intersection point of the centre lines. Record deflection of the in-fill at the point of maximum deflection of the in-fill.

J-4.6.6.2 *Uniform load test*

Subject the top rail or other rail of the test specimen to the required maximum uniform test load applied vertically and horizontally, as required by the performance specification. Required vertical and horizontal test loads shall be applied separately and sequentially, unless otherwise required by the performance specification or to meet the purpose of the test. Record the deflection at mid-span during each test.

J-4.6.6.3 *Concentrated load test*

The top rail or other subject rail of the test specimen shall be subjected to the concentrated test load applied vertically and horizontally, as required by the performance specification. Apply required vertical and horizontal test separately and sequentially, unless otherwise required by the performance specification or to meet the purpose of the test. The required concentrated test load shall be applied at the following three critical locations – rail mid-span between posts, top rail adjacent to a post (maximum 75 mm from post) and top of a single post. Test loads are to be applied and deflection recorded sequentially, one location at a time. The load applied on the rail adjacent to the post can be applied to the opposite post from which the top-of-post-test was performed. In the case of swing gates, if the structure of the gate requires locating the test load more than 75 mm from a post due to hinge or latch construction, the load should be located as close to the 75 mm maximum distance from the post as is possible. Record the deflection at the point of application of the load.

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J-4.6.6.4 *Shot bag impact test*

Any protective masking or decorative components shall be removed from the glazing material.

To position the impactor at the selected drop height, a traction force shall be applied to raise the impactor such that the axis of the impactor shall be aligned with the suspension cable, with the cable remaining taut. To ensure this, the top and bottom ends of the impactor shall be connected to the release device by a suitable link.

To reduce bag deformation during testing, the bag shall be rotated about the axis of its suspension device before each specimen or sample set, by no less than 30º, and by no more than 90º.

To reduce bag damage during testing, a thin homogeneous or non-woven plastic film no more than 0.13 mm thick or a loosely draped woven cloth towel weighing no more than 0.05 g/cm^2 shall be permitted to be suspended vertically from its top edge directly in front of the surface of the specimen at a distance no more than 10 mm.

The impactor shall be suspended from an overhead support, located so when at rest it will, at its maximum diameter, be located no more than 12 mm from the surface of the specimen and no more than 50 mm from the impact location of the glazing infill specimen.

Raise the impactor to the required height (460 mm, 1 220 mm or 1 525 mm) and stabilize it. The suspension device shall be taut and the axes of the impactor and cable shall be aligned.

The impactor, stabilized in the launch position in a vertical plane normal to the test specimen, shall be released and falls without initial velocity or axial rotation.

If the glazing does not break, the impactor is to be raised to the next designated height and glazing impacted from the new height. Glazing found to be unbroken and remaining completely in the frame after impact shall be recorded as intact.

J-5 GLASS

The use of glass in railings, the type and thickness of glass required for various support conditions shall be as given below. If designed properly, glass can form a structural component of the railing, or the railing may be designed to have a structural component made from another material such as steel or aluminium, with glass acting as an infill panel only. This places a lesser load on the glass as the major load is taken by the structural component.

NOTE — There are certain situations, particularly under some lighting conditions, where the presence of transparent glass is not readily evident and suitable manifestation should be provided.

J-5.1 Types of Glass

It is recommended to use laminated safety glass for all types of glass railing applications, whether framed or frameless. In the event of glass breakage, the glass panel shall remain in place without falling from its position.

J-5.2 Working of Glass

When toughened glass or heat-strengthened glass is used in laminated glass, it shall conform to accepted standard [C-8(7)] for toughened glass and accepted standard [C-8(8)] for heat-strengthened glass.

J-5.3 Fixing of Glass

Contact between glass and any other hard material (including other glass parts) should be prevented. Rubber gaskets or other glazing materials should be used with frame sections. The frame section should give a minimum of 15 mm edge cover to the glass unless it can be shown by test or calculation that the frame will retain the glass under load while conforming to design requirements.

The frame and/or fastenings should be designed so that they do not tend to distort the glass panel. This is particularly important with bolted connections, which can exert considerable forces on the glass, where particular attention should be paid to the alignment and position of the fasteners in order to avoid unnecessary stresses being developed.

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J-5.3.1 *Bolt Fixing of Infill Panels*

At the bolted connections there should be clamping plates and gaskets on both sides of the glass that provide a minimum of 50 mm diameter cover to the glass. These plates should be not less than 6 mm thick in steel or 10 mm thick in other suitable metals.

Where the length of a glass pane is greater than the span between the bolted connectors, giving rise to a cantilevered portion of the pane, the cantilevered portion should be less than one-quarter of the span between the bolted connectors.

Under the design loads, the railing should be designed so that the relative in-plane movement of the bolted connections in the same panel is not greater than 2 mm.

The glass, framing system and connections of railings and infill panels should be capable of sustaining and safely transmitting the design loads to the supporting structure.

J-5.4 Design of Glass

J-5.4.1 *Design of Glass in Full Height Protective Railings*

Glass partly or totally below the minimum railing height should be designed to satisfy the appropriate design criteria. Any part of a glass pane below the minimum railing height should sustain the infill loads. Where there is glass at the minimum railing height given in Table 35, the glass should also sustain the line load applied at the design level.

For any specific information on the glass size relative to the glass type, support system, loadings and thickness, available standards or the manufacturer recommendations should be sought.

J-5.4.2 *Design of Glass Infill Panels*

Infill panels should be designed to satisfy the appropriate design criteria as below:

a) *Fully framed infill panels* — The deflection of the glass in fully framed infill panels should be as per recommended deflections;

b) *Two-edge framed infill panels* — The deflection of the glass in two-edge framed infill panels should be as recommended under **J-4.4** taking *L* as the distance between the supports;

c) *Clipped infill panels* — The clips should be positioned around the periphery of the infill panel, at a maximum spacing of 600 mm. Each clip should be not less than 50 mm in length and should give a minimum depth of cover to the glass of 25 mm;

d) *Bolt fixing of glass infill panels* — Where glass is supported by bolted connections through holes in the glass, toughened/toughened laminated glass should be used; and

e) *Position of infill panels relative to the main frame* — Infill panels should be fully contained within the supporting structure. In order not to apply unintended loads to the infill panels, they should have handrails above the glass or attached to the side to which the public have access.

J-5.4.3 *Design of Free-Standing Glass Protective Railings*

The glass should be designed to satisfy the appropriate design criteria given in **J-4**. The deflection of the glass should not be more than 25 mm at any point.

J-5.4.3.1 *Handrail attachment*

Where the railing protects a difference in level greater than 600 mm, a handrail shall be used unless a laminated safety glass construction is used that remains *in-situ*, if a panel fails. Continuous fixing should be used for fixing the handrail to the glass, or individual fixings where calculations or tests demonstrate that component failure will not occur.

Where handrail is not desired, the laminated safety glass should stay in place in the event of any glass panel breakage.

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The handrail should be attached to the glass in such a manner that, should a glass panel gets fractured, the handrail,

a) will remain in position; and

b) will not fail if the design load is applied across the resulting gap.

NOTE — The above condition (b) can be relaxed where the glass pane is an end pane and protects a difference in level of 600 mm or less, for example at the foot of a flight of stairs. In cases where an end pane protects a difference in level greater than 600 mm, there may normally be some adjacent structure to which the handrail could be attached, thus enabling it to meet condition (b).

J-5.4.4 *Base Fixing of Free-Standing Railings*

J-5.4.4.1 *Point of fixing clamps*

The fixing clamps on each side of the glass should be not less than 100 mm to 150 mm and should be made of a suitable metal of minimum thickness 12 mm. The number and design of fixing clamps shall be in accordance with structural requirements.

J-5.4.4.2 *Continuous fixing clamps*

The fixing clamps on each side of the glass should be not less than 100 mm wide and should be made of a suitable metal of minimum thickness 12 mm. The clamps should be continuous for the entire length of the glass pane and have a maximum bolt spacing of 500 mm.

J-5.4.4.3 *Other clamping systems*

Different clamping methods could be used, provided they cater for an effective continuous clamping over the length of the glass pane. Where a clamping system that does not rely on bolts through the glass is used, the depth over which the clamping force operates should be not less than 75 mm, unless specific tests have been carried out to prove the integrity of the system and that it meets the design criteria given.

