NATIONAL BUILDING CODE OF INDIA 2005

Group 2

NATIONAL BUILDING CODE OF INDIA 2005

GROUP 2

PART 0	INTEGRATED APPROACH — PREREQUISITE FOR APPLYING PROVISIONS OF THE CODE		
PART 6	STRUCTURAL DESIGN		
	Section 1	Loads, Forces and Effects	
	Section 2	Soils and Foundations	
	Section 3	Timber and Bamboo	
		3A	Timber
		3B	Bamboo
	Section 4	Masonry	
	Section 5	Concrete	
		5A	Plain and Reinforced Concrete
		5B	Prestressed Concrete
	Section 6	Steel	
	Section 7	Prefabrication, Systems Building and Mixed/Composite Construction	
		7A	Prefabricated Concrete
		7B	Systems Building and Mixed/Composite Construction

BUREAU OF INDIAN STANDARDS

SP 7 (Group 2) : 2005

FIRST PUBLISHED 1970
FIRST REVISION 1983
SECOND REVISION 2005

ICS 0.120; 91.040.01
ISBN 81-7061-026-5

PRICE Rs. 4390.00

PUBLISHED BY BUREAU OF INDIAN STANDARDS, MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG, NEW DELHI 110 002, PRINTED AT SUNSHINE PROCESS, C-105/5, NARAINA INDUSTRIAL AREA, PHASE I, NEW DELHI 110 028 (INDIA).

BIS Website: www.bis.org.in

FOREWORD

Construction programmes are interwoven in a large measure in all sectors of development, be it housing, transport, industry, irrigation, power, agriculture, education or health. Construction, both public and private, accounts for about fifty percent of the total outlay in any Five Year Plan. Half of the total money spent on construction activities is spent on buildings for residential, industrial, commercial, administrative, education, medical, municipal and entertainment uses. It is estimated that about half of the total outlay on buildings would be on housing. It is imperative that for such a large national investment, optimum returns are assured and wastage in construction is avoided.

Soon after the Third Plan, the Planning Commission 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 found in the 'Report on Economies in Construction Costs' published in 1968.

One of the facets of building construction, namely, controlling and regulating buildings through municipal byelaws and departmental handbooks received the attention of the Panel and a study of these regulatory practices revealed that some of the prevailing methods of construction were outmoded; some designs were overburdened with safety factors and there were other design criteria which, in the light of newer techniques and methodologies, could be rationalized; and building byelaws and regulations of municipal bodies which largely regulate the building activity in the country wherever they exist, were outdated. They did not cater to the use of new building materials and the latest developments in building designs and construction techniques. It also became clear that these codes and byelaws lacked uniformity and they were more often than not 'specification oriented' and not 'performance oriented'.

These studies resulted in a recommendation that a National Building Code 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 preparation of the National Building Code. For fulfilling this task a Guiding Committee for the preparation of the Code was set up by the Civil Engineering Division Council of the Indian Standards Institution in 1967. This Committee, in turn, set up 18 specialist panels to prepare the various parts of the Code. The Guiding Committee and its panels were constituted with architects, planners, materials experts, structural, construction, electrical illumination, air conditioning, acoustics and public health engineers and town planners. These experts were drawn from the Central and State Governments, local bodies, professional institutions and private agencies. The first version of the Code was published in 1970.

After the National Building Code of India was published in 1970, a vigorous implementation drive was launched by the Indian Standards Institution to propagate the contents and use of the Code among all concerned in the field of planning, designing and construction activities. For this, State-wise Implementation Conferences were organized with the participation of the leading engineers, architects, town planners, administrators, building material manufacturers, building and plumbing services installation agencies, contractors, etc.

These Conferences were useful in getting across the contents of the Code to the interests concerned. These Conferences had also helped in the establishment of Action Committees to look into the actual implementation work carried out by the construction departments, local bodies and other agencies in different States. The main actions taken by the Action Committees were to revise and modernize their existing regulatory media, such as, specifications, handbooks, manuals, etc, as well as building byelaws of local bodies like municipalities at city and town levels, zilla parishads, panchayats and development authorities, so as to bring them in line with the provisions contained in the National Building Code of India. In this process, the Indian Standards Institution rendered considerable support in redrafting process.

Since the publication in 1970 version of the National Building Code of India, a large number of comments and useful suggestions for modifications and additions to different parts and sections of the Code were received as a result of use of the Code by all concerned, and revision work of building byelaws of some States. Based on the comments and suggestion received the National Building Code of India 1970 was revised in 1983.

Some of the important changes in 1983 version included : addition of development control rules, requirements for greenbelts and landscaping including norms for plantation of shrubs and trees, special requirements for low income housing; fire safety regulations for high rise buildings; revision of structural design section based on new and revised codes, such as Concrete Codes (plain and reinforced concrete and prestressed concrete), Earthquake Code, Masonry Code; addition of outside design conditions for important cities in the country, requirements relating to noise and vibration, air filter, automatic control, energy conservation for air conditioning; and guidance on the design of water supply system for multi-storeyed buildings.

The National Building Code of India is a single document in which, like a network, the information contained in various Indian Standards is woven into a pattern of continuity and cogency with the interdependent requirements of Sections carefully analyzed and fitted in to make the whole document a cogent continuous volume. A continuous thread of 'preplanning' is woven which, in itself, contributes considerably to the economies in construction particularly in building and plumbing services.

The Code contains regulations which can be immediately adopted or enacted for use by various departments, municipal administrations and public bodies. It lays down a set of minimum provisions designed to protect the safety of the public with regard to structural sufficiency, fire hazards and health aspects of buildings; so long as these basic requirements are met, the choice of materials and methods of design and construction is left to the ingenuity of the building professionals. The Code also covers aspects of administrative regulations, development control rules and general building requirements; fire protection requirements; stipulations regarding materials and structural design; rules for design of electrical installations, lighting, air conditioning and lifts; regulation for ventilation, acoustics and plumbing services, such as, water supply, drainage, sanitation and gas supply; measures to ensure safety of workers and public during construction; and rules for erection of signs and outdoor display structures.

Some other important points covered by the Code include 'industrialized systems of building' and 'architectural control'. The increase in population in the years to come will have a serious impact on the housing problem. It has been estimated that the urban population of India will continue to increase with such pace as to maintain the pressure on demand of accommodation for them. Speed of construction is thus of an utmost importance and special consideration has to be given to industrialized systems of building. With increased building activity, it is also essential that there should be some architectural control in the development of our cities and towns if creation of ugliness and slum-like conditions in our urban areas is to be avoided.

Since the publication of 1983 version of National Building Code of India, the construction industry has gone through major technological advancement. In the last two decades, substantial expertise has been gained in the areas of building planning, designing and construction. Also, lot of developments have taken places in the techno-legal regime and techno-financial regime, apart from the enormous experience gained in dealing with natural calamities like super cyclones and earthquakes faced by the country. Further, since the last revision in 1983 based on the changes effected in the Steel Code, Masonry Code and Loading Code as also in order to update the fire protection requirements, three amendments were brought out to the 1983 version of the Code. Considering these, it was decided to take up a comprehensive revision of the National Building Code of India.

The changes incorporated in the present Code, which is second revision of the Code, have been specified in the Foreword to each Part/Section of the Code. Some of the important changes are:

a) A new Part 0 'Integrated Approach — Prerequisite for Applying the Provisions of the Code' emphasizing on multi-disciplinary team approach for successfully accomplishing building/development project, has been incorporated.
b) New chapters on significant areas like structural design using bamboo, mixed/composite construction and landscaping have been added.
c) Number of provisions relating to reform in administration of the Code as also assigning duties and responsibilities to all concerned professionals, have been incorporated/modified. Also detailed provisions/ performance to ensure structural sufficiency of buildings, have been prescribed so as to facilitate implementation of the related requirements to help safely face the challenges during natural disasters like earthquake.
d) Planning norms and requirements for hilly areas and rural habitat planning, apart from detailed planning norms for large number of amenities have been incorporated.
e) Fire safety aspects have been distinctly categorized into fire prevention, life safety and fire protection

giving detailed treatment to each based on current international developments and latest practices followed in the country.

f) Aspects like energy conservation and sustainable development have been consistently dealt with in various parts and sections through appropriate design, usage and practices with regard to building materials, construction technologies and building and plumbing services. Renewable resources like bamboo and practices like rain water harvesting have been given their due place.

g) The latest revised earthquake code, IS 1893 (Part 1) : 2002 'Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings', has been incorporated, due implementation of the provisions of which in applicable seismic zone of the country, needs to be duly adhered to by the Authorities.

The Code now published is the third version representing the present state of knowledge on various aspects of building construction. The process of preparation of the 2005 version of the Code had thrown up a number of problems; some of them were answered fully and some partially. Therefore, a continuous programme will go on by which additional knowledge that is gained through technological evolution, users' views over a period of time pinpointing areas of clarification and coverage and results of research in the field, would be incorporated in to the Code from time to time to make it a living document. It is, therefore, proposed to bring out changes to the Code periodically.

The provisions of this Code are intended to serve as a model for adoption by Public Works Departments and other government construction departments, local bodies and other construction agencies. Existing PWD codes, municipal byelaws and other regulatory media could either be replaced by the National Building Code of India or suitably modified to cater to local requirements in accordance with the provisions of the Code. Any difficulties encountered in adoption of the Code could be brought to the notice of the Sectional Committee for corrective action.

This publication forms part of the National Building Code of India 2005 and contains the following Parts:

PART 0 INTEGRATED APPROACH — PREREQUISITE FOR APPLYING PROVISIONS OF THE CODE

PART 6 STRUCTURAL DESIGN

Section 1 Loads, Forces and Effects
Section 2 Soils and Foundations
Section 3 Timber and Bamboo
3A Timber
3B Bamboo
Section 4 Masonry
Section 5 Concrete
5A Plain and Reinforced Concrete
5B Prestressed Concrete
Section 6 Steel
Section 7 Prefabrication, Systems Building and Mixed/Composite Construction
7A Prefabricated Concrete
7B Systems Building and Mixed/Composite Construction

The provisions contained in this publication will essentially serve the concerned professionals in dealing with the structural design of buildings using various materials and technology streams.

National Building Code Sectional Committee, CED 46

Chairman
Dr H. C. Visvesvaraya
'Chandrika', at 15th Cross, 63-64 East Park Road
Malleswaram, Bangalore 560 003

Vice-Chairman
Shri V. Suresh
P-233/3, Officers Enclave,
Air Force Station, Rajokari, New Delhi 110 038

Organization	*Representative(s)*
Ahmedabad Municipal Corporation, Ahmedabad	Shri Vatsal S. Patel Shri Jagdish A. Patel (*Alternate*)
Bangalore Mahanagara Palike, Bangalore	Shri M. R. Sreenivasa Murthy Shri R. Ramegowda (*Alternate* I) Shri N. Krishna (*Alternate* II)
Builders Association of India, Mumbai	Shri B. G. Ahuja
Building Materials and Technology Promotion Council, New Delhi	Shri T. N. Gupta & Shri D. B. N. Rao
Bureau of Energy Efficiency (Ministry of Power), New Delhi	Representative
Central Building Research Institute (CSIR), Roorkee	Shri V. K. Mathur Shri B. S. Gupta (*Alternate*)
Central Public Health and Environmental Engineering Organisation (Ministry of Urban Development and Poverty Alleviation), New Delhi	Shri B. B. Uppal Shri V. K. Chaurasia (*Alternate*)
Central Public Works Department (Central Designs Organization), New Delhi	Chief Engineer (Designs) Superintending Engineer (S & S) (*Alternate*)
Central Public Works Department (Electrical Department), New Delhi	Chief Engineer (Electrical) I
Centre for Disaster Mitigation and Management, Anna University, Chennai	Director
Chennai Metropolitan Development Authority, Chennai	Member Secretary Shri N. V. Rakhunath (*Alternate*)
Construction Industry Development Council, New Delhi	Shri P. R. Swarup Shri Anil Chadha (*Alternate*)
Council of Architecture, New Delhi	Shri Premendra Raj Mehta Shri Sudhir Vohra (*Alternate*)
Delhi Development Authority, New Delhi	Engineer Member Chief Engineer (HQ) (*Alternate*)
Delhi Fire Service, Government of National Capital Territory of Delhi, Delhi	Shri R. C. Sharma
Department of Science and Technology (Ministry of Science and Technology), New Delhi	Shri V. Rao Alyagari
Directorate General of Employment and Training, New Delhi	Shri Ashwani Kumar
Engineer-in-Chief's Branch, Army Headquarters, New Delhi	Brig S. K. Sharma Shri D. K. Dinker (*Alternate*)
Forest Research Institute (Indian Council for Forestry Research and Education), Dehra Dun	Director General Director (*Alternate*)
Housing and Urban Development Corporation Ltd, New Delhi	Chairman & Managing Director Shri R. K. Safaya (*Alternate*)
Indian Geotechnical Society, New Delhi	Shri D. B. Mahajan Dr M. D. Desai (*Alternate*)