J-5.4.4.5 *Structural movement*

The structural movement due to live or dead loads or creep of the structure to which a railing pane is clamped should be not greater than 2 mm over the length of any railing panel, if more than two bolts are used to clamp that panel.

J-5.4.4.6 *Fixing clamps*

The attachment of the fixing clamps to the structure should be capable of withstanding the turning moment induced at the fixing clamps.

J-6 IMPACT RESISTANCE AND CONTAINMENT

J-6.1 General

In addition to resisting the design loads given, glass below the minimum railing height (*see* Table 35) should also be able to resist impact forces appropriate to whether the railing indicates a route or protects people from a hazard.

J-6.2 Glass in Full Height Railings

The glass in full height railings should be selected to resist the appropriate design loads given and for its impact performance in accordance with the safety glazing recommendations.

J-6.3 Railing with Glass In-fills or Free-Standing Balustrades

The glass in railings with glass in-fills or free-standing should be selected to resist the appropriate design loads given, and to provide containment, that is, it should meet the recommended impact class without penetration.

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J-7 INSTALLATION

The installation of a railing should be supervised by a suitably qualified person or persons, who should ensure that the design assumptions have been effectively implemented. Inspections and investigations should be carried out as necessary to establish the integrity of the materials and the elements of construction used.

J-7.1 Fasteners and Fittings

Fasteners and fittings should be of stainless steel, or should be hot-dip galvanized. Thin zinc plated or cadmium plated coatings should not be used for external exposure without additional protection.

J-8 MAINTENANCE

All glass should be regularly cleaned and fixings should be checked for corrosion and loosening. Maintenance recommendations, and the frequency at which railings should be inspected, are be governed by environment, usage, incidence of malicious damage and the protection applied. Protective railings assembled entirely from metal-coated steel and situated in normal environments should not require any maintenance for a very long period. They should, however, be inspected regularly and the residual coating thickness should be measured.

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ANNEX K

(*Informative*)

(*Foreword*)

CONFORMITY ASSESMENT SHEET FOR GLASS AND GLAZING SYSTEM

K-1 For the purpose of conformity assessment by building regulator for glass and glazing aspects, following sheet shall be used.

Sl No.	Parameters	Details
(1)	(2)	(3)
i) GENERAL		
	a) Building typology	
	b) Location (City, State)	
	c) Orientation	
	d) Climate zone	
	e) Window wall ratio (WWR)	
ii) GLASS		
	a) Component of building (Glass on roof/floor/wall/railing/glass fin)	
	b) Type of Glass:	
	Combination (if needed)	
	c) Tinted/Clear/Coated:	
	if yes, Side (Outer/Inner)	
	Description	
	d) Patterned/Non- patterned:	
	if yes, Side (Outer/Inner)	
	Dimension of Glass (L × B) (mm)	
	e) Thickness and Deflection:	
	Calculated Deflection as per this NBCS	
	Calculated Thickness (mm) as per this NBCS	
	Actual Thickness (mm)	
	Glass specifications as per this NBCS	**Yes/No**
	f) Safety:	

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Sl No.	Parameters	Details
(1)	(2)	(3)
	Glass Height from Ground	
	Presence of Residual Protection? (Yes / No)	
	Presence of Window Restrictor for children?	
	Anti-Fallout norms followed? (Yes / No)	
	Safety glass norms followed as per this NBCS	**Yes/No**
	g) Energy Performance:	
	U-value	
	SHGC/Solar factor	
	Visible light transmittance (VLT)	
	Shading co-efficient	
iii) FRAME		
	a) Thickness	
	b) Dimension	
	c) Frame Material	
	d) Frame supported sides (Glass inside structures can also be considered as framed)	
	e) Deflection of frames	
	f) Face width	
	g) U-value of frame	
	h) Depth of the profile	
	Frame specifications as per this NBCS	Yes/No
iv) GLAZING SYSTEM		
	a) Thickness and Dimension	
	Dimension	
	Thickness	
	b) Energy	
	U-factor	

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Sl No.	Parameters	Details
(1)	(2)	(3)
	Solar Factor / SHGC	
	Visible Light Transmittance (VLT)	
	Energy efficiency specifications as per this NBCS	Yes/No
	c) Structural parameters	
	Design load and wind load	
	Other loads	
	Air Permeability	
	Deflection	
	Seismic parameters	
	Water tightness (classification)	
	Structural aspects as per this NBCS	Yes/No
	d) Glazing Material	
	Front and back clearance	
	Edge clearance	
	Edge cover	
	Rebate depth	
	e) Fire Resistant Glazing system:	
	If Fire Resistant, Category (E/EW/EI)	
	Duration of resistance to fire (in min)	
	Thickness of glazing system	
	f) Acoustics	
	Airborne Sound Insulation	
	Impact Sound Insulation	
v) THIRD PARTY TEST CERTIFICATES (FOR ALL PRESCRIBED PROPERTIES)		

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Sl No.	Parameters	Details
(1)	(2)	(3)
	a) Glass	Yes/No
	b) Frames	Yes/No
	c) Glazing systems — Energy performance	Yes/No
	d) Glazing systems — Structural aspects	Yes/No
	e) Glazing systems — Fire resistance (if applicable)	Yes/No
	f) Glazing systems — Acoustics	Yes/No

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LIST OF STANDARDS

The following list records those standards which are acceptable as 'good practice' and 'accepted standards' in the fulfilment of the requirements of this NBCS. The latest version of a standard shall be adopted at the time of enforcement of this NBCS. The standards listed may be used for conformance with the requirements of the referred clauses in this NBCS.

In the following list, the number appearing in the first column within parentheses indicates the number of the reference in this Section:

	IS No.	*Title*
(1)	IS 14900 : 2018	Transparent float glass — Specification (*first revision*)
	IS 2553 (Part 1) : 2018	Safety glass — Specification: Part 1 Architectural, building and general uses (*fourth revision*)
(2)	IS 3808 : 1979	Method of test for non-combustibility of building materials (*first revision*)
(3)	IS 17004 : 2018	Testing methods for processed glass
(4)	IS 19331 : 2025	Tinted float glass — Specification
(5)	IS 2835 : 1987	Specification for flat transparent sheet glass (*third revision*)
(6)	IS 14900 : 2018	Specification for Transparent float glass (*first revision*)
(7)	IS 2553 (Part 1) : 2018	Safety glass — Specification: Part 1 Architectural, building and general uses (*fourth revision*)
(8)	IS 16982 : 2018	Heat strengthened glass — Specification
(9)	IS 17346 : 2020	Insulating glazing unit — Specification
(10)	IS 5437 : 2024	Rolled glass — Patterned, extra clear patterned, wired and wired-patterned glass — Specification (*second revision*)
(11)	Doc no. CED 13 (27189)	Structural glazing silicone sealants – Specification (*under preparation*)
(12)	IS 1948 : 2024	Aluminium framed doors, windows and sliders — Specification (*first revision*)
(13)	SP 32 : 1986	Handbook on functional requirements of industrial buildings (lighting and ventilation)
	SP 41 : 1987	Handbook on functional requirements of buildings (other than industrial buildings)
(14)	IS 18648 : 2024	Classification and performance requirements for doors, windows and sliders — Specification
(15)	IS 875 (Part 3) : 2015	Design loads (other than earthquake) for buildings and structures — Code of practice: Part 3 Wind loads (*third revision*)
(16)	IS 1893 (Part 1) : 2016	Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings (*sixth revision*)
(17)	IS 875 (Part 2) : 1987	Code of Practice for design loads (other than earthquake) for buildings and structures: Part 2 Imposed loads (*second revision*)
(18)	IS 19328 : 2025	Fire resistant glass — Specification
(19)	IS/ISO 834-1 : 1999	Fire-resistance tests — Elements of building construction: Part 1 General requirements
	IS/ISO 834-4 : 2000	Fire-resistance tests — Elements of building construction: Part 4 Specific requirements for load bearing vertical separating elements

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	IS No.	*Title*
	IS/ISO 834-5 : 2000	Fire-resistance tests — Elements of building construction: Part 5 Specific requirements for load bearing horizontal separating elements
	IS/ISO 834-6 : 2000	Fire-resistance tests — Elements of building construction: Part 6 Specific requirements for beams
	IS/ISO 834-7 : 2000	Fire-resistance tests — Elements of building construction: Part 7 Specific requirements for columns
	IS/ISO 834-8 : 2002	Fire-resistance tests — Elements of building construction: Part 8 Specific requirements for non-load bearing vertical separating elements
	IS/ISO 834-9 : 2003	Fire-resistance tests — Elements of building construction: Part 9 Specific requirements for non-load bearing ceiling elements
	IS 17518 (Part 1) : 2022/ ISO 3008-1 : 2019	Fire resistance tests — Door and shutter assemblies: Part 1 General requirements
	IS/ISO 3009 : 2003	Fire-resistance tests — Elements of building construction — Glazed elements
	IS 16945 : 2018	Fire resistance test for glass walls
	IS 16947 : 2018	Fire resistance tests for doors with glass panes, openable glass windows and sliding glass doors
(20)	IS 18190 : 2023	Fire resistance of perimeter fire barrier joint system — Method of tests
(21)	IS 16231 (Part 3) : 2019	Use of glass in buildings — Code of practice: Part 3 Fire and loading (*first revision*)
	IS 875	Code of Practice for design loads (other than earthquake) for buildings and structures:
	(Part 1) : 2026	Dead loads — Unit weights of building, construction and stored materials (*third revision*)
	(Part 2) : 1987	Imposed loads (*second revision*)
(22)	IS 16231 (Part 3) : 2019	Use of glass in buildings — Code of practice: Part 3 Fire and loading (*first revision*)
(23)	IS 17953 : 2023	uPVC profiles for windows and doors — Specification
(24)	IS 1038 : 1983	Specification for steel doors, windows and ventilators (*third revision*)
(25)	IS 11050 (Part 1) : 2023/ ISO 717-1 : 2020	Rating of sound insulation in buildings and of building elements: Part 1 Airborne sound insulation (*first revision*)
(26)	IS 11050 (Part 2) : 2023/ ISO 717-2 : 2020	Rating of sound insulation in buildings and of building elements: Part 2 Impact sound insulation (*first revision*)
(27)	IS 875	Code of practice for design loads (other than earthquake) for buildings and structures:
	(Part 1) : 2026	Dead loads — Unit weights of building, construction and stored materials (*third revision*)
	(Part 2) : 1987	Imposed loads (*second revision*)
	(Part 3) : 2015	Wind loads (*third revision*)
	(Part 5) : 1987	Special loads and load combinations (*second revision*)

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	IS No.	*Title*
(28)	IS 733 : 2025	Wrought aluminium and aluminium alloy bars, rods, sections, extruded tubes, hollow sections and profiles for general engineering purposes — Specification (*fourth revision*)
	IS 738 : 1994	Specification for wrought aluminium alloys, drawn tube for general engineering purposes (*third revision*)
(29)	IS 6911 : 2026	Stainless steel plate, sheet and strip – Specification (*third revision*)
(30)	IS 1946 (Part 2) : 2025	Design of post-installed anchorage to concrete — Code of practice
(31)	IS 1946 (Part 5) : 20xx	Installation and on-site inspection of post-installed anchorage to concrete — Code of practice (*under preparation*)
(32)	IS 18434 : 2023	Field measurement of air permeability and water penetration through installed curtain wall, windows, doors, sliders and skylights under static air pressure difference — Method of test
(33)	IS 18459 : 2024	Water penetration of curtain walls, windows, sliders, doors and skylights by uniform static air pressure difference — Method of test
(34)	IS 18647 : 2024	Water penetration of curtain walls, windows, sliders and doors by dynamic air pressure — Method of test
(35)	IS 18268 : 2023/ ISO 9379 : 2005	Operating forces — Test methods — Doors
(36)	IS 17909 : 2022/ ISO 8274 : 2005	Windows and doors — Resistance to repeated opening and closing — Test method
(37)	IS 18473 : 2024	Wind resistance for curtain walls, windows, sliders, doors and skylights — Method of test
(38)	IS 17911 : 2022/ ISO 15099 : 2003	Thermal performance of windows, doors and shading devices — Detailed calculations
(39)	IS 17920 : 2022/ ISO 10077-2 : 2017	Thermal performance of windows, doors and shutters — Calculation of thermal transmittance — Numerical method for frames
(40)	IS 19006 : 2023/ ISO 8248 : 1985	Windows and door height windows — Mechanical tests
(41)	IS 17518 (Part 1) : 2022/ ISO 3008-1 : 2019	Fire resistance tests — Door and shutter assemblies: Part 1 General requirements
(42)	IS 16945 : 2018	Fire resistance test for glass walls
(43)	IS 16947 : 2018	Fire resistance tests for doors with glass panes, openable glass windows and sliding glass doors
(44)	IS 16978	Glass in building — Forced-entry security glazing: Part 1 Test and classification by repetitive ball drop (*first revision*)
	(Part 1) : 2022/ ISO 16936-1 : 2020	Test and classification by repetitive ball drop (*first revision*)
	(Part 2) : 2018/ ISO 16936-2 : 2005	Test and classification by repetitive impact of a hammer and axe at room temperature
	(Part 3) : 2018/ ISO 16936-3 : 2005	Test and classification by manual attack
	(Part 4) : 2018/ ISO 16936-4 : 2005	Test and classification by pendulum impact under thermally and fire stressed conditions
(45)	IS 18518 : 2023	Bullet resistant security glass — Quality and performance requirements

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	IS No.	*Title*
(46)	IS 16978	Glass in building — Forced-entry security glazing:
	(Part 1) : 2022/ ISO 16936-1 : 2020	Test and classification by repetitive ball drop (*first revision*)
	(Part 2) : 2018/ ISO 16936-2 : 2005	Test and classification by repetitive impact of a hammer and axe at room temperature
(47)	IS 18472 : 2024	Air permeability for curtain walls, windows, sliders, doors and skylights — Method of test
(48)	IS 875	Code of Practice for design loads (other than earthquake) for buildings and structures:
	(Part 1) : 2026	Dead loads — Unit weights of building, construction and stored materials (*third revision*)
	(Part 2) : 1987	Imposed loads (*second revision*)
	(Part 3) : 2015	Wind loads (*third revision*)
	(Part 4) : 2021	Snow loads (*third revision*)
	(Part 5) : 1987	Special loads and load combinations (*second revision*)
	IS 1893 (Part 1) : 2016	Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings (*sixth revision*)
	IS 16700 : 2023	Criteria for structural safety of tall concrete buildings *(first revision)*

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INDEX OF TERMS

(Lists the terms appearing in the Part/Section of the NBCS)

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*A*95 — Part D/Section 5A

Acceleration — Part D/Section 5A, Part D/ Section 5B

Access Panel — Part E/Section 1

Access Point — Part D/Section 6

Accessibility — Part D/Section 5A, Part F

Accessory — Part D/Section 2

Accidental Loads — Part C/Section 6A, Part C/ Section 6B

*Accompanying Lo*ad — Part C/Section 6A, Part C/ Section 6B

Action — Part C/Section 6A, Part C/Section 6B

Action Effect or Load Effect — Part C/Section 6A, Part C/Section 6B

Actual Length — Part C/Section 6A

Admixtures (*Including Superplasticizer and Accelerator*) — Part C/Section 7

Air Admittance Valve — Part E/Section 2

*Air Brea*k — Part E/Section 2

Air Change per Hour (*ACPH*) — Part D/Section 1, Part F

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Air Conditioning — Part D/Section 3

Air Gap, Drainage — Part E/Section 2

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Alternating Current Variable Voltage Variable Frequency (*ACVVVF*) *Control* — Part D/Section 5A

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Ambient Noise — Part D/Section 4

*Anaerobic Diges*tion — Part E/Section 3

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Approved Agency — Part E/Section 4

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Apron — Part D/Section 5A

Area of Parking — Part D/Section 5C

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Authorities Concerned — Part F

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Automatic Fire Detection and Alarm System — Part F

Automatic Operation — Part D/Section 5A

Automatic Parking System — Part D/Section 5C

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Automobile Lift — Part D/Section 5A

Auxiliary or Emergency or Safety Brake — Part D/Section 5B

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Available Head — Part E/Section 1