Organization	*Representative(s)*
Indian Institute of Technology (Centre for Energy Studies), New Delhi	PROF N. K. BANSAL
Indian Roads Congress, New Delhi	CHIEF ENGINEER (DESIGN), CPWD SUPERINTENDING ENGINEER (DESIGN), CPWD (*Alternate*)
Institute of Town Planners, India, New Delhi	DR S. K. KULSHRESTHA
Institution of Fire Engineers (India), New Delhi	PRESIDENT GENERAL SECRETARY (*Alternate*)
Ministry of Home Affairs, New Delhi	FIRE ADVISOR
Ministry of Home Affairs (Disaster Management Division), New Delhi	SHRI M. P. SAJNANI SHRI S. K. SWAMI (*Alternate*)
Ministry of Non-Conventional Energy Sources, New Delhi	DR T. C. TRIPATHI
Ministry of Road Transport and Highways, New Delhi	SHRI S. B. BASU SHRI P. HALDER (*Alternate*)
Municipal Corporation of Greater Mumbai, Mumbai	DIRECTOR (ENGG SERVICES & PROJECTS) CITY ENGINEER (*Alternate*)
National Buildings Construction Corporation, New Delhi	SHRI B. PRASAD SHRI N. P. AGARWAL (*Alternate*)
National Council for Cement and Building Materials, Ballabgarh	SHRI SHIBAN RAINA DR ANIL KUMAR (*Alternate*)
National Design and Research Forum, The Institution of Engineers (India), Bangalore	PROF R. NARAYANA IYENGAR SHRI B. SURESH (*Alternate*)
National Environmental Engineering Research Institute (CSIR), Nagpur	DR ARINDAM GHOSH DR V. P. DESHPANDE (*Alternate*)
North Eastern Council, Shillong	SHRI P. K. DEB
Public Works Department (Roads and Buildings), Gandhinagar	SHRI V. P. JAMDAR SHRI M. S. JALLUNDHWALA (*Alternate*)
Research, Designs and Standards Organization (Ministry of Railways), Lucknow	SHRI R. K. GUPTA SHRI J. P. DAS (*Alternate*)
School of Planning and Architecture, New Delhi	DIRECTOR
Structural Engineering Research Centre (CSIR), Chennai	SHRI C. V. VAIDYANATHAN SHRI K. MANI (*Alternate*)
Suri and Suri Consulting Acoustical Engineers, New Delhi	SHRI GAUTAM SURI
The Energy and Resources Institute, New Delhi	MS MILI MAJUMDAR MS VIDISHA SALUNKE-PALSULE (*Alternate*)
The Indian Institute of Architects, New Delhi	SHRI BALBIR VERMA SHRI ABHIJIT RAY (*Alternate*)
The Institution of Engineers (India), Kolkata	PROF G. P. LAL SHRI O. P. GOEL (*Alternate*)
The Institution of Surveyors, New Delhi	SHRI K. S. KHARB SHRI R. K. BHALLA (*Alternate*)
Town and Country Planning Organization, New Delhi	SHRI K. T. GURUMUKHI SHRI J. B. KSHIRSAGAR (*Alternate*)
U.P. Housing and Development Board, Lucknow	SHRI HARI GOPAL
Unitech Ltd, Gurgaon	SHRI SUSHIL SHARMA SHRI SHAHID MAHMOOD (*Alternate*)
In personal capacity (*5, Sunder Nagar, New Delhi 110 003*)	DR J. R. BHALLA
BIS Directorate General	SHRI S. K. JAIN, Director & Head (Civil Engineering) [Representing Director General (*Ex-officio Member*)]

Member Secretary
SHRI SANJAY PANT
Joint Director (Civil Engineering), BIS

Special Panel for Guiding and Co-ordinating the Revision of National Building Code of India, CED 46:SP

Organization	*Representative(s)*
In personal capacity (*P-233/3, Officers Enclave, Air Force Station, Rajokari, New Delhi 110 038*)	SHRI V. SURESH (***Convener***)
Building Materials and Technology Promotion Council, New Delhi	SHRI T. N. GUPTA
Central Building Research Institute (CSIR), Roorkee	SHRI V. K. MATHUR
Central Public Works Department, New Delhi	SHRI H. S. DOGRA
Council of Architecture, New Delhi	SHRI PREMENDRA RAJ MEHTA
Engineer-in-Chief's Branch, Army Headquarters, New Delhi	LT-GEN HARI UNIYAL
The Institution of Engineers (India), Kolkata	PROF G. P. LAL SHRI O. P. GOEL (*Alternate*)
Bureau of Indian Standards, New Delhi	SHRI SANJAY PANT

Ad-hoc Group for Part 0 of NBC, CED 46:AG

Organization	*Representative(s)*
In personal capacity (*'Chandrika', at 15th Cross, 63-64, East Park Road, Malleswaram, Bangalore 560 003*)	DR H. C. VISVESVARAYA (***Convener***)
Council of Architecture, New Delhi	SHRI PREMENDRA RAJ MEHTA
In personal capacity (*P-233/3, Officers Enclave, Air Force Station, Rajokari, New Delhi 110 038*)	SHRI V. SURESH
In personal capacity (*A-39/B, DDA Flats, Munirka, New Delhi 110 067*)	SHRI P. B. VIJAY
In personal capacity (*EA-345, Maya Enclave, New Delhi 110 064*)	SHRI J. N. BHAVANI PRASAD

Panel for Loads, Forces and Effects, CED 46:P4

Organization	*Representative(s)*
National Council for Cement and Building Materials, Ballabgarh	DR ANIL KUMAR (***Convener***)
Building Materials and Technology Promotion Council, New Delhi	SHRI T. N. GUPTA SHRI I. S. SIDHU (*Alternate*)
Central Building Research Institute, Roorkee	SHRI B. S. GUPTA SHRI A. K. MITTAL (*Alternate*)
Central Public Works Department, New Delhi	SHRI N. M. D. JAIN SHRI ABHAY SINHA (*Alternate*)
Centre for Disaster Mitigation and Management, Anna University, Chennai	DR R. K. BHANDARI
Mahendra Raj Consultants Pvt Ltd, New Delhi	SHRI MAHENDRA RAJ SHRI J. GHOSE (*Alternate*)
Structural Engineering Research Centre (CSIR), Chennai	SHRI C. V. VAIDYANATHAN SHRI K. MANI (*Alternate*)
The Institution of Engineers (India), Kolkata	SHRI P. P. DHARWADKAR
In personal capacity (*Professor of Bridge Engineering, Railway Chair, Department of Civil Engineering, Indian Institute of Technology, Roorkee 247 667*)	DR PREM KRISHNA
In personal capacity (*Emeritus Scientist, Structural Engineering Research Centre, Madras CSIR Campus, Taramani, Chennai 600 113*)	DR T. V. S. R. APPA RAO
In personal capacity (*P-233/3, Officers Enclave, Air Force Station, Rajokari, New Delhi 110 038*)	SHRI V. SURESH

Panel for Soils and Foundations, CED 46:P5

Organization	*Representative(s)*
Centre for Disaster Mitigation and Management, Anna University, Chennai	Dr R. K. Bhandari (***Convener*** before 19 September 2003)
Central Building Research Institute (CSIR), Roorkee	Shri Chandra Prakash (***Convener*** since 19 September 2003) Dr Surendra Kumar (*Alternate*)
Afcons Infrastructure Limited, Mumbai	Shri S. B. Joshi Shri D. G. Bhagwat (*Alternate*)
Central Public Works Department, New Delhi	Shri Bhagwan Singh Shri R. K. Singhal (*Alternate*)
Delhi Development Authority, New Delhi	Shri S. P. Rustogi Shri J. M. Joshi (*Alternate*)
Engineer-in-Chief's Branch, Army Headquarters, New Delhi	Col R. N. Malhotra Col N. B. Saxena (*Alternate*)
Indian Geotechnical Society, New Delhi	Maj Gen S. N. Mukerjee Shri Sanjay Gupta (*Alternate*)
National Council for Cement and Building Materials, Ballabgarh	Dr Anil Kumar Shri H. K. Julka (*Alternate*)
The Institution of Engineers (India), Kolkata	Prof Janardan Jha

Panel for Timber, CED 46:P6

Organization	*Representative(s)*
In personal capacity (*Pratap Nursery Lane, Near Gurdwara, Panditwari, Dehra Dun 248007*)	Shri K. S. Pruthi (***Convener***)
Bamboo Society of India, Bangalore	Shri A. C. Lakshmanan Dr K. A. Kushalappa (*Alternate*)
Building Materials and Technology Promotion Council, New Delhi	Shri T. N. Gupta Shri Rajesh Malik (*Alternate*)
Central Building Research Institute (CSIR), Roorkee	Shri S. K. Mittal Shri B. S. Rawat (*Alternate*)
Central Public Works Department, New Delhi	Shrimati P. Verma Shri G. C. Khattar (*Alternate*)
Engineer-in-Chief's Branch, Army Headquarters, New Delhi	Shri A. K. Singh Shri P. K. Gupta (*Alternate*)
Forest Research Institute (Indian Council for Forestry Research and Education), Dehra Dun	Shri B. K. Bhatia
Housing and Urban Development Corporation Ltd, New Delhi	Chairman and Managing Director Shri S. K. Taneja (*Alternate*)
Indian Plywood Industries Research and Training Institute, Bangalore	Shri K. Shyamasundar Shri H. Guruva Reddy (*Alternate*)
North Eastern Council, Shillong	Shri P. K. Deb
The Institution of Engineers (India), Kolkata	Shri Krishna Kumar
In personal capacity [*No. 179 (710), 24th B-Cross, 3rd Block, Jayanagar, Bangalore 560 011*]	Dr H. N. Jagadeesh
In personal capacity (*103/II, Vasant Vihar, P. O. New Forest, Dehra Dun 248 006*)	Shri S. S. Rajput

Panel for Masonry, CED 46:P7

Organization	*Representative(s)*
Delhi Tourism and Transportation Development Corporation, New Delhi	SHRI JOSE KURIAN (***Convener***)
Building Materials and Technology Promotion Council, New Delhi	SHRI T. N. GUPTA SHRI PANKAJ GUPTA (*Alternate*)
Central Building Research Institute (CSIR), Roorkee	SHRI A. K. MITTAL SHRI SHAILESH KUMAR (*Alternate*)
Central Public Works Department, New Delhi	DR A. K. MITTAL SHRI NEERAJ MISHRA (*Alternate* I) SHRI A. K. JHA (*Alternate* II)
Delhi Development Authority, New Delhi	SHRI S. P. RUSTOGI SHRI J. M. JOSHI (*Alternate*)
Engineer-in-Chief's Branch, Army Headquarters, New Delhi	SHRI D. R. KURDIYA SHRI SUBODH KUMAR (*Alternate*)
Indian Institute of Science (Centre for Astra), Bangalore	DR B. V. VENKATARAMA REDDY DR K. S. NANJUNDA RAO (*Alternate*)
Indian Institute of Technology, Kanpur	DR DURGESH C. RAI DR C. V. R. MURTY (*Alternate* I) DR SUDHIR K. JAIN (*Alternate* II)
Indian Institute of Technology, New Delhi	DR S. N. SINHA
Public Works Department, Government of Maharashtra, Mumbai	SHRI P. K. NINAVE
Structural Engineering Research Centre (CSIR), Chennai	SHRI R. JAYARAMAN SHRI A. CHELLAPPAN (*Alternate*)
The Institution of Engineers (India), Kolkata	SHRI S. L. GARG

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

Organization	*Representative(s)*
In personal capacity (*35, Park Avenue, Annamma, Naicker Street, Kuniamuthur, Coimbatore 641 008*)	DR C. RAJKUMAR (***Convener***)
Central Building Research Institute (CSIR), Roorkee	DR B. S. GUPTA DR B. K. RAO (*Alternate* I) DR AWADESH KUMAR (*Alternate* II)
Central Public Works Department, New Delhi	SHRI N. M. D. JAIN SHRI ABHAY SINHA (*Alternate*)
Delhi Tourism and Transportation Development Corporation, New Delhi	SHRI JOSE KURIAN
Gammon India Ltd, Mumbai	SHRI S. A. REDDI
Hindustan Prefab Limited, New Delhi	SHRI HAZARI LAL SHRI M. KUNDU (*Alternate*)
Larsen and Toubro Ltd, ECC Construction Group, Chennai	SHRI K. P. RAGHAVAN SHRI S. KANAPPAN (*Alternate*)
Ministry of Road Transport and Highways, New Delhi	SHRI T. B. BANERJEE SHRI SATISH KUMAR (*Alternate*)
National Council for Cement and Building Materials, Ballabgarh	DR ANIL KUMAR SHRI H. K. JULKA (*Alternate*)
Research, Designs and Standards Organization (Ministry of Railways), Lucknow	SHRI R. K. GUPTA SHRI J. P. DAS (*Alternate* I) SHRI A. K. GUPTA (*Alternate* II)
Structural Engineering Research Centre (CSIR), Chennai	DR N. LAKSHMANAN SHRI H. G. SREENATH (*Alternate*)
Tandon Consultants, New Delhi	SHRI MAHESH TANDON
The Institution of Engineers (India), Kolkata	SHRI S. S. CHAKRABARTY

Panel for Steel, CED 46:P9

Organization	*Representative(s)*
MECON Ltd, Ranchi	Shri A. Basu (***Convener***)
Central Public Works Department, New Delhi	Chief Engineer Suprintending Engineer (P & A) (*Alternate*)
Engineer-in-Chief's Branch, Army Headquarters, New Delhi	Shri D. K. Dinker Col V. K. Tyagi (*Alternate*)
Indian Institute of Technology, Chennai	Dr V. Kalyanaraman
Institute for Steel Development and Growth, Kolkata	Dr T. K. Bandyopadhyay Shri Arijit Guha (*Alternate* I) Shri P. L. Rao (*Alternate* II)
Kalpataru Power Transmission Ltd, Gandhinagar	Shri M. C. Mehta Shri B. K. Satish (*Alternate*)
M. N. Dastur and Co Ltd, Kolkota	Shri Satyaki Sen Shri Tapan Kumar Bhaumik (*Alternate*)
Research, Designs and Standards Organization (Ministry of Railways), Lucknow	Shri R. K. Gupta Shri D. K. Singh (*Alternate*)
SPECO Engineering Pvt Ltd, New Delhi	Shri Onkar Singh
Structural Engineering Research Centre (CSIR), Chennai	Dr S. Seetharaman Shri S. Arul Jayachandran (*Alternate*)
The Institute of Engineers (India), Kolkata	Shri R. P. Gupta

Panel for Prefabrication and Systems Building, CED46:P10

Organization	*Representative(s)*
Larsen and Toubro Ltd, Chennai	Shri A. Ramakrishna (***Convener***) Shri K. V. Rangaswami (*Alternate*)
B. G. Shrike Construction Technology Pvt Ltd, Pune	Shri G. R. Bharitkar Shri R. P. Jakhalekar (*Alternate*)
Central Building Research Institute (CSIR), Roorkee	Shri B. N. Hira Shri D. K. Gautam (*Alternate*)
Central Public Works Department, New Delhi	Chief Engineer (NDZ-III) Shri A. K. Garg (*Alternate*)
Engineer-in-Chief's Branch, Army Headquarters, New Delhi	Col R. N. Malhotra Shri P. K. Gupta (*Alternate*)
Hindustan Prefab Limited, New Delhi	Shri Hazari Lal Shri M. Kundu (*Alternate*)
Institute for Steel Development and Growth, Kolkata	Dr T. K. Bandyopadhyay Shri Alok Baishya (*Alternate*)
Lloyd Insulations (India) Ltd, New Delhi	Shri Mohit Khanna Shri K. K. Mitra (*Alternate*)
National Council for Cement and Building Materials, Ballabgarh	Shri H. K. Julka Shri Satish Sharma (*Alternate*)
Shirish Patel and Associates Consultants Pvt Ltd, Mumbai	Shri Shirish B. Patel Shri P. H. Srinivasachar (*Alternate*)
Structural Engineering Research Centre (CSIR), Chennai	Shri H. G. Sreenath Shri R. Jayaraman (*Alternate*)

Organization	*Representative(s)*
System Building Technologists, New Delhi	SHRI G. B. SINGH
The Indian Institute of Architects, Mumbai	SHRI S. R. SIKKA
The Institution of Engineers (India), Kolkata	DR R. K. BHANDARI SHRI P. B. VIJAY (*Alternate*)

Member Secretary
SHRI SANJAY PANT
Joint Director (Civil Engineering), BIS

Joint Member Secretary
SHRI S. K. VERMA
Deputy Director (Civil Engineering), BIS

Important Explanatory Note for Users of Code

In this Code, where reference is made to 'accepted standards' in relation to material specification, testing or other related information or where reference is made to 'good practice' in relation to design, constructional procedures or other related information, the Indian Standards listed at the end of the concerned Parts/Sections may be used to the interpretation of these terms.