Available Car Area — Part D/Section 5A

A-Weighted Sound Pressure — Part D/Section 4

A-Weighted Sound Pressure Level — Part D/Section 4

Axial Flow Fan — Part D/Section 1

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Background Noise — Part D/Section 4

Balancing Weight — Part D/Section 5A

Balustrade — Part C/Section 8, Part D/Section 5B

Balustrade Decking — Part D/Section 5B

Bamboo — Part C/Section 3B

Bamboo Bore/GHOON Hole — Part C/Section 3B

Bamboo Clump — Part C/Section 3B

Bamboo Culm — Part C/Section 3B

Bamboo Mat Board — Part C/Section 3B

Bamboo Mat Corrugated Sheets — Part C/ Section 3B

Bamboo Mat Corrugated Sheet — Part C/Section 3B

Bamboo Mat Ridge Cap (*BMRC*) — Part C/Section 3B

Bamboo Mat Veneer Composite — Part C/Section 3B

Barrel — Part E/Section 1

Barrier — Part D/Section 2

Base — Part C/Section 1, Part E/Section 1

Base Layer — Part C/Section 7

Basic Protection — Part D/Section 2

Basic Module — Part C/Section 7

Bath Room Group — Part E/Section 1, Part E/ Section 2

Bather Load — Part E/Section 1

Batter Pile (*Raker Pile*) — Part C/Section 2

Battery Charging Stations — Part D/Section 2

Battery Energy Storage System (*BESS*) — Part D/ Section 2

Battery of Fixtures — Part E/Sction 2

Beads or Glazing Beads — Part C/Section 8

Beam — Part C/Section 3B, Part C/Section 6A

Beam, Built-Up-Laminated — Part C/Section 3A

Beam, Glued-Laminated — Part C/Section 3A

Bearing Capacity, Safe — Part C/Section 2

Bearing Capacity, Ultimate — Part C/Section 2

Bearing Pressure, Allowable (*Gross or Net*) — Part C/Section 2

Bearing Type Connection — Part C/Section 6A

Bed Block — Part C/Section 4

Bed Pan Washer — Part E/Section 2

Bedding — Part E/Section 1

Benching — Part E/Section 2

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Biodegradable Substance — Part E/Section 3

Biomethanation — Part E/Section 3

Black Water — Part E/Section 2

Block (*Setting Block*) — Part C/Section 8

Bond — Part C/Section 4 Section 7

Bonding Conductor — Part D/Section 2

Bonding Ring Conductor (*BRC*) — Part D/Section 2

Bored Cast In-situ Pile — Part C/Section 2

Bored Compaction Pile — Part C/Section 2

Bored Pile — Part C/Section 2

Bottom Car Clearance — Part D/Section 5A

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Bottom Counterweight Run-by — Part D/Section 5A

Bottom Refuge Space — Part D/Section 5A

Box System — Part C/Section 4

Braced Member — Part C/Section 6A

Brake Load — Part D/Section 5B

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Branch Vent Pipe (*BVP*) — Part E/Section 2

Branch Waste Pipe (*BWP*) — Part E/Section 2

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Breadth — Part C/Section 1
Break-In — Part D/Section 4
Breaking Strength — Part C/Section 3B
Break-Out — Part D/Section 4
Bressummer — Part C/Section 3A
Brightness Ratio or Contrast — Part D/Section 1
Brittle Cladding — Part C/Section 6A
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Buckling Load — Part C/Section 6A
Buckling Strength or Resistance — Part C/Section 6A
Buffer — Part D/Section 5A
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Building Drain-Combined — Part E/Section 2
Building Drain-Sanitary — Part E/Section 2
Building Drain-Storm — Part E/Section 2
Building Trap — Part E/Section 2
Building Energy Simulation (*BES*) — Part F, Part D/Section 3
Buildability — Part C/Section 7
Building Height — Part C/Section 5
Building Integrated Renewable Energy — Part D/Section 3
Building Line — Part A
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Building Sub-drain — Part E/Section 2
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Burried Direct — Part D/Section 2
Bus Bar — Part D/Section 6
Busbar Trunking System — Part D/Section 2
Business Buildings — Part C/Section 1
Buttress — Part C/Section 4
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C

C_{TR} — Part D/Section 4
Cable — Part D/Section 2
Cable, Fire Survival — Part D/Section 2
Cable, Flame Retardant (*FR*) — Part D/Section 2
Cable, Flame Retardant Low Smoke Halogen Free (*FR-LSH*) — Part D/Section 2
Cable, Flexible — Part D/Section 2
Cable, Metal-Sheathed — Part D/Section 2
Cable, PVC Sheathed-insulated — Part D/Section 2
Cable, Weatherproof — Part D/Section 2
Cable, XLPE — Part D/Section 2
Cable Armoured — Part D/Section 2
Cable Bracket — Part D/Section 2
Cable Channel — Part D/Section 2
Cable Cleat — Part D/Section 2
Cable Coupler — Part D/Section 2
Cable Ducting — Part D/Section 2
Cable Ladder — Part D/Section 2
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Cable Trunking — Part D/Section 2
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Camber — Part C/Section 6A and 6B
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Car Door Electric Contact — Part D/Section 5A
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Catwalk — Part F
Ceiling Rose — Part D/Section 2
Cell — Part C/Section 3B
Cellular Concrete — Part C/Section 7

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Cellulose — Part C/Section 3B
Central Field – Part D/Section 1
Centre Internode — Part C/Section 3B
Centrifugal Fan — Part D/Section 1
Cesspool — Part E/Section 2
Chair — Part E/Section 1
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*Channe*l — Part E/Section 1
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Circuit — Part D/Section 2
Circuit, Final Sub — Part D/Section 2
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Circuit Breaker, Linked — Part D/Section 2
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Clear Design Sky — Part D/Section 1
Clear Glass — Part C/Section 8
Clear Waste Water — Part E/Section 2
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Closely-spaced Modules — Part C/Section 1
Coaxial Cable — Part D/Section 6
Coefficient of Linear Expansion — Part C/Section 8
Coefficient of Performance, Compressor, Heat Pump — Part D/Section 3
Coefficient of Performance, Compressor, Refrigerating — Part D/Section 3
Coefficient of Performance (*Heat Pump*) — Part D/Section 3
Coefficient of Performance (*Refrigerating*) — Part D/Section 3
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Comb Plate Switch — Part D/Section 5B
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Combined System — Part E/Section 2
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Compensation Means — Part D/Section 5A
Competent Authority — Part E/Section 4
Competent Maintenance Person — Part D/ Section 5A, Part D/Section 5B
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Conductor, Bare — Part D/Section 2
Conductor, Earthed — Part D/Section 2
Conductor, Insulated — Part D/Section 2
Conduit — Part D/Section 2
Connection — Part E/Section 2
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Connector for Portable Appliances — Part D/ Section 2
Consolidation Point (*CP*) — Part D/Section 6

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Constant Stress Range — Part C/Section 6A

Construction and Demolition Waste — Part E/ Section 3

Consumer — Part E/Section 1, Part E/Section 2

Consumer's Pipe — Part E/Section 1

Consumer's Terminals — Part D/Section 2

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Core — Part C/Section 5

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Cross-Connection — Part E/Section 1

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Dedicated Outdoor Air System (*DOAS*) — Part D/Section 3

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Deep Manhole – Part E/Section 2
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Deflector Sheave — Part D/Section 5A
Delamination — Part C/Section 3B
Demand Based Ventilation — Part D/Section 3
Demand Recirculation Water System — Part E/Section 1
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Design Acceleration — Part C/Section 1, Part D/ Section 5A
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Design Life — Part C/Section 6A
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F

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Flameproof Enclosure — Part D/Section 2
Flattened Bamboo — Part C/Section 3B
Flight Time — Part D/Section 5A
Flexible Shear Connector — Part C/Section 6 B
Flexural Stiffness — Part C/Section 6A
Float Glass — Part C/Section 8
Float Operated Valve — Part E/Section 1
Flood Level Rim — Part E/Section 1, Part E/Section 2
Floor Area (*Gross*) — Part F
Floor Area Ratio (*FAR*) — Part F
Floor Area (*Net*) — Part F
Floor Levelling Switch — Part D/Section 5A
Floor Stopping Switch — Part D/Section 5A
Flow Pressure — Part E/Section 1
Flush Over Panel — Part C/Section 8
Flushing Cistern — Part E/Section 1
Flushometer Tank — Part E/Section 1
Foam Protection System — Part F
Footing — Part C/Section 2
Foyer — Part F
Force Coefficient — Part C/Section 1
Formation — Part E/Section 1, Part E/Section 2
Foundation — Part C/Section 2

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Foundation, Raft — Part C/Section 2
Frame — Part C/Section 8
Frame Building — Part C/Section 5
Framed-Tube System — Part C/Section 5
Frameless Glazing — Part C/Section 8
Free-Field Level — Part D/Section 4
French Drain or Rubble Drain — Part E/Section 2
Frequency — Part D/Section 4, Part D/Section 6
Fresh Air or Outside Air — Part D/Section 1
Friction Type Connection — Part C/Section 6A
Front Putty — Part C/Section 8
Frost Line — Part E/Section 1, Part E/Section 2
Full Culm — Part C/Section 3B
Full Load Pressure — Part D/Section 5A
Fully Framed Glazing — Part C/Section 8
Functional Earthing — Part D/Section 2
Fundamental or Ultimate Stress — Part C/Section 3A, Part C/Section 3B

G

Gap — Part C/Section 8
Gas Fitter — Part E/Section 4
Gas-Based Suppression Systems — Part F
Gauge — Part C/Section 6A
Geared Machine — Part D/Section 5A
Gearless Machine — Part D/Section 5A
General Ventilation — Part D/Section 1
General Washing Place — Part E/Section 1
Generator of Wastes — Part E/Section 3
Geothermal Heat Pump — Part D/Section 3
Geyser — Part E/Section 1
Glare — Part D/Section 1
Glass — Part C/Section 8
Glass Appearance Faults — Part C/Section 8
Glazing — Part C/Section 8
Glazing-bite — Part C/Section 8
Glide — Part D/Section 3
Global Warming Potential (*GWP*) — Part D/Section 3
Globe Temperature — Part D/Section 1
Glue — Part C/Section 3A
Glued Laminated (*Glulam*) — Part C/Section 3A
Glued Laminated, Horizontally — Part C/Section 3A
Glued Laminated, Vertically — Part C/Section 3A
Goods Lift — Part D/Section 5A
Gradient Height — Part C/Section 1
Gravel — Part C/Section 2
Gravity Columns — *Part C/Section* 5
Gravity Load — Part C/Section 6A
Grease Interceptor (*or Grease Trap*) — Part E/Section 2
Grey water — Part E/Section 2
Grid — Part D/Section 5C
Gridded Water Distribution System — Part E/Section 1
Ground Improvement — Part C/Section 2
Ground Roughness — Part C/Section 1
Ground-Borne Noise — Part D/Section 4
Group Automatic Operation — Part D/Section 5A
Grout — Part C/Section 4
Grouted Cavity Reinforced Masonry — Part C/Section 4
Grouted Hollow-unit Masonry — Part C/Section 4
Grouted Masonry — Part C/Section 4
Grouted Multi-Wythe Masonry — Part C/Section 4
Guard Rail — Part C/Section 8
Guide Rails — Part D/Section 5A
Guide Rails Fixing — Part D/Section 5A
Guide Shoe — Part D/Section 5A
Gully Chamber — Part E/Section 2
Gully Trap — Part E/Section 2
Gusset Plate — Part C/Section 6A
Gust — Part C/Section 1

H

Hairline Scratch — Part C/Section 8
Halo — Part C/Section 8
Hand-Held Equipment — Part D/Section 2
Handling Capacity (*HC*) — Part D/Section 5A
Handrail — Part D/Section 5B, Part F
Hardening Behaviour – Part C/Section 5
Harmonics (*Current and Voltage*) — Part D/Section 2
Haunching — Part E/Section 1
Hazardous Live-Part — Part D/Section 2
Hazardous Material — Part F
Headway — Part F

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Heat Pump — Part D/Section 3
Heat Recovery — Part D/Section 3
Heat Soaking — Part C/Section 8
Heat Strengthened Glass — Part C/Section 8
Heating Load — Part D/Section 3
Headroom/Overhead — Part D/Section 5A
Heel Rest Bend or Duck-foot Bend — Part E/Section 1
Hemi Cellulose — Part C/Section 3B
Hermetic Seal — Part C/Section 8
High Activity Area — Part C/Section 8
High Altitudes — Part E/Section 1, Part E/Section 2
High Rise Building — Part F
High Risk Area — Part C/Section 8
High Shear — Part C/Section 6A
Highway Authority — Part E/Section 1, Part E/Section 2
Hoisting Beam — Part D/Section 5A
Hollow Unit — Part C/Section 4
Home Lift — Part D/Section 5A
Horizontal Cabling — Part D/Section 6
Horizontal Cabling Media Distribution and Building Pathway — Part D/Section 6
Horizontal Cross-connect (*HC*) — Part D/Section 6
Horizontal Exit — Part F
Horizontal Movements — Part D/Section 5C
Horizontal Pipe — Part E/Section 1, Part E/Section 2
Hospital Lift — Part D/Section 5A
Hot Water Tank — Part E/Section 1
Humidification — Part D/Section 1
Humidity, Absolute — Part D/Section 1
Humidity, Relative — Part D/Section 1
Hybrid Building — Part D/Section 3
Hydrant System — Part F
Hydraulic Lift — Part D/Section 5A
Hydraulic Parking System — Part D/Section 5C
Hydro-pneumatic Systems — Part E/Section 1
Hydronic Systems — Part D/Section 3
Hydronic System Balancing — Part D/Section 3

I

Illuminance — Part D/Section 1
Impact Sound Pressure Level, L_i — Part D/Section 4
Imposed Load (*Live Load*) — Part C/Section 1, Part C/Section 6A
Impulse Current — Part D/Section 2
Impulse Withstand Voltage — Part D/Section 2
In Building Solution (*IBS*) — Part D/Section 6
Increments — Part C/Section 7
Incidental Occupancy within Station — Part F
Incoming Traffic — Part D/Section 5A
Indirect Acting Lift — Part D/Section 5A
Indirect Contact — Part D/Section 2
Indirect-Direct Cooling — Part D/Section 3
Indirect Waste Pipe — Part E/Section 2
Indoor Air Quality (*IAQ*) — Part D/Section 3
Indoor Ambient Noise — Part D/Section 4
Indoor Artificial Lighting — Part D/Section 1
Industrial Buildings — Part C/Section 1
Industrial Plugs and Sockets — Part D/Section 2
Infill Balustrades — Part C/Section 8
Infiltration/Exfiltration — Part D/Section 3
Inflammable Material — Part D/Section 2
Initial Dead Load — Part C/Section 6B
Initial Load Test — Part C/Section 5B
Injection — Part C/Section 2
Inlet — Part E/Section 1
Inlet Fittings — Part E/Section 2
Inlet Hopper — Part E/Section 2
Inner Diameter — Part C/Section 3B
Insanitary — Part E/Section 2
Insertion Loss — Part D/Section 4
Inside Location — Part C/Section 3A, Part C/Section 3B
Inside Plant (*ISP*) — Part D/Section 6
Inspection Chamber — Part E/Section 2
Inspection Door — Part D/Section 5B
Inspector — Part D/Section 5A
Instability — Part C/Section 6A
Installation — Part D/Section 5A, Part E/Section 4, Part D/Section 5B
Installation (*Electrical*) — Part D/Section 2
Installer — Part D/Section 5A, Part D/Section 5B
Instantaneous Safety Gear — Part D/Section 5A