At the time of publication, the editions indicated in the above Indian Standards were valid. All standards are subject to revision and parties to agreements based on the Parts/Sections 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 each Part/Section, the number appearing in the first column indicates the number of the reference in that Part/Section. For example:

a) good practice [6-1(1)] refers to the standard given at serial number 1 of the list of standards given at the end of Section 1 of Part 6, that is 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*)'.
b) accepted standard [6-2(8)] refers to the standard given at serial number 8 of the list of standards given at the end of Section 2 of Part 6, that is IS 1888 : 1982 'Method of load tests on soils (*second revision*)'.
c) accepted standard [6-3A(5)] refers to the standard given at serial number 5 of the list of standards given at the end of Subsection 3A of Part 6, that is IS 3629 : 1986 'Specification for structural timber in building (*first revision*)'.
d) good practice [6-4(4)] refers to the standard given at serial number 4 of the list of standards given at the end of Section 4 of Part 6, that is IS 2212 : 1981 'Code of practice for brickwork (*first revision*)'.
e) good practice [6-5A(25)] refers to the standard given at serial number 25 of the list of standards given at the end of Section 5A of Part 6, that is IS 14687 : 1999 'Guidelines for falsework for concrete structure'.

INFORMATION FOR THE USERS

For the convenience of the users, the National Building Code of India 2005 is available as a comprehensive volume as well as in the following five groups, each incorporating the related Parts/Sections dealing with particular area of building activity:

Group 1	**For Development, Building Planning and Related Aspects**	Part 0:	Integrated Approach — Prerequisite for Applying Provisions of the Code
		Part 2:	Administration
		Part 3:	Development Control Rules and General Building Requirements
		Part 4:	Fire and Life Safety
		Part 5:	Building Materials
		Part 10:	Landscaping, Signs and Outdoor Display Structures Section 1 Landscape Planning and Design Section 2 Signs and Outdoor Display Structures
Group 2	**For Structural Design and Related Aspects**	Part 0:	Integrated Approach — Prerequisite for Applying Provisions of the Code
		Part 6:	Structural Design Section 1 Loads, Forces and Effects Section 2 Soils and Foundations Section 3 Timber and Bamboo 3A Timber 3B Bamboo Section 4 Masonry Section 5 Concrete 5A Plain and Reinforced Concrete 5B Prestressed Concrete Section 6 Steel Section 7 Prefabrication, Systems Building and Mixed/Composite Construction 7A Prefabricated Concrete 7B Systems Building and Mixed/ Composite Construction
Group 3	**For Construction Related Aspects including Safety**	Part 0:	Integrated Approach — Prerequisite for Applying Provisions of the Code
		Part 7:	Constructional Practices and Safety
Group 4	**For Aspects Relating to Building Services**	Part 0:	Integrated Approach — Prerequisite for Applying Provisions of the Code
		Part 8:	Building Services Section 1 Lighting and Ventilation Section 2 Electrical and Allied Installations Section 3 Air conditioning, Heating and Mechanical Ventilation Section 4 Acoustics, Sound Insulation and Noise Control Section 5 Installation of Lifts and Escalators
Group 5	**For Aspects Relating to Plumbing Services including Solid Waste Management**	Part 0:	Integrated Approach — Prerequisite for Applying Provisions of the Code
		Part 9:	Plumbing Services Section 1 Water Supply, Drainage and Sanitation (including Solid Waste Management) Section 2 Gas Supply

The information contained in different groups will essentially serve the concerned professionals dealing in the respective areas.

The National Building Code of India consists of the following Parts and Sections:

NATIONAL BUILDING CODE OF INDIA

PART 0 INTEGRATED APPROACH — PREREQUISITE FOR APPLYING PROVISIONS OF THE CODE

BUREAU OF INDIAN STANDARDS

CONTENTS

National Building Code Sectional Committee, CED 46

FOREWORD

In order to provide safe and healthy 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. In those days those buildings were conceptualized and built by master builders with high levels of artisan skills. 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. These perforce 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, etc in conceptualization, spatial planning, design and construction of buildings of various material and technology streams, with due regard to various services including operation, maintenance, repairs and rehabilitation aspects throughout the service life of the building.

This Code, besides prescribing the various provisions, also allows 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 Code 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, proneness to natural disasters, ecologically appropriate practices, use of eco-friendly materials, reduction of pollution, protection and improvement of local environment and also socio-economic considerations, towards the creation of sustainable human settlements.

This Part of the Code dealing with 'integrated approach' is being included for the first time. It gives an overall direction for practical applications of the provisions of different specialized aspects of spatial planning, designing and construction of buildings, creation of services, and proposes an integrated approach 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. The 'integrated approach' should not only take care of functional, aesthetic and safety aspects, but also the operational and maintenance requirements. Also, cost optimization has to be achieved through proper selection of materials, techniques, equipment installations, etc. Further, value engineering and appropriate management techniques should be applied to achieve the aim set forth for the purpose of construction of a building fully meeting the specified and implied needs of spatial functions, safety and durability aspects, life and health safety, comfort, services, etc in the building.

The aim of the 'integrated approach' is to get the maximum benefit from the building and its services in terms of quality, timely completion and cost-effectiveness. In the team approach which is an essential pre-requisite for integrated approach, the aim clearly is to maximize the efficiency of the total system through appropriate optimization of each of its sub-systems. In other words, in the team, the inputs from each of the professional disciplines have to be so optimized that the total system's efficiency becomes the maximum. It may be re-emphasized that maximizing the efficiencies of each sub-system may not necessarily assure the maximization of the efficiency of the total system. It need hardly to be stated that specified or implied safety will always get precedence over functional efficiency and economy. Further, progressive approach such as that relating to the concept of intelligent buildings would be best taken care of by the 'integrated approach' as laid down in this Part.

Quality systems approach and certification thereunder covering the various dimensions brought out above may go a long way in achieving the above goal of real integrated approach.

NATIONAL BUILDING CODE OF INDIA

PART 0 INTEGRATED APPROACH — PREREQUISITE FOR APPLYING PROVISIONS OF THE CODE

1 SCOPE

This Part covers guidelines to be followed for judicious implementation of the provisions of various Parts/Sections of the Code.

2 TERMINOLOGY

2.0 For the purpose of this Part, the following definitions and those given in Part 1 'Definitions' shall apply.

2.1 Authority Having Jurisdiction — The Authority which has been created by a statute and which, for the purpose of administering the Code/Part, may authorize a committee or an official or an agency to act on its behalf; hereinafter called the 'Authority'.

2.2 Building — Any structure for whatsoever purpose and of whatsoever materials constructed and every part thereof whether used as human habitation or not and includes foundation, plinth, walls, floors, roofs, chimneys, plumbing and building services, fixed platforms, *VERANDAH*, balcony, cornice or projection, part of a building or anything affixed thereto or any wall enclosing or intended to enclose any land or space and signs and outdoor display structures. Tents/ *SHAMIANAHS/PANDALS*, tarpaulin shelters, etc, erected for temporary and ceremonial occasions shall not be considered as building.

2.3 Owner — Person or body having a legal interest in land and/or building thereon. This includes free holders, leaseholders or those holding a sub-lease which both bestows a legal right to occupation and gives rise to liabilities in respect of safety or building condition.

In case of lease or sub-lease holders, as far as ownership with respect to the structure is concerned, the structure of a flat or structure on a plot belongs to the allottee/ lessee till the allotment/lease subsists.

NOTE — For the purpose of the Code, the word 'owner' will also cover the generally understood terms like 'client', 'user', etc.

3 GENERAL

3.1 Buildings, shall be classified as Residential, Educational, Institutional, Assembly, Business, Mercantile, Industrial, Storage and Hazardous in groups and sub-division as classified in Part 4 'Fire and Life Safety'.

For further sub-classification of buildings and various related provisions thereof with respect to administration; development control rules and general building requirements; building materials; fire and life safety; structural design; constructional practices and safety; building and plumbing services; and landscaping, signs and outdoor display structures, other parts/sections of the Code may be referred to.

3.2 The scope of various Parts/Sections of the Code which cover detailed provisions on different aspects of development of land/building construction activity, are given in Annex A, with a view to providing an overview for the users of the Code.

4 TEAM APPROACH

A land development/building project comprises the following major stages:

a) Location/siting,
b) Conceptualization and planning,
c) Designing and detailing,
d) Construction/execution, and
e) Maintenance and repair.

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, in an effective manner.

Appropriate multi-disciplinary teams need to be constituted to successfully meet the requirements of different stages. Each team may comprise need based professionals out of the following depending upon the nature, magnitude and complexity of the project:

a) Architect,
b) Civil engineer,
c) Structural engineer,
d) Electrical engineer,
e) Plumbing engineer,
f) Fire protection engineer,
g) HVAC engineer,
h) Environment specialist,
j) Town planner,
k) Urban designer,
m) Landscape architect,
n) Security system specialist,
p) Interior designer,
q) Quantity surveyor,
r) Project/construction manager, and
s) Other subject specialist(s).

4.1 Design Team

In building projects various aspects like form; space planning; aesthetics; fire and life safety; structural adequacy; plumbing services; lighting and natural ventilation; electrical and allied installations; air conditioning, heating and mechanical ventilation; acoustics, sound insulation and noise control; installation of lifts and escalators; building automation; data and voice communication; other utility services installations; landscape planning and design; urban planning; etc need to be kept in view right at the concept stage. The project requiring such multi-disciplinary inputs need a co-ordinated approach among the professionals for proper integration of various design inputs. For this, and to take care of the complexities of multi-disciplinary requirements, a design team of professionals from required disciplines shall be constituted at the appropriate stage. Here, it is desirable that the multi-disciplinary integration is initiated right from the concept stage. The team shall finalize the plan. The composition of the team shall depend on the nature and magnitude of the project. Design is an evolutionary and participatory process, where participation of owner constitutes a very important input at all stages, and the same shall be ensured by the design team.

To ensure proper implementation of the design, the design team, may be associated during the construction/ execution stage.

4.2 Project Management and Construction Management Teams

The objective of project management or construction management is primarily to achieve accomplishment of project in accordance with the designs and specifications in a stipulated time and cost framework, with a degree of assurance prior to commencement and satisfaction on accomplishment.

For large projects, separate teams of experienced professionals from the required disciplines may be constituted for project management and for construction management depending upon the complexities of the project. However, for smaller projects these teams may be combined. The teams shall be responsible for day-to-day execution, supervision, quality control, etc and shall ensure inter-disciplinary co-ordination during the construction stage. The team shall be responsible to achieve satisfactory completion of the project with regard to cost, time and quality. Some members of the design team may also be included in the project management team and/or associated actively during the project execution stage. It is important that leaders and members of project management/construction management teams, depending on the size and complexity of the project, are carefully selected considering their qualification, experience and expertise in these fields.

4.3 Operation and Maintenance Team

Operation, maintenance and repairs also require a multi-disciplinary approach to ensure that all the requirements of the users are satisfactorily met. During maintenance and repairs, the jobs requiring inter-disciplinary co-ordination have to be executed in such a manner as not only to cause least inconvenience to the user but also to ensure that there is no mismatch or damage to the structure, finishings, fittings and fixtures. For carrying out routine maintenance/repair jobs, utilization of the services of trained technicians preferably having multi-disciplinary skills should be encouraged.

Special repairs, rehabilitation and retrofitting are specialized jobs which demand knowledge of the existing structure/installations. Association of concerned specialists may be helpful for these works.

The Operation and Maintenance Team may also be known as Asset Management or Estate Management Team.

5 PLANNING, DESIGNING AND DEVELOPMENT

5.1 The main functions of design team (*see* **4.1**) constituted for the planning, designing and development, are as under:

a) Formalization of design brief in consultation with the owner.
b) Site investigation/survey.
c) Preparation of alternative concept designs.
d) Selection of a concept in consultation with and with the consent of owner.
e) Sizing the system.
f) Development of design, covering :
    1) Integration of architecture, structure and services,
    2) Synthesis of requirements of each discipline, and
    3) Interaction with each other and with the owner.
g) Preparation of preliminary designs and drawings and obtaining owner's approval.
h) Preparation of preliminary cost estimates for approval of owner.
j) Preparation of work-breakdown structure and programme for pre-construction activities.
k) Assisting client to obtain approvals of the Authority.
m) Preparation of detailed specification and

construction working drawings with integration of engineering inputs of all concerned disciplines.

n) Preparation of detailed design of each discipline for various services.

p) Peer review/proof checking of the drawings/ designs in case of important projects, depending upon their complexity and sensitivity.

q) Preparation of detailed cost estimate.

r) Obtaining final approval of client.

s) Preparation of bill of quantities, specifications and tender documents.

5.2 The following considerations, as may be applicable to the project, may be considered during planning, notwithstanding other relevant aspects specifically prescribed in concerned parts/sections of this Code; these considerations in general are with the objective of addressing to the important issues like environmental protection, energy conservation, cultural issues, creating barrier free built-environment, safety aspects, etc, all of these leading towards sustainable development, and have to be applied with due regard to the specific requirements of size and type of project:

a) Geoclimatic, geological and topographical features.

b) Varied sociological pattern of living in the country.

c) Effective land use to cater to the needs of the society in a most convenient manner.

d) Modular planning and standardization to take care of future planning giving due consideration to the specified planning controls.

e) Emphasis on daylight utilization, natural ventilation, shielding, and window area and its disposition; daylighting to be supplemented with an integrated design of artificial lighting.

f) Optimum utilization of renewable energy sources duly integrated in the overall energy system design; with consideration of active and passive aspects in building design including thermal performance of building envelope.

g) Rain water harvesting, and use of appropriate building materials considering aspects like energy consumption in production, transportation and utilization, recyclability, etc for promoting sustainable development.

h) Requisite mandatory provisions for handicapped persons.

j) Acoustical controls for buildings and the surroundings.

k) Promotion of artwork in buildings, specially buildings of importance.

m) Due cognizance of recommendations of the Archeological Survey of India with regard to national monuments and construction in archeologically important sites.

n) Due cognizance of relevant provisions of applicable coastal zone regulation act.

p) Conservation of heritage structures and areas.

q) Environmental and social impact analysis.

r) Design of services with emphasis on aspects of energy efficiency, environment friendliness and maintainability.

s) Integrated waste management.

t) Voice and data communication, automation of building services, and intelligent building; use of security and surveillance system in important and sensitive buildings, such as, access control for the people as well as for vehicle.

u) Interlinking of fire alarm system, fire protection system, security system, ventilation, electrical systems, etc.

v) Analysis of emergency power, standby power requirement and captive power systems.

w) Cost optimization through techniques like value engineering.

y) Adoption of innovative technologies giving due consideration to constructability and quality aspects.

z) Instrumentation of buildings and monitoring and use of information so generated to effect improvements in planning and design of future building projects.

6 CONSTRUCTION/EXECUTION (ACTUALIZATION)

6.1 The main functions of the teams (*see* **4.2**) constituted for Project Management/Construction Management may be, to :

a) specify criteria for selection of constructors;

b) specify quality control, quality audit system and safety system;

c) short-list constructors;

d) have pre-bid meetings with the intending constructors;

e) receive and evaluate tenders;

f) select constructors;

g) execution and supervision;

h) monitor quality, time and cost control;

j) prepare/certify the completion (as-built) drawings; and

k) ensure availability of operation manuals for field use.

6.2 Apart from the specific provisions laid down in the concerned Parts/Sections of the Code, the following considerations, as may be applicable to the project concerned, shall be given due attention:

a) Adopting scientific principles of construction management, quality management, cost and time control.

b) Engagement of executing and supervising agencies, which meet the specified norms of skills, specialization, experience, resourcefulness, etc for the work.

c) Ensuring inter-disciplinary co-ordination during construction.

d) Contract management and techno-legal aspects.

e) Completion, commissioning and trial run of installations/equipments and their operation and maintenance through the suppliers/other teams, where necessary.

f) Make available shop drawings as well as as-built drawings for the building and services.

g) Arrange all maintenance and operation manual from the concerned suppliers/ manufacturers.

6.3 The team of professionals (*see* **4.2**) shall work and monitor the project activities for successful construction/execution of the project with regard to cost, time, quality and safety.

7 OPERATION AND MAINTENANCE

7.1 The team of professionals (*see* **4.3**) shall set up a system of periodic maintenance and upkeep of constructed buildings.

7.2 The operation and maintenance team shall be responsible for preparation/application of operation and maintenance manual, and draw maintenance schedule/frequencies and guidelines for maintenance personnel. Apart from the specific provisions laid down in concerned Parts/Sections of the Code, the following, as may be applicable to the project concerned shall additionally be taken into account:

a) Periodic validation of buildings by competent professionals through inspection of the buildings in respect of structural safety and safety of electrical and other installations and ensuring that all fire safety equipments/ systems are in proper working condition.

b) Preparation of preventive maintenance schedules for all installations in the building and strictly following the same; the record of the preventive maintenance to be properly kept.

c) Ensuring inter-disciplinary co-ordination during maintenance and repairs; deployment of trained personnel with multi-disciplinary skills to be encouraged.

d) Condition survey of structures and installations, identification of distress of various elements and initiating plans for rehabilitation/retrofitting well in time.

7.3 The proposals for rehabilitation/retrofitting should be prepared after detailed investigations through visual inspection, maintenance records and testing as required and got executed through specialized agencies under the guidance and supervision of competent professionals.

ANNEX A

(*Clause* 3.2)

BRIEF DETAILS OF THE COVERAGE OF VARIOUS PROVISIONS UNDER DIFFERENT OTHER PARTS/SECTIONS OF THIS CODE

A-1 PART 1 DEFINITIONS

It lists the terms appearing in all the Parts/Sections of the Code. However, some common definitions are reproduced in this Part also.

A-2 PART 2 ADMINISTRATION

It covers the administrative aspects of the Code, such as applicability of the Code, organization of building department for enforcement of the Code, procedure for obtaining development and building permits, and responsibility of the owner and all professionals involved in the planning, design and construction of the building.

A-3 PART 3 DEVELOPMENT CONTROL RULES AND GENERAL BUILDING REQUIREMENTS

It covers the development control rules and general building requirements for proper planning and design at the layout and building level to ensure health safety, public safety and desired quality of life.

A-4 PART 4 FIRE AND LIFE SAFETY

It covers the requirements for fire prevention, life safety in relation to fire, and fire protection of buildings. The Code specifies planning and construction features and fire protection features for all occupancies that are necessary to minimize danger to life and property.

A-5 PART 5 BUILDING MATERIALS

It covers the requirements of building materials and components, and criteria for accepting new or alternative building materials and components.

A-6 PART 6 STRUCTURAL DESIGN

This Part through its seven sections provides for structural adequacy of buildings to deal with both internal and external environment, and provide guidance to engineers/structural engineers for varied usage of material/technology types for building design.

A-6.1 Section 1 Loads, Forces and Effects

It covers basic design loads to be assumed in the design of buildings. The live loads, wind loads, seismic loads, snow loads and other loads, which are specified therein, are minimum working loads which should be taken into consideration for purposes of design.

A-6.2 Section 2 Soils and Foundations

It covers structural design (principles) of all building foundations, such as, raft, pile 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.

A-6.3 Section 3 Timber and Bamboo

A-6.3.1 *Section 3A Timber*

It covers the use of structural timber in structures or elements of structures connected together by fasteners/ fastening techniques.

A-6.3.2 *Section 3B Bamboo*

It covers the use of bamboo for constructional purposes in structures or elements of the structure, ensuring quality and effectiveness of design and construction using bamboo. It covers minimum strength data, dimensional and grading requirements, seasoning, preservative treatment, design and jointing techniques with bamboo which would facilitate scientific application and long-term performance of structures. It also covers guidelines so as to ensure proper procurement, storage, precautions and design limitations on bamboo.

A-6.4 Section 4 Masonry

It covers the structural design aspects of unreinforced load bearing and non-load bearing walls, constructed using various bricks, stones and blocks permitted in accordance with this Section. This, however, also covers provisions for design of reinforced brick and reinforced brick concrete floors and roofs. It also covers guidelines regarding earthquake resistance of low strength masonry buildings.

A-6.5 Section 5 Concrete

A-6.5.1 *Section 5A Plain and Reinforced Concrete*

It covers the general structural use of plain and reinforced concrete.

A-6.5.2 *Section 5B Prestressed Concrete*

It covers the general structural use of prestressed concrete. It covers both work carried out on site and the manufacture of precast prestressed concrete units.

A-6.6 Section 6 Steel

It covers the use of structural steel in general building construction including the use of hot rolled steel sections and steel tubes.

A-6.7 Section 7 Prefabrication, Systems Building and Mixed/Composite Construction

A-6.7.1 *Section 7A Prefabricated Concrete*

It covers recommendations regarding modular planning, component sizes, prefabrication systems, design considerations, joints and manufacture, storage, transport and erection of prefabricated concrete elements for use in buildings and such related requirements for prefabricated concrete.

A-6.7.2 *Section 7B Systems Building and Mixed/ Composite Construction*

It covers recommendations regarding modular planning, component sizes, joints, manufacture, storage, transport and erection of prefabricated elements for use in buildings and such related requirements for mixed/composite construction.

A-7 PART 7 CONSTRUCTIONAL PRACTICES AND SAFETY

It covers the constructional planning, management and practices in buildings; storage, stacking and handling of materials and safety of personnel during construction operations for all elements of a building and demolition of buildings. It also covers guidelines relating to maintenance management, repairs, retrofitting and strengthening of buildings. The objective can be best achieved through proper coordination and working by the project management and construction management teams.

A-8 PART 8 BUILDING SERVICES

This Part through its five elaborate sections on utilities provides detailed guidance to concerned professionals/ utility engineers for meeting necessary functional requirements in buildings.

A-8.1 Section 1 Lighting and Ventilation

It covers requirements and methods for lighting and ventilation of buildings.

A-8.2 Section 2 Electrical and Allied Installations

It covers the essential requirements for electrical and allied installations in buildings to ensure efficient use of electricity including safety from fire and shock. This Section also includes general requirements relating to lightning protection of buildings.

A-8.3 Section 3 Air Conditioning, Heating and Mechanical Ventilation

This Section covers the design, construction and installation of air conditioning and heating systems and equipment installed in buildings for the purpose of providing and maintaining conditions of air temperature, humidity, purity and distribution suitable for the use and occupancy of the space.

A-8.4 Section 4 Acoustics, Sound Insulation and Noise Control

It covers requirements and guidelines regarding planning against noise, acceptable noise levels and the requirements for sound insulation in buildings with different occupancies.

A-8.5 Section 5 Installation of Lifts and Escalators

It covers the essential requirements for the installation, operation, maintenance and also inspection of lifts (passenger lifts, goods lifts, hospital lifts, service lifts and dumb-waiter) and escalators so as to ensure safe and satisfactory performance.

A-9 PART 9 PLUMBING SERVICES

This Part through its two sections gives detailed guidance to concerned professionals/plumbing engineers with regard to plumbing and other related requirements in buildings.

A-9.1 Section 1 Water Supply, Drainage and Sanitation (Including Solid Waste Management)

It covers the basic requirements of water supply for residential, business and other types of buildings, including traffic terminal stations. This Section also deals with general requirements of plumbing connected to public water supply and design of water supply systems.

It also covers the design, layout, construction and maintenance of drains for foul water, surface water and sub-soil water and sewage; together with all ancillary works, such as connections, manholes and inspection chambers used within the building and from building to the connection to a public sewer, private sewer, individual sewage-disposal system, cess-pool, soakaway or to other approved point of disposal/ treatment work. It also includes the provisions on solid waste management.

A-9.2 Section 2 Gas Supply

It covers the requirements regarding the safety of persons and property for all piping uses and for all types of gases used for fuel or lighting purposes in buildings.

A-10 PART 10 LANDSCAPING, SIGNS AND OUTDOOR DISPLAY STRUCTURES

A-10.1 Section 1 Landscape Planning and Design

It covers requirements of landscape planning and design with the view to promoting quality of outdoor built environment and protection of land and its resources.

A-10.2 Section 2 Signs and Outdoor Display Structures

It covers the requirements with regard to public safety, structural safety and fire safety of all signs and outdoor display structures including the overall aesthetical aspects of imposition of signs and outdoor display structures in the outdoor built environment.

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 1 Loads, Forces and Effects

BUREAU OF INDIAN STANDARDS

CONTENTS

National Building Code Sectional Committee, CED 46

FOREWORD

This Section 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 load, snow load, special loads and load combinations.

This Code was first published in 1970 and revised in 1983. Subsequently the first revision of this Section was modified in 1987 through Amendment No. 2 to the 1983 version of the Code to bring this Section in line with the latest revised loading code. Now, in view of the revision of the important Indian Standard on earthquake resistant design of structure, that is IS 1893, a need to revise this Part was felt. This revision has therefore been prepared to take into account this revised standard, IS 1893 (Part 1) : 2002 'Criteria for earthquake resistant design of structures: Part 1 General provision and buildings (*fifth revision*)' and also incorporate latest information on additional loads, forces and effects as also the details regarding multi-hazard risk in various districts of India.

The significant changes incorporated in this revision include:

a) The seismic zone map is revised with only four zones, instead of five. Erstwhile Zone I has been merged in to Zone II. Hence, Zone I does not appear in the new zoning; only Zones II, III, IV and V do.

b) The values of seismic zone factors have been changed; these now reflect more realistic values of effective peak ground acceleration considering Maximum Considered Earthquake (MCE) and service life of structure in each seismic zone.

c) Response spectra are now specified for three types of founding strata, namely rock and hard soil, medium soil and soft soil.

d) Empirical expression for estimating the fundamental natural period T_a of multi-storeyed buildings with regular moment resisting frames has been revised.

e) This revision adopts 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. Then, the concept of response reduction due to ductile deformation or frictional energy dissipation in the cracks is brought in this Section explicitly, by introducing the 'response reduction factor' in place of the earlier performance factor.

f) A lower bound is specified for the design base shear of buildings, based on empirical estimate of the fundamental natural period T_a.

g) The soil-foundation system factor is dropped. Instead, a clause has been introduced to restrict the use of foundations vulnerable to differential settlements in severe seismic zones.

h) Torsional eccentricity values have been revised upwards in view of serious damages observed in buildings with irregular plans.

j) Modal combination rule in dynamic analysis of buildings has been revised.

k) Other clauses have been redrafted where necessary for more effective implementation.

m) A new clause on multi-hazard risk in various districts of India and a list of districts identified as multi-hazard prone districts have been included.

n) Latest amendments issued to IS 875 have been incorporated.

p) A clause on vibration in buildings has been introduced for general guidance.

q) Reference has been included to the Indian Standards on landslide control and design of retaining walls, formulated after the last revision of the Section.

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

IS 1893 (Part 1) : 2002	Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings (*fifth revision*)

IS 875 (Part 2) : 1987	Code of practice for design loads (other than earthquake) for buildings and structures: Part 2 Imposed loads (*second revision*)
IS 875 (Part 3) : 1988	Code of practice for design loads (other than earthquake) for buildings and structures: Part 3 Wind loads (*second revision*)
IS 875 (Part 4) : 1987	Code of practice for design loads (other than earthquake) for buildings and structures: Part 4 Snow loads (*second revision*)
IS 875 (Part 5) : 1987	Code of practice for design loads (other than earthquake) for buildings and structures: Part 5 Special loads and load combinations (*second revision*)

This Section has to be read together with Sections 2 to 7 of Part 6 'Structural Design'.