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Integrated Fire Safety System — Part F
Institutional Buildings — Part C/Section 1
Insulated — Part D/Section 2
Insulating Glass Unit (*IGU*) — Part C/Section 8
Insulation — Part D/Section 2
Insertion Loss (*LIL*) — Part D/ Section 4
Insulation, Basic — Part D/Section 2
Insulation, Double — Part D/Section 2
Insulation, Reinforced — Part D/Section 2
Insulation, Supplementary — Part D/Section 2
Interceptor — Part E/Section 2
Interceptor Manhole or Interceptor Chamber — Part E/Section 2
Inter Floor Traffic — Part D/Section 5A
Interior Panel — Part D/Section 5B
Interlayer — Part C/Section 8
Intermediate Cross-Connect — Part D/Section 6
Internal (*Interior*) *Exit Stairway* — Part F
Internal Partition — Part C/Section 8
Internal Reflected Component (*IRC*) — Part D/ Section 1
Interval — Part D/Section 5A
Invert — Part E/Section 2
Inverter — Part D/Section 2
Internet of Things — Part D/Section 6
Isolation — Part D/Section 2
Isolator — Part D/Section 2
Isoptera — Part C/Section 3A

J

Jack — Part D/Section 5A
Jerk — Part D/Section 5A
Joint — Part C/Section 1, Part C/Section 3B, Part C/Section 4
Joist — Part C/Section 3B
Jointed Detailing System — Part C/Section 7
Joint Reinforcement — Part C/Section 4
Junction Box — Part D/Section 2
Junction Pipe — Part E/Section 1, Part E/Section 2

K

Key Elements — Part C/Section 5
Knot — Part C/Section 3A
Knot Hole — Part C/Section 3A

L

Lagging — Part E/Section 1, Part E/Section 2
Lamella Roof — Part C/Section 3A
Laminated Glass — Part D/Section 5A
Layer — Part C/Section 7
Laminated Safety Glass — Part C/Section 8
Laminated Veneer Lumber — Part C/Section 3A
Laminations — Part C/Section 3A
Landfilling — Part E/Section 3
Landing Call Push — Part D/Section 5A
Landing Door — Part D/Section 5A
Landing Zone — Part D/Section 5A
Large CPRF — Part C/Section 2
Large Lobby — Part D
Large Lobby at the Level of Exit Discharge — Part F
Latent Heat — Part D/Section 3
Latent Heat Load — Part D/Section 3
Lateral Force-Resisting System (*LFRS*) — Part C/ Section 1
Lateral Restraint for a Beam — Part C/Section 6A
Lateral Support — Part C/Section 4
Latewood — Part C/Section 3A
Leachate — Part E/Section 3
Leading Imposed Load — Part C/Section 6A
Leaf — Part C/Section 4
Leakage Current — Part D/Section 2
Length of Internode — Part C/Section 3B
Levelling — Part D/Section 5A
Levelling Accuracy — Part D/Section 5A
Levelling Zone — Part D/Section 5A
Levelling Device, Lift Car — Part D/Section 5A
Levelling Device, One Way Automatic — Part D/Section 5A
Levelling Devices — Part D/Section 5A
Levelling Device, Two-Way Automatic Maintaining — Part D/Section 5A
Levelling Device, Two-way Automatic Non-maintaining — Part D/Section 5A
Levelling Zone — Part D/Section 5A
LEMP Protection Measures (*SPM*) — Part D/ Section 2

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License — Part D/Section 5A
Licensed (*or Registered*) *Plumber* — Part E/Section 1, Part E/Section 2
Lift — Part D/Section 5A
Lift Car — Part D/Section 5A
Lift Landing — Part D/Section 5A
Lift Lobby — Part F
Lift Machine — Part D/Section 5A
Lift Pit — Part D/Section 5A
Lift Ride Quality — Part D/Section 5A
Lift Well — Part D/Section 5A
Lift Well Enclosure — Part D/Section 5A
Light Output Ratio (*LOR*) *or Efficiency* — Part D/ Section 1
Light Pipe — Part D/Section 1
Light Shelf — Part D/Section 1
Light Transmission — Part C/Section 8
Light Transmittance — Part C/Section 8
Light-Weight Concrete — Part C/Section 7
Lighting — Part D/Section 1
Lightning Electromagnetic Impulse — Part D/ Section 2
Lightning Protection — Part D/Section 2
Lightning Protection Level (*LPL*) — Part D/Section 2
Lightning Protection Zone — Part D/Section 2
Lightning Protection System (*LPS*) — Part D/Section 2
Lightning Protection System, External — Part D/Section 2
Lightning Protection System, Internal — Part D/Section 2
Lignin — Part C/Section 3B
Limit State — Part C/Section 6A & 6B
Limiting Capacity — Part C/Section 2
Line Conductor — Part D/Section 2
Linear Defects — Part C/Section 8
Linking Beams (*Also called Coupling Beams*) — Part C/Section 5
Link Load — Part F
Liquefaction — Part C/Section 1 and 2
Live Knot — Part C/Section 3A
Live or Alive — Part D/Section 2
Live Load — Part C/Section 6A
Live Part — Part D/Section 2
Loads — Part C/Section 6A and 6B
Load Bearing Member — Part D/Section 5A
Load Bearing Wall — Part C/Section 4
Load Eccentricity (*eL or eB*) — Part C/Section 2
Loaded Edge Distance — Part C/Section 3A
Loaded End or Compression End Distance — Part C/Section 3B
Local Area Network (*LAN*) — Part D/Section 6
Local Exhaust Ventilation — Part D/Section 1
Local Multipoint Distribution System (*LMDS*) — Part D/Section 6
Location — Part C/Section 3A
Locations, Industrial — Part D/Section 2
Locations, Non-Industrial — Part D/Section 2
Locking Devices — Part D/Section 5C
Loose Grain (*Loosened Grain*) — Part C/Section 3A
Loose Knot — Part C/Section 3A
Lower Inner Decking — Part D/Section 5B
Lower Outer Decking — Part D/Section 5B
Low Rise Building — Part C/Section 1
Low-Voltage Switchgear and Controlgear Assembly — Part D/Section 2
Lumen — Part D/Section 1
Luminaire — Part D/Section 2
Luminance (*At a point of a Surface in a Given Direction*) (*Brightness*) — Part D/Section 1
Luminous Flux — Part D/Section 1

M

m-k Factor — Part C/Section 6 B
Machette — Part C/Section 3B
Machine Room — Part D/Section 5A
Machine Room Less Lift — Part D/Section 5A and 5B
Machine to Machine (*M2M*) *Communication* — Part D/Section 6
Machinery — Part D/Section 5A and 5B
Material Efficiency — Part C/Section 6
Machinery Spaces — Part D/Section 5A and 5B
Maximum Capacity — Part D/Section 5B
Maximum CPRF Settlement (*Spr,Max*) — Part C/Section 2
Made-Up Ground — Part C/Section 2
Main Cross-connect — Part D/Section 6

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Main Distribution Frame (*MDF*) — Part D/Section 6
Main Drain — Part E/Section 1
Main Earthing Terminal — Part D/Section 2
Main Member — Part C/Section 6A
Main Soil Pipe (*MSP*) — Part E/Section 2
Main Soil and Waste Pipe (*MSWP*) — Part E/ Section 2
Main Vent Pipe (*MVP*) — Part E/Section 2
Main Waste Pipe (*MWP*) – Part E/Section 2
Maintenance — Part D/Section 5A, Part D/Section 5B
Maintenance Factor — Part D/Section 1
Maintenance Organization — Part D/Section 5A and 5B
Make-Up Air — Part D/Section 1
Make-Up Water — Part E/Section 1
Manhole — Part E/Section 2
Manhole Chamber — Part E/Section 2
Manifestation — Part C/Section 8
Manifold — Part E/Section 4
Manufacturer — Part D/Section 5A, Part D/ Section 5B
Masonry — Part C/Section 4
Masonry Buildings — Part C/Section 1
Masonry Unit — Part C/Section 4
Mass Rapid Transit — Part F
Mast — Part D/Section 5A
Mast Section — Part D/Section 5A
Mast Tie — Part D/Section 5A
Mat — Part C/Section 3B
Material Efficiency — Part C/Section 6A
Maximum Thickness — Part C/Section 8
Mean Ground Level — Part C/Section 1
Mean Outer Diameter — Part C/Section 3B
Mean Radiant Temperature — Part D/Section 3
Mean Wall Thickness — Part C/Section 3B
Means of Egress — Part F
Means of Escape — Part F
Mechanical Ventilation — Part D/Section 1
Mechanically Laminated — Part C/Section 3A
Mercantile Buildings — Part C/Section 1
Meridian — Part D/Section 1
Meshed Bonding Network (*MESH-BN*) — Part D/Section 2
Metropolitan Area Network (*MAN*) — Part D/Section 6
Mill Tolerance — Part C/Section 6A
Miniature Circuit Breaker (*MCB*) — Part D/Section 2
Minimum Breaking Load — Part D/Section 5A
Minimum Thickness — Part C/Section 8
Mixed Mode Building — Part D/Section 3
*Mixed Occupa*ncy — Part F
Mixed Building or Hybrid Building — Part C/ Section 5
Mobile Equipment — Part D/Section 2
Modular Coordination — Part C/Section 7
Modular Grid — Part C/Section 7
Module — Part C/Section 7
Modulus of Elasticity (*Young's Modulus*) — Part C/Section 8
Moment Frame System — Part C/Section 5
Moment Frame-Structural Wall System — Part C/Section 5
Monitoring — Part D/Section 2
Monolithic Glass — Part C/Section 8
Mortise and Tenon — Part C/Section 3B
Mould — Part C/Section 3A
Moulded Case Circuit Breaker (*MCCB*) — Part D/ Section 2
Moving Walk — Part D/Section 5B
Mullion — Part C/Section 8
Multichannel Multipoint Distribution System (*MMDS*) — Part D/Section 6
Multimodule — Part C/Section 7
Multiple-Tube System — Part C/Section 5
Multi-Tower Structures Linked by Podium — Part C/Section 5
Multiple Earthed Neutral System — Part D/Section 2
*Multiple Occupa*ncy — Part F
Multi-User Telecommunications Outlet Assembly (*MUTOA*) — Part D/Section 6
Municipal Authority — Part E/Section 3
Municipal Solid Waste — Part E/Section 3