A reference to SP 64 (S&T) : 2001 'Explanatory Handbook on Indian Standard Code of practice for design loads (other than earthquake) for buildings and structures: Part 3 Wind loads IS 875 (Part 3) : 1987' may be useful. This publication gives detailed background information on the provisions for wind loads and also the use of these provisions for arriving at the wind loads on buildings and structures while evaluating their structural safety.

Reference may also be made to the Vulnerability Atlas of India, 1997 and Landslide Hazard Zonation Atlas of India, 2003 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.

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 1 Loads, Forces and Effects

1 SCOPE

1.1 This Section covers basic design loads to be assumed in the design of buildings. The imposed loads, wind loads, seismic loads, snow loads and other loads, which are specified herein, are minimum working loads which should be taken into consideration for purposes of design.

1.2 This Section does not take into consideration loads incidental to construction.

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 in the building and shall conform to good practice [6-1(1)].

3 IMPOSED LOAD

3.1 This clause covers imposed loads (live 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, 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.

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 or processed like assembly plants, power plants, refineries, gas plants, mills, dairies, factories, workshops, 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, sanitoria, 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. The occupancy classification is given in the following groups.

3.2.1.9 *Office Buildings* — 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, punching cards or tapes, drawing of matter for publication and the editorial preparation of matter for publication.

3.2.1.10 *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.

3.2.1.11 *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 one or multi-family dwellings, apartment houses (flats), lodging or rooming houses, restaurants, hostels, dormitories and residential hotels.

3.2.1.12 *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, transity 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.

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.

Table 1 Imposed Floor Loads for Different Occupancies

(*Clause* 3.3.1)

Sl No.	Occupancy Classification	Uniformly Distributed Load (UDL)	Concentrated Load
(1)	(2)	(3)	(4)
i)	*Residential Buildings*	KN/m^2	KN
	a) Dwelling houses:		
	1) All rooms and kitchens	2.0	1.8
	2) Toilets and bathrooms	2.0	—

Table 1 — *Continued*

(1)	(2)	(3)	(4)
	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 [6-1(2)] only:		
	1) Habitable rooms, kitchens, and toilets and bathrooms	1.5	1.4
	2) Corridors, passages and staircases including fire escapes	1.5	1.4
	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
	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 services (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 roomsto 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 tonnes gross weight, including access ways and ramps — to be calculated but not less than	2.5	9.0
	2) Garage floors for vehicles not exceeding 4.0 tonnes gross weight (including access ways and ramps) — to be calculated but not less than	5.0	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	5.0	—
	f) Kitchens	3.0	4.5
	g) Toilets and bathrooms	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 calculated but not less than	4.0	4.5
	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 metre run concentrated at the outer edge

Table 1 — *Continued*

(1)	(2)	(3)	(4)
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
	h) Boiler rooms and plant rooms — to be calculated but not less than	5.0	4.5
	j) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 metre run concentrated at the outer edge
iv)	*Assembly Building*		
	a) Assembly areas:		
	1) With fixed seats [2)]	4.0	—
	2) Without fixed seats	5.0	3.6
	b) Restaurants (subject to assembly), museums and art galleries and gymnasia	4.0	4.5
	c) Projection rooms	5.0	—
	d) Stages	5.0	4.5
	e) Office rooms, kitchens and laundries	3.0	4.5
	f) Dressing rooms	2.0	1.8
	g) Lounges and billiards rooms	2.0	2.7
	h) Toilets and bathrooms	2.0	—
	j) Corridors, passages and staircases including fire escapes	4.0	4.5
	k) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 metre run concentrated at the outer edge
	m) Boiler rooms and plant rooms including weight of machinery	7.5	4.5
	n) 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
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 equipment)	3.5	4.5
	e) Records/files store rooms and storage space	5.0	4.5
	f) Vaults and strong rooms — to be calculated but not less than	5.0	4.5
	g) Cafeterias and dinning rooms	3.0[1)]	2.7
	h) Kitchens	3.0	2.7
	j) Corridors, passages, lobbies, staircases including fire escapes — as per the floor serviced (excluding stores) but not less than	4.0	4.5
	k) Bath and toilets rooms	2.0	—
	m) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 metre run concentrated at the outer edge
	n) Stationary stores	4.0 for each metre of storage height	9.0
	p) Boiler rooms and plant rooms — to be calculated but not less than	5.0	6.7
	q) Libraries	*See* Sl No. (ii)	

Table 1 — *Concluded*

(1)	(2)	(3)	(4)
vi)	*Merchantile 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, staircases 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 metre run concentrated at the outer edge
vii)	*Industrial Buildings*[4]		
	a) Work areas without machinery/equipment	2.5	4.5
	b) Work areas with machinery/equipment[3]		
	1) Light duty — To be calculated but not less than	5.0	4.5
	2) Medium duty — To be calculated but not less than	7.0	4.5
	3) Heavy duty — To be calculated but not less than	10.0	4.5
	c) Boiler rooms and plant rooms — to be calculated but not less than	5.0	6.7
	d) Cafeterias and dinning rooms	3.0[1]	2.7
	e) Corridors, passages, staircases including fire escapes	4.0	4.5
	f) Corridors, passages, lobbies, staircases subject to machine loads and wheeled vehicles — to be calculated but not less than	5.0	4.5
	g) Kitchens	3.0	4.5
	h) 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

[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.

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.

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)	(2)
1	0
2	10
3	20
4	30
5 to 10	40
Over 10	50

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. However, 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.

   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 A illustrating the reduction of imposed loads in a multi-storeyed building in the design of column members.

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. However, 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, girdes 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.

DESIGNED IMPOSED FLOOR LOADING

Distributed………………………kN/m^2

Concentrated………………………..kN

Label Indicating Designed Imposed Floor Loading

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 lable.

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

(*Clause* 3.4.1)

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°	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°	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°	$(0.75 - 0.52\,\alpha^2)$ kN/m^2 where $\alpha = h/l$ 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).	Subject to a minimum of 0.4 kN/m^2

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.

3.4.1.1 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 12.5 cm^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**, **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.

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 12.5 cm placed at the most favourable 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 and Balustrades

3.5.1 *Parapets, Parapet Walls and Balustrades*

Parapets, parapet walls and balustrades, 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.

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:

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
(1)	(2)	(3)
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)	All other stairways, landings and balconies and all parapets and handrails to roofs [except those subject to overcrowding covered under (iv)]	0.75
iv)	Parapets and balustrades in place of assembly, such as theatres, cinemas, churches, schools, places of entertainment, sports and buildings and buildings likely to be overcrowded	2.25

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.

	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*

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.

3.6.2.1 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 in informal table 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.

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* [6-1(3)] for classification (Class I to IV) of cranes.

Impact Allowance for Crane Girders

(*Clause* 3.6.3)

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

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*

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

b) 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.

4 WIND LOAD

4.1 General

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

NOTES

1 It is believed that ultimately wind load estimation will be made by taking into account the random variation of wind speed with time, but available theoretical methods have not matured sufficiently at present for use in the Section. For this reason, equivalent static load estimation which implies a steady wind speed, which has proved to be satisfactory for normal, short and heavy structures, is given in **4.5** and **4.6**. However, a beginning has been made to take account of the random nature of the wind speed by requiring that the along-wind or drag load on structures which are prone to wind induced oscillations, be also determined by the gust factor method (*see* **4.8**) and the more severe of the two estimates be taken for design.

A large majority of structures met within practice do not, however, suffer wind induced oscillations and generally do not require to be examined for the dynamic effects of wind including use of gust factor method. Nevertheless, there are various types of structures or their components, such as some tall buildings, etc, which require investigation of wind induced oscillations. In identifying and analyzing such structures **4.6** shall be followed.

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

4.1.1 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 to 30 m above ground.

4.1.2 Very strong wind speeds (greater 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-Eastern India.

4.1.3 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 the 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 the 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 the design of structures with high slenderness ratios.

4.1.4 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.

4.1.5 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.

4.1.6 Buildings shall also be designed with due attention to the effects of wind on the comfort of people inside and outside the buildings.

4.1.7 The stability calculations of the building 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 parts of the building above the general roof level.

4.2 Notations

The notations to be followed, unless otherwise specified in relevant clauses under wind loads, are given in Annex B.

4.3 Terminology

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

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

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

4.3.1.3 *Depth* — Depth 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 the wind, whereas length and width are dimensions related to the plan.

4.3.1.4 *Developed height*

Developed height 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 equals that of the unpenetrated earlier velocity profile at that height.

4.3.1.5 *Effective frontal area*

The projected area of the structure normal to the direction of the wind.

4.3.1.6 *Element 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.

4.3.1.10 *Peak gust*

Peak gust or peak gust speed is the wind speed associated with the maximum amplitude.

4.3.1.11 *Fetch length*

Fetch length 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*

Gradient height 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 *Mean ground level*

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

4.3.1.14 *Pressure coefficient*

Pressure coefficient is the ratio of the difference between the pressure acting at a point on a surface and the static pressure of the incident wind to the design wind pressure, where the static and design wind pressure 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.

4.3.1.15 *Return period*

Return period is the number of years, the reciprocal of which gives the probability of extreme wind exceeding a given wind speed in any one year.

4.3.1.16 *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.

4.3.1.17 *Suction*

Suctions means pressure less than the atmospheric (static) pressure and is taken to act away from the surface.

4.3.1.18 *Solidity ratio*

Solidity ratio 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.

4.3.1.19 *Terrain category*

Terrain category 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.

4.3.1.20 *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.

4.3.1.21 *Topography*

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

4.4 Wind Speed and Pressure

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 a maximum at a height called the gradient height. There is usually a slight change in direction (Ekman effect) but this is ignored in the 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 magnitude into an average or mean value and a fluctuating component around this average value. The average value depends on the averaging time employed in analyzing the meteorological data and this averaging time varies from a few seconds to several minutes. The magnitude of the fluctuating component of the wind speed, which is called as gust, depends on the averaging time. In general, smaller the averaging interval, greater 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 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 C.

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 V_z, design wind speed at any height for the chosen structure.

a) risk level;
b) terrain roughness, height and size of structure; and
c) local topography.

It can be mathematically expressed as follows:

$$V_z = V_b k_1 k_2 k_3$$

where

V_z = design of wind speed at any height z in m/s;
V_b = basic wind speed in m/s (Fig. 1);
k_1 = probability factor (risk coefficient) (**4.4.3.1**);
k_2 = terrain, height and structure size factor (**4.4.3.2**); and
k_3 = topography factor (**4.4.3.3**)

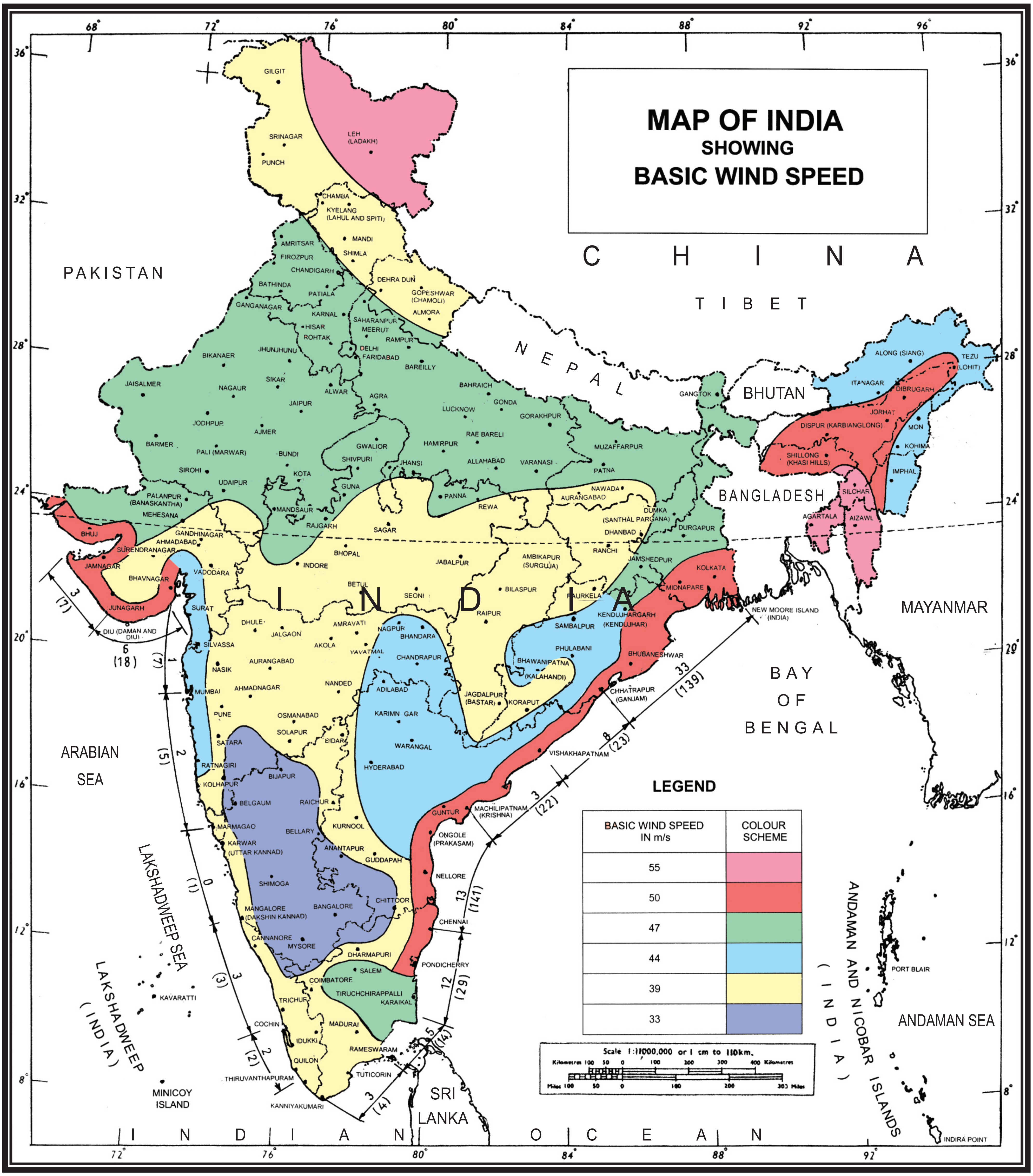
NOTE — Design wind speed up to 10 m height from mean ground level shall be considered constant.

4.4.3.1 *Risk coefficient* (k_1)

Figure 1 gives basic wind speeds for terrain Category 2 as applicable at 10 m above ground level based on 50 year mean return period. The suggested life period to be assumed in design and the corresponding k_1 factors for different classes of structure for the purpose of design is given in Table 4. In the design of all 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 and structure size factor* (k_2)

a) *Terrain* — Selection of terrain categories shall be made with due regard to the effect of the obstruction which constitute the ground surface roughness. The terrain category used in the



Based upon Survey of India Outline Map printed in 1993.

The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.
The boundary of Meghalaya shown on this map is as interpreted from the North-Eastern Areas (Reorganisation) Act, 1971, but has yet to be verified.
Responsibility for correctness of internal details shown on the map rests with the publisher.
The state boundaries between Uttaranchal & Uttar Pradesh, Bihar & Jharkhand and Chhatisgarh & Madhya Pradesh have not been verified by Governments concerned.

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

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

(*Clause* 4.4.3.1)

Class of Structure	Mean Probable Design Life of Structure in Years	k_1 Factor for Basic Wind Speed (m/s) of					
		33	39	44	47	50	55
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
All general buildings and structures	50	1.0	1.0	1.0	1.0	1.0	1.0
Temporary sheds, structures such as those used during construction operations (for example, formwork and falsework), structures during construction stages and boundary walls	5	0.82	0.76	0.73	0.71	0.70	0.67
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
Important buildings and structures, such as hospitals, communications buildings/towers and power plant structures	100	1.05	1.06	1.07	1.07	1.08	1.08

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

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 at least once in N successive years), nominal value = 0.63;

X_N, P_N = extreme wind speed for given values of N and P_N; and

$X_{50,\,0.63}$ = extreme wind speed for N = 50 years and P_N = 0.63

A and *B* are coefficients having the following values for different basic wind speed zones:

Zone	*A*	*B*
33 m/s	83.2	9.2
39 m/s	84.2	14.0
44 m/s	88.0	18.0
47 m/s	88.0	20.5
50 m/s	88.8	22.8
55 m/s	90.8	27.3

NOTE — The factor k_1 is based on statistical concepts which take account of 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; the greater the period of years over which there will be exposure to wind, the greater is the probability. Higher return periods ranging from 100 to 1 000 years (implying lower risk level) in association with greater periods of exposure may have to be selected for exceptionally important structures, such as nuclear power reactors and satellite communication towers. Equation given above may be used in such cases to estimate k_1 factors for different periods of exposure and chosen probability of exceedence (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 in Table 4.

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.

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

Category 1 — Exposed open terrain with few or no obstructions and in which the average height of any objects surrounding the structure is less than 1.5 m.

NOTE — This category includes open sea-coasts and flat treeless plains.

Category 2 — Open terrain with well scattered obstructions having heights generally between 1.5 and 10 m.

NOTE — This is the criterion for measurement of regional basic wind speeds and includes airfields, open parklands 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.

Category 3 — Terrain with numerous closely spaced obstruction having the size of building-structures up to 10 m in height with or without a few isolated tall structures.

NOTES

1 This category includes well wooded areas and shrubs, towns and industrial areas fully or partially developed.

2 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.

3 Particular attention must be given to the performance of the obstructions in areas affected by fully developed tropical cyclones. Vegetation, which is likely to be blown down or defoliated, cannot be relied upon to maintain Category 3 conditions. Where such situation may exist, either an intermediate category with velocity multipliers midway between the values for Categories 2 and 3 given in Table 5 or Category 2 should be selected having due regard to local conditions.

Category 4 — Terrain with numerous large high closely spaced obstructions.

NOTE — This category includes large city centres, generally with obstructions above 25 m and well developed industrial complexes.

b) *Variation of Wind Speed with Height for Different Sizes of Structure in Different Terrains (k_2 Factor)* — 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 for different sizes of buildings/structures.

The buildings/structures are classified into the following three different classes depending upon their size:

Class A — Buildings and/or their components, such as cladding, glazing, roofing etc, having maximum dimension (greatest horizontal or vertical dimension) less than 20 m.

Class B — Buildings and/or their components, such as cladding, glazing, roofing etc, having maximum dimension (greatest horizontal or vertical dimension) between 20 m and 50 m.

Class C — Buildings and/or their components, such as cladding, glazing, roofing etc, having maximum dimension (greatest horizontal or vertical dimension) greater than 50 m.

c) *Terrain Categories in Relation to the Direction of Wind* — The terrain category used in the design of a building 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.

Table 5 k_2 Factors to Obtain Design Wind Speed Variation with Height in Different Terrains for Different Classes of Building Structures

[*Clause* 4.4.3.2 (b)]

Height (m)	Terrain Category 1 Class			Terrain Category 2 Class			Terrain Category 3 Class			Terrain Category 4 Class		
	A	B	C	A	B	C	A	B	C	A	B	C
10	1.05	1.03	0.99	1.00	0.98	0.93	0.91	0.88	0.82	0.80	0.76	0.67
15	1.09	1.07	1.03	1.05	1.02	0.97	0.97	0.94	0.87	0.80	0.76	0.67
20	1.12	1.10	1.06	1.07	1.05	1.00	1.01	0.98	0.91	0.80	0.76	0.67
30	1.15	1.13	1.09	1.12	1.10	1.04	1.06	1.03	0.96	0.97	0.93	0.83
50	1.20	1.18	1.14	1.17	1.15	1.10	1.12	1.09	1.02	1.10	1.05	0.95
100	1.26	1.24	1.20	1.24	1.22	1.17	1.20	1.17	1.10	1.20	1.15	1.05
150	1.30	1.28	1.24	1.28	1.25	1.21	1.24	1.21	1.15	1.24	1.20	1.10
200	1.32	1.30	1.26	1.30	1.28	1.24	1.27	1.24	1.18	1.27	1.22	1.13
250	1.34	1.32	1.28	1.32	1.31	1.26	1.29	1.26	1.20	1.28	1.24	1.16
300	1.35	1.34	1.30	1.34	1.32	1.28	1.31	1.28	1.22	1.30	1.26	1.17
350	1.37	1.35	1.31	1.36	1.34	1.29	1.32	1.30	1.24	1.31	1.27	1.19
400	1.38	1.36	1.32	1.37	1.35	1.30	1.34	1.31	1.25	1.32	1.28	1.20
450	1.39	1.37	1.33	1.38	1.36	1.31	1.35	1.32	1.26	1.33	1.29	1.21
500	1.40	1.38	1.34	1.39	1.37	1.32	1.36	1.33	1.28	1.34	1.30	1.22

NOTES

1 *See* **4.4.3.2** (b) for definitions of Class A, Class B and Class C structures.

2 Intermediate values may be obtained by linear interpolation, if desired. It is permissible to assume constant wind speed between two heights for simplicity.

d) *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 develops gradually to height (h_x), which increases with the fetch or upwind distance (x)

1) *Fetch and Developed Height Relationship* — The relation beween 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.

2) For buildings of heights greater than the developed height (h_x) in Table 6, the velocity profile may be determined in accordance with the following:

i) The less or least terrain; or

ii) The method described in Annex D.

Table 6 Fetch and Developed Height Relationship

[*Clause* 4.4.3.2 (b)]

Developed Height, h_x in m

Fetch (x) km	Terrain Category 1	Terrain Category 2	Terrain Category 3	Terrain Category 4
(1)	(2)	(3)	(4)	(5)
0.2	12	20	35	60
0.5	20	30	35	95
1	25	45	80	130
2	35	65	110	190
5	60	100	170	300
10	80	140	250	450
20	120	200	350	500
50	180	300	400	500

4.4.3.3 *Topography (k_3, factor)*

The basic wind speed V_b given in Fig. 1 takes account of the general level of the 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 will be significant at a site when the upwind slope is greater 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 greater than 3°. A method of evaluating the value of k_3 for slope greater than 3° is given in Annex E. 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.

4.4.4 *Design Wind Pressure*

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

$$p_z = 0.6\, V_z^2$$

where

p_z = design wind pressure in N/m² at height Z, and

V_z = design wind velocity in m/s at height Z.

NOTE — 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 appropriate Indian atmospheric conditions.

4.4.5 *Offshore Wind Velocity*

Cyclonic storms form far way 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.

4.5 Wind Pressure and Forces on Buildings/ Structure

4.5.1 *General*

The wind load on a building shall be calculated for:

a) the building as a whole;

b) individual structural elements as roofs and walls; and

c) individual cladding units including glazing and their fixings.

4.5.2 *Pressure Coefficients*

The pressure coefficients are always given for a particular surface or part of the surface of a building. 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 **4.5.2.2** and **4.5.2.3**.

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, 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 the different tables have been obtained mainly from measurements on models in wind tunnels, and the great majority of data available have been obtained in conditions of relatively smooth flow. Where sufficient field data exist 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 collapse of the 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 advise may be sought from specialists in the subject.

4.5.2.1 *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 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.

4.5.2.2 *External pressure coefficients*

a) *Walls* — The average external pressure coefficient for the walls of clad buildings of rectangular plan shall be as given in Table 7. In addition, local pressure concentration coefficients are also given.

b) *Pitched Roofs of Rectangular Clad Buildings* — The average external pressure coefficients and pressure concentration coefficients for pitched roofs of rectangular clad building shall be as given in Table 8. Where no pressure concentration coefficients are given, the average coefficients apply. The pressure coefficients on the underside of any over-hanging roof shall be taken in accordance with **4.5.2.2** (g).

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.

c) *Monoslope Roofs of Rectangular Load Buildings* — The average pressure coefficient and pressure concentration coefficient for monoslope (lean-to) roofs of rectangular clad buildings shall be as given in Table 9.

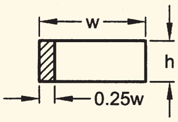
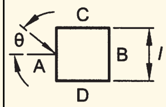
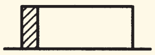
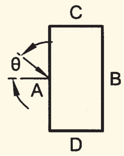
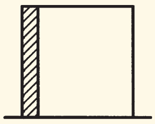
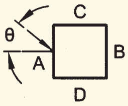
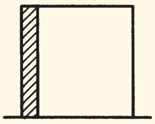
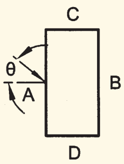
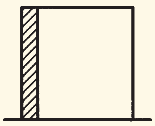
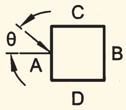
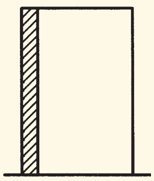
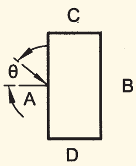
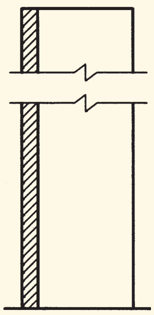
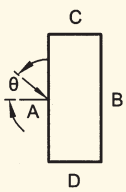
d) *Canopy Roofs with* ¼ < h/w < 1 and 1 < L/w < 3 — The pressure coefficients are given in Tables 10 and 11 separately for monopitch and double pitch canopy roofs, such as open-air parking garages, shelter areas, outdoor areas, railway platforms, stadiums and theatres. The coefficients take account 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 ø is equal to the area of obstruction under the canopy divided by the gross area under the canopy, both areas normal to the wind direction. ø = 0 represents a canopy with no obstructions underneath. ø = 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. Downwind of the position of maximum blockage the coefficients for ø = 0 may be used.

In addition to the pressure forces normal to

Table 7 External Pressure Coefficients (C_{pe}) for Walls of Rectangular Clad Buildings

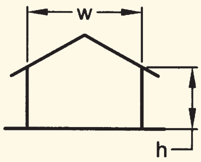
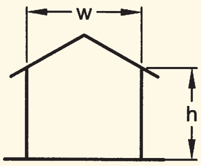
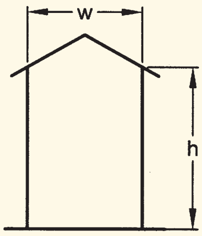
[*Clause* 4.5.2.2 (a)]

Building Height Ratio	Building Plan Ratio	Elevation	Plan	Wind Angle θ Degree	C_{pe} for Surface A	B	C	D	Local C_{pe}
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
$\frac{h}{w} \le \frac{1}{2}$	$1 < \frac{l}{w} \le \frac{3}{2}$			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
$\frac{1}{2} < \frac{h}{w} \le \frac{3}{2}$	$1 < \frac{l}{w} \le \frac{3}{2}$			0 90	+0.7 –0.6	–0.25 –0.6	–0.6 +0.7	–0.6 –0.25	–1.1
	$\frac{3}{2} < \frac{l}{w} < 4$			0 90	+0.7 –0.5	–0.3 –0.5	–0.7 +0.7	–0.7 –0.1	–1.1
$\frac{3}{2} < 1\frac{h}{w} < 6$	$1 < \frac{l}{w} \le \frac{3}{2}$			0 90	+0.8 –0.8	–0.25 –0.8	–0.8 +0.8	–0.8 –0.25	–1.2
	$\frac{3}{2} < \frac{l}{w} < 4$			0 90	+0.7 –0.5	–0.4 –0.5	–0.7 +0.8	–0.7 –0.1	–1.2
$\frac{h}{w} \ge 6$	$\frac{l}{w} = \frac{3}{2}$			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	–0.25 –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 of eaves or parapet, *l* is the greater horizontal dimension of a building and *w* is the lesser horizontal dimension of a building.

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

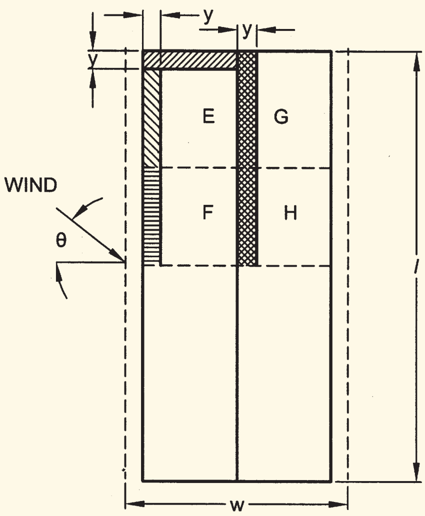
[*Clause* 4.5.2.2 (b)]

Building Height Ratio		Roof Angle degrees	Wind Angle θ 0° EF	Wind Angle θ 0° GH	Wind Angle θ 90° EG	Wind Angle θ 90° FH	Local Coefficients			
(1)		(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
$\frac{h}{w} \leq \frac{1}{2}$		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
$\frac{1}{2} < \frac{h}{w} \leq \frac{3}{2}$		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	–1.5	–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	–1.5	–1.2
		30	–1.0	–0.5	–0.8	–0.7	–1.5			
		40	–0.2	–0.5	–0.8	–0.7	–1.0			
		50	+0.2	–0.5	–0.8	–0.7				
		60	+0.5	–0.5	–0.8	–0.7				

NOTES

1 h is the height to caves or parapet, w is the lesser horizontal dimension of a building.