N

Natural Ventilation — Part D/Section 1
Natural Period — Part C/Section 5

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Naturally Conditioned Building — Part D/Section 3

Negative Skin Friction — Part C/Section 2

Net Section — Part C/Section 3B

Neutral Conductor — Part D/Section 2

Newel — Part D/Section 5B

Node — Part C/Section 3B

Noise — Part D/Section 4

Noise Exposure Forecast (*NEF*) — Part D/Section 4

Noise Criteria — Part D/Section 4

Noise Rating (NR) — Part D/Section 4

Noise Reduction Co-efficient (*NRC*) — Part D/ Section 4

Nominal Speed — Part D/Section 5B

Nominal Thickness — Part C/Section 8

Nominal Travel Time/Theoretical Time of Travel — Part D/Section 5A

Non-Combustible Material — Part C/Section 8

Non-Return Valve — Part D/Section 5A

Non-Selective Collective Automatic Operation — Part D/Section 5A

Non-transit Occupancy — Part F

Normal Operation — Part D/Section 5A

Normal Stress — Part C/Section 6A

Normalized Impact Sound Pressure Level — Part D/Section 4

North and South Points — Part D/Section 1

Notice of Completion — Part A

O

Occupancy or Use Group — Part A, Part F, Part C/Section 1

Occupant Load — Part F

Occupant Load Factor (*OLF*) — Part F

Occupier — Part A

Occupational Exposure Limit (*OEL*) — Part D/ Section 3

Octave Band — Part D/Section 4

Offset — Part C/Section 2, Part E/Section 1, Part E/Section 2

Oil Buffer Stroke — Part D/Section 5A

One Pipe-Partially Ventilated System — Part E/ Section 2

One Pipe-Fully Ventilated System — Part E/Section 2

One-Way Restrictor — Part D/Section 5A

Open time — Part C/Section 7

Open Station — Part F

Operating Device — Part D/Section 5A

Operation — Part D/Section 5A

*Operational Construction/Installatio*n — Part A

Operational or Services Brake — Part D/Section 5B

Operating Panel — Part D/Section 5C

Operative Temperature — Part D/Section 3

Operator of a Facility — Part E/Section 3

Optical Distribution Frame — Part D/Section 6

Optical Fibre — Part D/Section 6

Ordinary moment resisting frame (*OMRF*) — Part C/Section 1

Orientation of Buildings — Part D/Section 1

Organic Coated Glass — Part C/Section 8

Origin of an Electrical Installation — Part D/ Section 2

Outdoor Artificial Lighting — Part D/Section 1

Outer Diameter — Part C/Section 3B

Out-of-Plane Deformation — Part C/Section 7

Outgoing Traffic — Part D/Section 5A

Outside Location – Part C/Section 3A, Part C/ Section 3B

Outside Plant (OSP) — Part D/Section 6

Overcurrent — Part D/Section 2

Overload Current (*of a Circuit*) — Part D/Section 2

Over Speed Governor — Part D/Section 5A

Overhead Beams — Part D/Section 5A

Owner of the Installation — Part D/Section 5A and 5B

Ozone Depletion Potential (*ODP*) — Part D/Section 3

P

Pane — Part C/Section 8

Panel — Part C/Section 8

Panel Wall — Part C/Section 4

Parking Equipment Attendant — Part D/Section 5C

Parking Unit — Part D/Section 5C

Partial Safety Factor — Part C/Section 6A and 6B

Partial Safety Factor for Loads — Part C/Section 6A and 6B

Partially Separate System — Part E/Section 2

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Partition — Part C/Section 8