2 Where no local coefficients are given the overall coefficients apply.

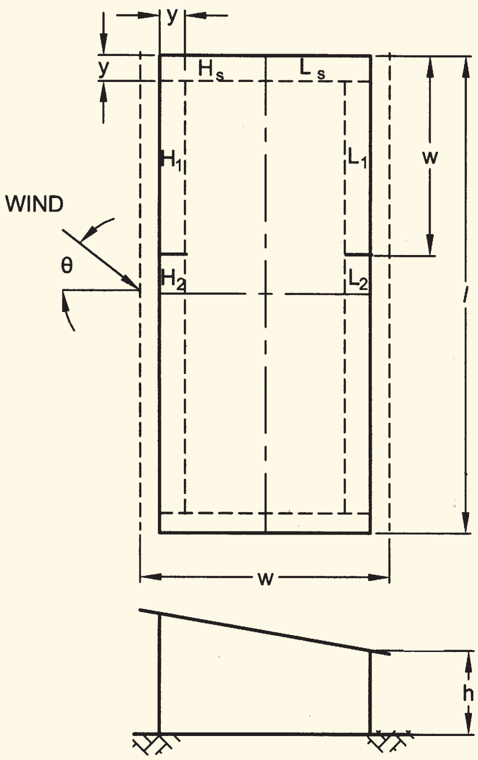


KEY PLAN

$y = h$ or $0.15\,w$,
whichever is the lesser

Table 9 External Pressure Coefficients (C_{pe}) for Monoslope Roofs for Rectangular

[*Clause* 4.5.2.2 (c)]



$y = h$ or $0.15\ w$, whichever is the lesser

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

OVERALL COEFFICIENTS

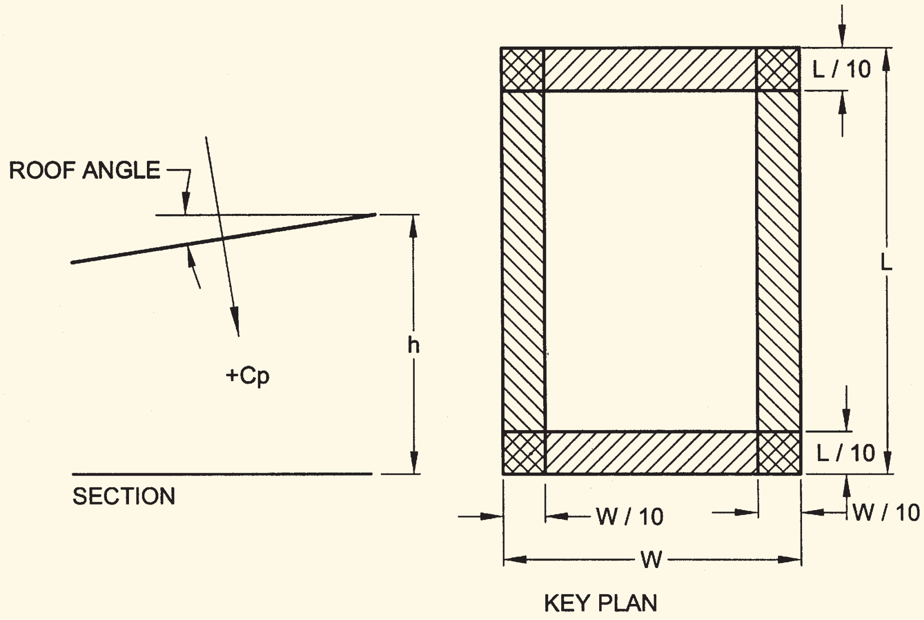
Roof Angle α	**Wind Angle θ**									
	0°		45°		90°		135°		180°	
Degree	*H*	*L*	*H*	*L*	*H & L*	*H & L*	*H*	*L*	*H*	*L*
					Applies to length *w*/2 from wind ward end	Applies to remainder				
5	–1.0	–0.5	–1.0	–0.9	–1.0	–0.5	–0.9	–1.0	–0.5	–1.0
10	–1.0	–0.5	–1.0	–0.8	–1.0	–0.5	–0.8	–1.0	–0.4	–1.0
15	–0.9	–0.5	–1.0	–0.7	–1.0	–0.5	–0.6	–1.0	–0.3	–1.0
20	–0.8	–0.5	–1.0	–0.6	–0.9	–0.5	–0.5	–1.0	–0.2	–1.0
25	–0.7	–0.5	–1.0	–0.6	–0.8	–0.5	–0.3	–0.9	–0.1	–0.9
30	–0.5	–0.5	–1.0	–0.6	–0.8	–0.5	–0.1	–0.6	0	–0.6

Roof Angle α	**LOCAL COEFFICIENTS C_{pe}**					
Degree	H_1	H_2	L_1	L_2	*H*	*L*
5	–2.0	–1.5	–2.0	–1.5	–2.0	–2.0
10	–2.0	–1.5	–2.0	–1.5	–2.0	–2.0
15	–1.8	–0.9	–1.8	–1.4	–2.0	–2.0
20	–1.8	–0.8	–1.8	–1.4	–2.0	–2.0
25	–1.8	–0.7	–0.9	–0.9	–2.0	–2.0
30	–1.8	–0.5	–0.5	–0.5	–2.0	–2.0

NOTE — *h* is the height to eaves at lower side, *l* is greater horizontal dimension of a building and *w* is the lesser horizontal dimension of a building.

Table 10 Pressure Coefficients for Free Standing Monosloped Roofs

[*Clause* 4.5.2.2 (d)]

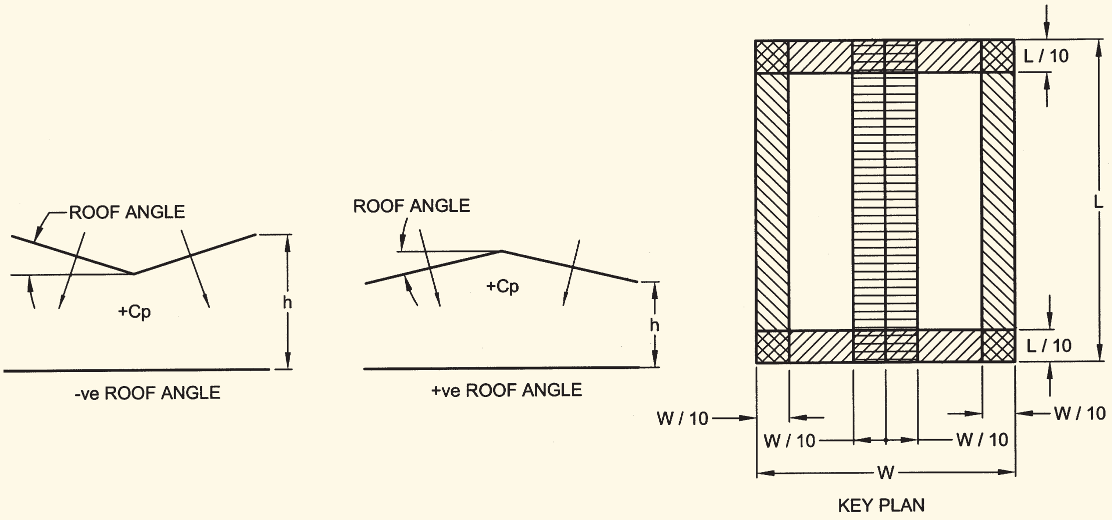


Roof Angle Degree	**Solidity Ratio**	**Maximum (Largest +ve) and Minimum (Largest –ve) Pressure Coefficients**			
		Overall coefficients	Local coefficients		
0	All values of Ø	+0.2	+0.5	+1.8	+1.1
5		+0.4	+0.8	+2.1	+1.3
10		+0.5	+1.2	+2.4	+1.6
15		+0.7	+1.4	+2.7	+1.8
20		+0.8	+1.7	+2.9	+2.1
25		+1.0	+2.0	+3.1	+2.3
30		+1.2	+2.2	+3.2	+2.4
0	Ø = 0	–0.5	–0.6	–1.3	–1.4
	Ø = 1	–1.0	–1.2	–1.8	–1.9
5	Ø = 0	–0.7	–1.1	–1.7	–1.8
	Ø = 1	–1.1	–1.6	–2.2	–2.3
10	Ø = 0	–0.9	–1.5	–2.0	–2.1
	Ø = 1	–1.3	–2.1	–2.6	–2.7
15	Ø = 0	–1.1	–1.8	–2.4	–2.5
	Ø = 1	–1.4	–2.3	–2.9	–3.0
20	Ø = 0	–1.3	–2.2	–2.8	–2.9
	Ø = 1	–1.5	–2.6	–3.1	–3.2
25	Ø = 0	–1.6	–2.6	–3.2	–3.2
	Ø = 1	–1.7	–2.8	–3.5	–3.5
30	Ø = 0	–1.8	–3.0	–3.8	–3.6
	Ø = 1	–1.8	–3.0	–3.8	–3.6

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

Table 11 Pressure Coefficients for Free Standing Double Sloped Roofs

[*Clause* 4.5.2.2 (d)]



Roof Angle Degree	Solidity Ratio	Maximum (Largest +ve) and Minimum (Largest –ve) Pressure Coefficients				
		Overall coefficients	Local coefficients			
–20		+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	All values of Ø	+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	Ø = 0	–0.7	–0.9	–1.3	–1.6	–0.6
	Ø = 1	–0.9	–1.2	–1.7	–1.9	–1.2
–15	Ø = 0	–0.6	–0.8	–1.3	–1.6	–0.6
	Ø = 1	–0.8	–1.1	–1.7	–1.9	–1.2
–10	Ø = 0	–0.6	–0.8	–1.3	–1.5	–0.6
	Ø = 1	–0.8	–1.1	–1.7	–1.9	–1.3
–5	Ø = 0	–0.5	–0.7	–1.3	–1.6	–0.6
	Ø = 1	–0.8	–1.5	–1.7	–1.9	–1.4
+5	Ø = 0	–0.6	–0.6	–1.4	–1.4	–1.1
	Ø = 1	–0.9	–1.3	–1.8	–1.8	–2.1
+10	Ø = 0	–0.7	–0.7	–1.5	–1.4	–1.4
	Ø = 1	–1.1	–1.4	–2.0	–1.8	–2.4
+15	Ø = 0	–0.8	–0.9	–1.7	–1.4	–1.8
	Ø = 1	–1.2	–1.5	–2.2	–1.9	–2.8
+20	Ø = 0	–0.9	–1.2	–1.8	–1.4	–2.0
	Ø = 1	–1.3	–1.7	–2.3	–1.9	–3.0
+25	Ø = 0	–1.0	–1.4	–1.9	–1.4	–2.0
	Ø = 1	–1.4	–1.9	–2.4	–2.1	–3.0
+30	Ø = 0	–1.0	–1.4	–1.9	–1.4	–2.0
	Ø = 1	–1.4	–2.1	–2.6	–2.2	–3.0

NOTE — 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 duopitch canopies the centre of pressure should be taken to act at the centre of each slope.

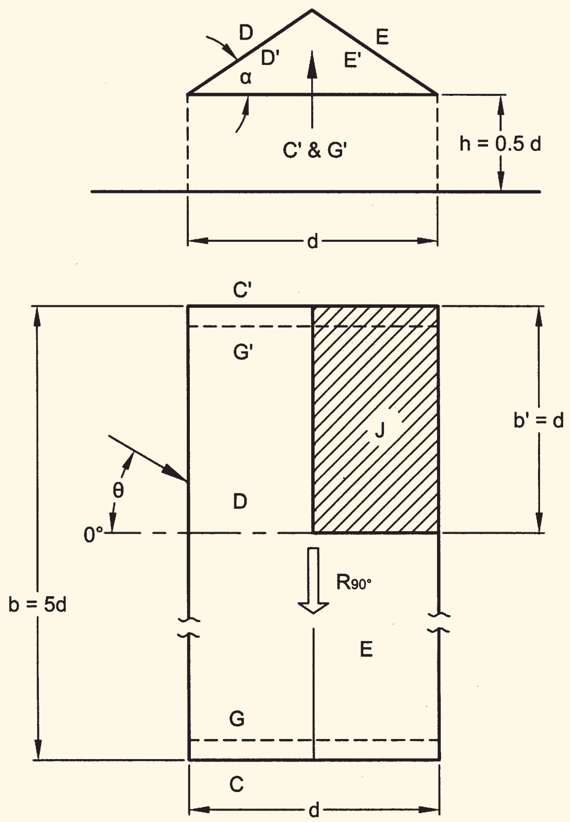
the canopy, there will be horizontal loads on the canopy due to the wind pressure on any fascia and to friction over the surface of the canopy. For any wind direction, only the greater of these two forces need be taken into account. Fascia loads should be calculated on the area of the surface facing the wind, using a force coefficient of 1.3. Frictional drag should be calculated using the coefficients given in **4.5.3.1**.

NOTE — Tables 12 to 17 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 Tables 12 to 17 any significant departure from it should be investigated carefully. No increase shall be made for local effects except as indicated.

e) *Curved Roofs* — For curved roofs, the external pressure coefficients shall be as given in Table 18. Allowance for local effects shall be made in accordance with Table 8.