Partly Framed or Unframed Glazing — Part C/ Section 8

Passenger — Part D/Section 5A

Passenger Arrival Rate — Part D/Section 5A

Passenger Average Waiting Time (*AWT*) — Part D/Section 5A

Passenger Lift — Part D/Section 5A

Passive Cooling — Part D/Section 3

Passive Fire Protection — Part F

Passive Heating — Part D/Section 3

Passive Solar Gain — Part C/Section 8

Patterned Glass — Part C/Section 8

Pawl Device — Part D/Section 5A

Peak Ground Acceleration — Part C/Section 1

Peak Gust — Part C/Section 1

Peak to Peak Vibration Levels — Part D/Section 5A

Peat — Part C/Section 2

Plenum — Part F

Pelletization — Part E/Section 3

Percentile Level — Part D/Section 4

Performance Based Design — Part F

Performance Time/Door-to-Door Time — Part D/Section 5A

Perimeter Fire Barrier — Part C/Section 8

Period of Structural Adequacy under Fire — Part C/Section 6A

Period of Supply — Part E/Section 1

Peripheral Field — Part D/Section 1

Permanent Load — Part C/Section 2

Permissible Stress — Part C/Section 3A, Part C/Section 3B, Part C/Section 6A

Pessral — Part D/Section 5A

PEN Conductor — Part D/Section 2

Phase Conductor — Part D/Section 2

Pier — Part C/Section 4

Pile-Enhanced Raft — Part C/Section 2

Pile Foundation — Part C/Section 2

Pile-Pile Interaction — Part C/Section 2

Pile-Raft Interaction — Part C/Section 2

Pile-Raft Coefficient or Load Sharing Ratio (α_{CPRF}) — Part C/Section 2

Pile Spacing — Part C/Section 2

Partition Wall — Part C/Section 4

Pilot — Part E/Section 4

Pink Noise — Part D/Section 4

Pipe System — Part E/Section 2

Pipe Work — Part E/Section 1

Pit — Part D/Section 5B

Pitch — Part C/Section 6A and 6B

Pitch Pocket — Part C/Section 3A

Place of Comparative Safety — Part F

Plastic Collapse — Part C/Section 6A

Plastic Design — Part C/Section 6A

Plastic Hinge — Part C/Section 6A

Plastic Moment — Part C/Section 6A

Plastic Section — Part C/Section 6A

Plenum — Part D/Section 3

Plug — Part D/Section 2

Plumbing — Part E/Section 1, Part E/Section 2

Plumbing System — Part E/Section 1, Part E/ Section 2

Pocket Type Reinforced Masonry — Part C/Section 4

Podium — Part F

Point of Safety — Part F

Point (*in Wiring*) — Part D/Section 2

Poisson's Ratio — Part C/Section 6A

Pool — Part E/Section 1

Pool Depth — Part E/Section 1

Population — Part D/Section 5A

Portable Equipment — Part D/Section 2

Position and/or Direction Indicator — Part D/ Section 5A

Positive Drive Lift — Part D/Section 5A

Positive Ventilation — Part D/Section 1, Part D/Section 3

Potable Water — Part E/Section 1

Precast Concrete Piles in Prebored Holes — Part C/ Section 2

Precast Driven Pile — Part C/Section 2

Printable Concrete — Part C/Section 7

Prefabricate — Part C/Section 7

Prefabricated Building — Part C/Section 7

Preliminary Operation — Part D/Section 5A

Preloading — Part C/Section 2

Premises — Part E/Section 1, Part E/Section 2

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Pressure Coefficient — Part C/Section 1
Pressure Balancing Valve — Part E/Section 1
Pressure Regulator — Part E/Section 4
Pressure Relief Valve — Part D/Section 5A
Pressurization — Part F
Pressurization Level — Part F
Pressurization System — Part F
Principal Rafter — Part C/Section 3B
Principal Occupancy or Use Group — Part F
Printability — Part C/Section 7
Print Plane — Part C/Section 7
Prism — Part C/Section 4
Processing — Part E/Section 3
Programmable Electronic System in Safety Related Applications for Lifts (*PESSRAL*) — Part D/ Section 5A
Progressive Safety Gear — Part D/Section 5A
Progressive Horizontal Evacuation — Part F
Proof Checking/Peer Reviewing — Part A
Proof Stress — Part C/Section 6A
Proof Testing — Part C/Section 6A
Propagation Delay — Part D/Section 6
Propeller Fan — Part D/Section 1
Prospective Fault Current (I_{pf}) — Part D/Section 2
Protection, Ingress — Part D/Section 2
Protection, Mechanical Impact — Part D/Section 2
Protective Coating(*s*) *Faults* — Part C/Section 8
Protective Conductor — Part D/Section 2
Protective Conductor Current — Part D/Section 2
Protective Earthing — Part D/Section 2
Protective Separation — Part D/Section 2
Prototype Testing — Part C/Section 6A
Prying Force — Part C/Section 6A
Psychrometric Chart — Part D/Section 3
Public Mobile Network — Part D/Section 6
Public Use — Part D/Section 5C
Public Way — Part F
Pumpability — Part C/Section 7
Puff Ventilation — Part E/Section 2
Pure Tone — Part D/Section 4
Pulley Room — Part D/Section 5A
Pulley Space — Part D/Section 5A
Purge — Part E/Section 4
Purlins — Part C/Section 3B
Puzzle Parking System — Part D/Section 5C

Q

Qualified Installing Agency — Part E/Section 4
Quetta Bond Reinforced Masonry — Part C/Section 4

R

Rack and Pinion Lift — Part D/Section 5A
Radio Frequency — Part D/Section 6
Raft-Enhanced Pile — Part C/Section 2
Raft-Pile Interaction — Part C/Section 2
Ramp — Part F
Rated Current — Part D/Section 2
Rated Impulse Withstand Voltage Level — Part D/ Section 2
Rated Load (*Lift*) — Part D/Section 5A, Part D/Section 5B
Rated Load — Part D/Section 5B
Rated Speed (*Lift*) — Part D/Section 5A
Rating Level — Part D/Section 4
Rebate — Part C/Section 8
Recirculated Air — Part D/Section 3
Recycling — Part E/Section 3, Part C/Section 6A
Reflected Glare — Part D/Section 1
Responsible Sourcing — Part C/Section 6A
Reflective Coated Glass — Part C/Section 8
Reflective Silver Coating Faults — Part C/Section 8
Refrigerant — Part D/Section 3
Refuge Area — Part F
Refuge Area, Capacity — Part F
Refuge Space — Part D/Section 5A, Part F
Registered Building Professional Architect, Engineer, Structural Engineer, Geotechnical Engineer, Supervisor, Town Planner, Landscape Architect, Urban Designer — Part A
Registered Building Professional — Part A
Reinforced Brick Slab (*RB Slab*) — Part C/Section 4
Reinforced Brick Concrete Slab (*RBC Slab*) — Part C/Section 4
Reinforced Masonry — Part C/Section 4
Relative Humidity — Part D/Section 3
Re-levelling — Part D/Section 5A
Relief Vent — Part E/Section 2
Reinforced Brick Slab — Part C/ Sec 4

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Reinforced Brick Concrete Slab — Part C/ Sec 4

Retaining Device — Part D/Section 5A

Rescue Operations — Part D/Section 5A

Residential Buildings — Part C/Section 1

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Residual Current — Part D/Section 2

Residual Current Device — Part D/Section 2

Residual Current Operated Circuit Breaker — Part D/Section 2

Residual Current Operated Circuit Breaker with Integral Overcurrent Protection (*RCBO*) — Part D/Section 2

Residual Current Operated Circuit Breaker without Integral Overcurrent Protection (*RCCB*) — Part D/Section 2

Residual Head — Part E/Section 1

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Restrictor — Part D/Section 5A

Retiring Cam/Coupler — Part D/Section 5A

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Return Air — Part D/Section 3

Reuse — Part C/Section 6A

Reveal — Part D/Section 1

Reverberation Time — Part D/Section 4

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Rigid Shear Connectors — Part C/Section 6B

Rise (*Travel*) — Part D/Section 5B

Riser — Part E/Section 4

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Roping Multiple — Part D/Section 5A

Rotation — Part C/Section 6A

Round Trip Time (*RTT*) — Part D/Section 5A

Routine Test — Part C/Section 2

Round Trip Time — Part D/Section 5A

Rupture Valve — Part D/Section 5A

S

S-N Curve — Part C/Section 6A and 6B

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Safe Load — Part C/Section 2

Safety Circuit — Part D/Section 5B

Safety Devices — Part D/Section 5B

Safety Gear — Part D/Section 5A, Part D/Section 5A, Part D/Section 5C

Safety Glass — Part C/Section 8

Safety Component — Part D/Section 5A

Safety Circuit — Part D/Section 5A

Safety Integrity Level — Part D/Section 5A

Safety Rope — Part D/Section 5A

Safety Switch — Part D/Section 5C

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Sand, Coarse — Part C/Section 2

Sand, Fine — Part C/Section 2

Sand, Medium — Part C/Section 2

Sandwich Concrete Panels — Part C/Section 7A

Sandwich, Structural — Part C/Section 3A

Sanitary Appliances — Part E/Section 2

Sap Stain — Part C/Section 3A

Sapwood — Part C/Section 3A

Secondary Member — Part C/Section 6A

Seismic Mode — Part D/Section 5A

Seismic Stand — By Mode — Part D/Section 5A

Seismic Trigger Level — Part D/Section 5A

Semi-compact Section — Part C/Section 6A

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Seismic Zone and Seismic Coefficient — Part C/Section 4

Selective Collective Automatic Operation — Part D/Section 5A

Self-Compacting Concrete — Part C/Section 7

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Sensible Cooling — Part D/Section 3

Sensible Heat — Part D/Section 3

Separate System — Part E/Section 2

Series — Part D/Section 5C

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Server — Part D/Section 6
Service — Part D/Section 2
Service Floor — Part F
Service Lift — Part D/Section 5A
Service Limit — Part C/Section 6B
Serviceability Loads — Part C/Section 6B
Service Pipe — Part E/Section 1, Part E/Section 4
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Serviceability Limit State — Part C/Section 6A
Set-Back Line — Part A
Settlement Reduction Ratio (*SRR*) — Part C/Section 2
Sewer — Part E/Section 2
Shade Factor — Part D/Section 3
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Shading Coefficient — Part C/Section 8
Shake — Part C/Section 3A
Shallow Area — Part E/Section 1
Shallow Foundation — Part C/Section 2
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Shear Lag — Part C/Section 6A
Shear Stress — Part C/Section 6A
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Soil Densification — Part C/Section 2
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*Soil Reinforc*ement — Part C/Section 2
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Soil, Highly Organic and Other Miscellaneous Soil Materials — Part C/Section 2
Solar Energy Absorption — Part C/Section 8

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Solar Energy Transmittance (*Direct*) — Part C/ Section 8
Solar Heat Gain Coefficient (*SHGC*) — Part C/Section 8
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Sound Source — Part D/Section 4
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Structural Analysis — Part C/Section 6A, Part C/Section 6B

Structural Wall System — Part C/Section 5

Structural Element — Part C/Section 3A

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Veneered Wall — Part C/Section 4

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