Table 12 Pressure Coefficients (Top and Bottom) for Pitched Roofs, α = 30°

[*Clause* 4.5.2.2 (d)]



α = 30°

θ = 0° – 45°, *D, D´, E, E´* full length

θ = 90°, *D, D´, E, E´* part length *b´*

PRESSURE COEFFICIENTS, C_p

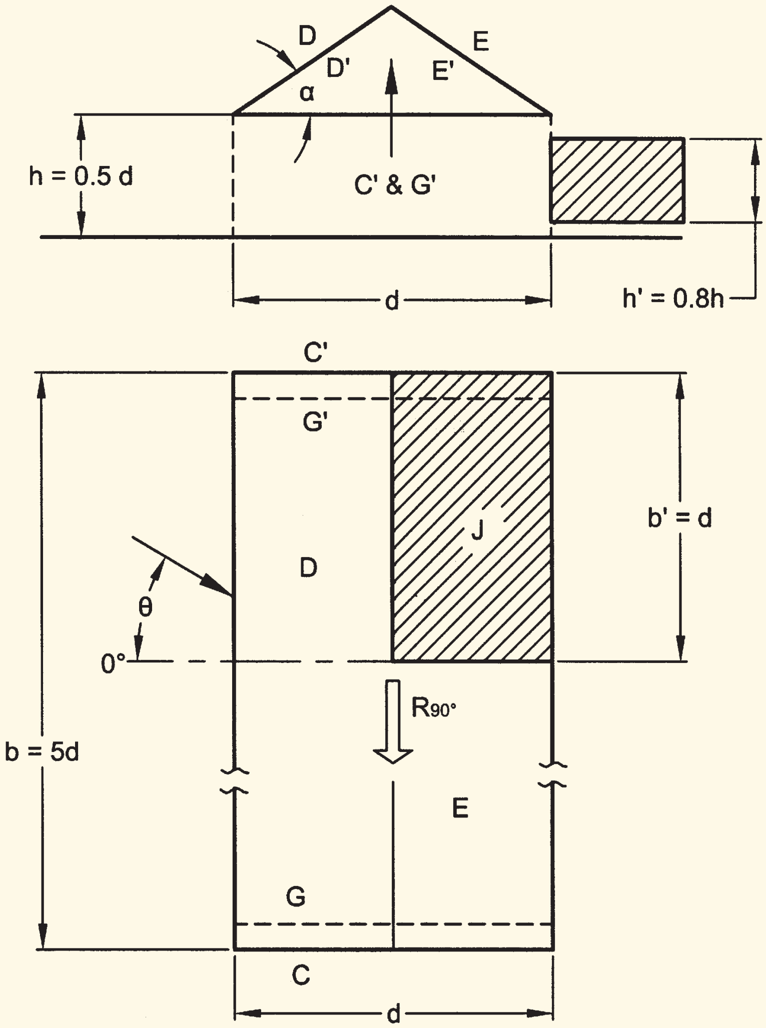
θ	D	D'	E	E'	End Surfaces			
					C	C'	G	G'
0°	0.6	–1.0	–0.5	–0.9	—	—	—	—
45°	0.1	–0.3	–0.6	–0.3	—	—	—	—
90°	–0.3	–0.4	–0.3	–0.4	–0.3	0.8	0.3	–0.4

45° For *j*: C_p top = –1.0; C_p bottom = –0.2.

90° Tangentially acting friction: $R_{90°} = 0.05\ p_d.db$.

Table 13 Pressure Coefficients (Top and Bottom) for Pitched Free Roofs, $\alpha = 30^\circ$ with Effects of Train or Stored Materials

[*Clause* 4.5.2.2 (d)]



α = 30°

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´*

PRESSURE COEFFICIENTS, C_p

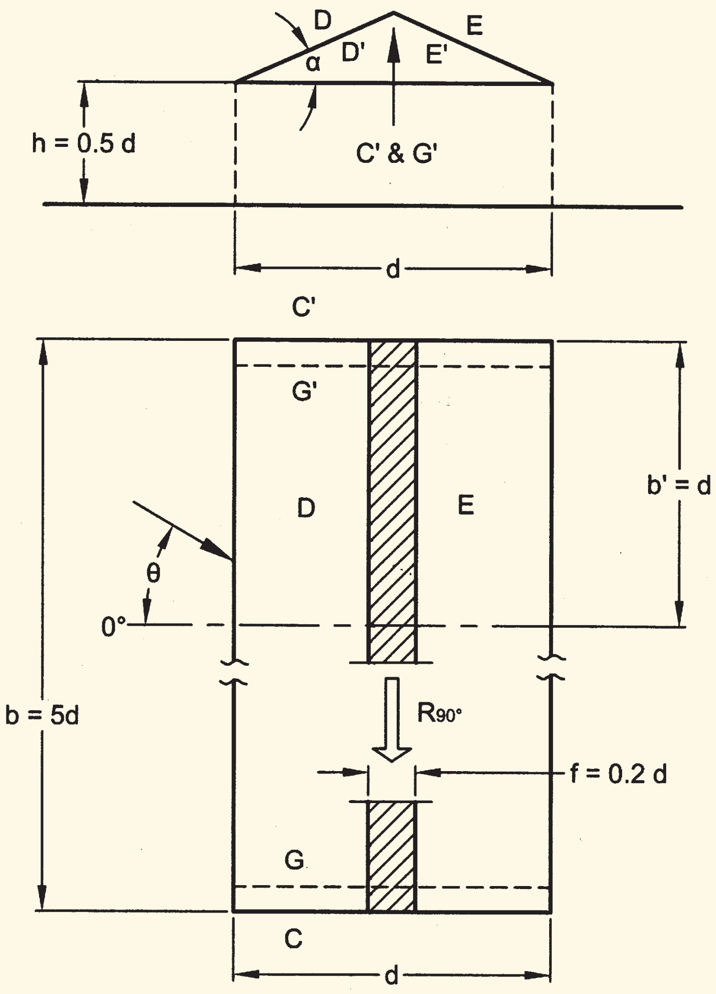
θ	D	D'	E	E'	End Surfaces			
					C	C'	G	G'
0°	0.1	0.8	–0.7	0.9	—	—	—	—
45°	–0.1	0.5	–0.8	0.5	—	—	—	—
90°	–0.4	–0.5	–0.4	–0.5	–0.3	0.8	0.3	–0.4
180°	–0.3	0.6	0.4	–0.6	—	—	—	—

45° For *j*: C_p top = –1.5; C_p bottom = 0.5.

90° Tangentially acting friction: $R_{90^\circ} = 0.05\, p_d.db$.

Table 14 Pressure Coefficients (Top and Bottom) for Pitched Free Roofs, α = 10°

[*Clause* 4.5.2.2 (d)]



α = 10°
θ = 0° – 45°, *D, D´, E, E´* full length
θ = 90°, *D, D´, E, E´* part length *b´*

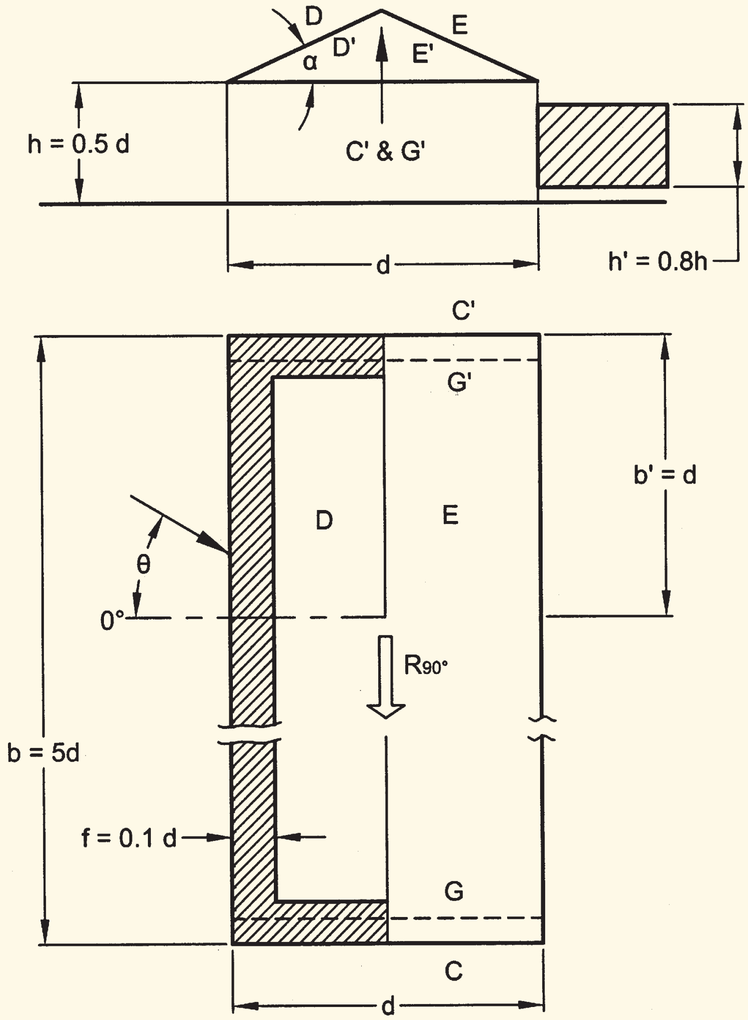
PRESSURE COEFFICIENTS, C_p

θ	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	–0.3	0.0	–0.4	0.8	0.3	–0.6

0° For *f*: C_p top = –1.0; C_p bottom = 0.40.
0-90° Tangentially acting friction: $R_{90°} = 0.1\, p_d.db$.

**Table 15 Pressure Coefficients (Top and Bottom) for Pitched Free Roofs, α = 10°
with Effects of Train or Stored Materials**

[*Clause* 4.5.2.2 (d)]



α = 10°

Effects of trains or stored materials:

θ = 0° – 45°, 135° – 180°, *D, D´, E, E´* full length

θ = 90°, *D, D´, E, E´* part length *b´*

PRESSURE COEFFICIENTS, C_p

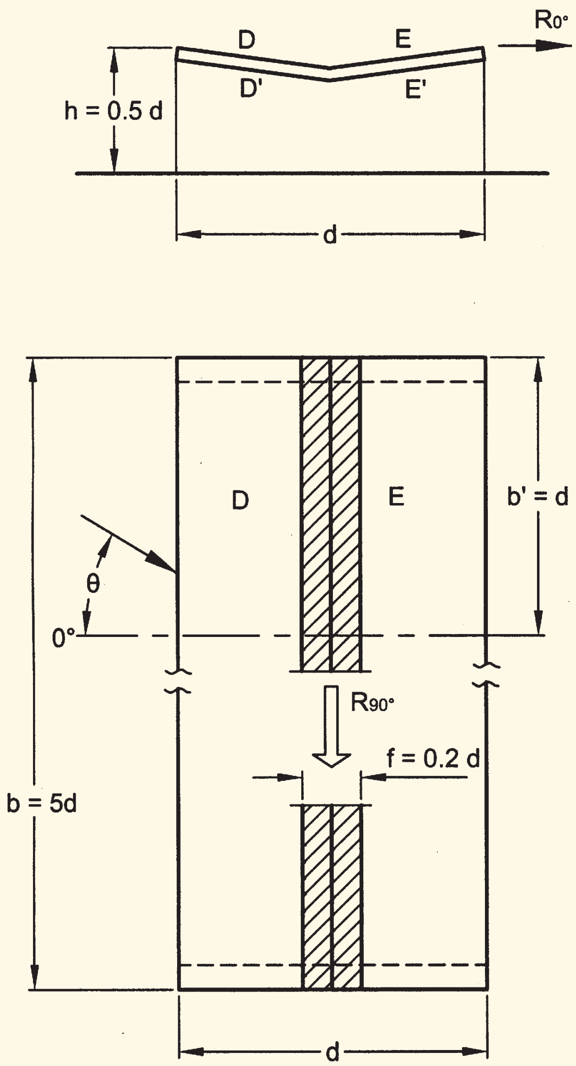
θ	*D*	*D′*	*E*	*E′*	End Surfaces			
					C	*C′*	*G*	*G′*
0°	–1.3	0.8	–0.6	0.7	—	—	—	—
45°	0.5	0.4	–0.3	0.3	—	—	—	—
90°	–0.3	0.0	–0.3	0.0	—	—	—	—
180°	–0.4	–0.3	–0.6	–0.3	–0.4	0.8	0.3	–0.6

0° For *f*: C_p top = –1.6; C_p bottom = 0.9.

0° – 180° Tangentially acting friction: $R_{90°} = 0.1\, p_d.db$.

Table 16 Pressure Coefficients for Troughed Free Roofs, α = 10°

[*Clause* 4.5.2.2 (d)]



Roof slope α = 10°

θ = 0° – 45°, *D, D′, E, E′* full length

θ = 90°, *D, D′, E, E′* part length *b′*

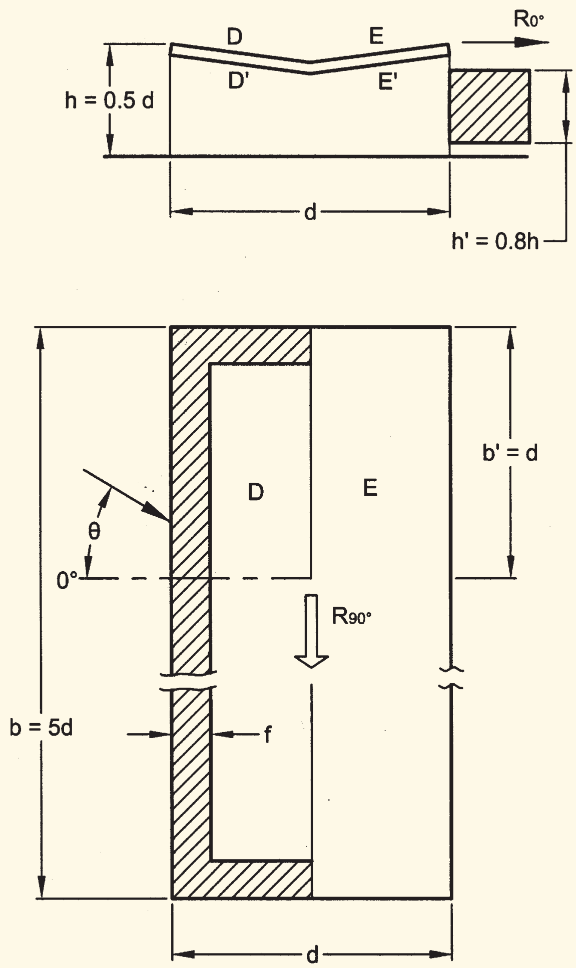
θ	PRESSURE COEFFICIENTS, C_p			
	D	*D′*	*E*	*E′*
0°	0.3	–0.7	0.2	–0.9
45°	0.0	–0.2	0.1	–0.3
90°	–0.1	0.1	–0.1	0.1

0° For *f*: C_p top = 0.4; C_p bottom = 1.5.

0° – 90° Tangentially acting friction: $R_{90°} = 0.1\ p_d.bd$.

Table 17 Pressure Coefficients for Troughed Free Roofs, α = 10° with Effects of Trains or Stored Materials

[*Clause* 4.5.2.2 (d)]



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′*

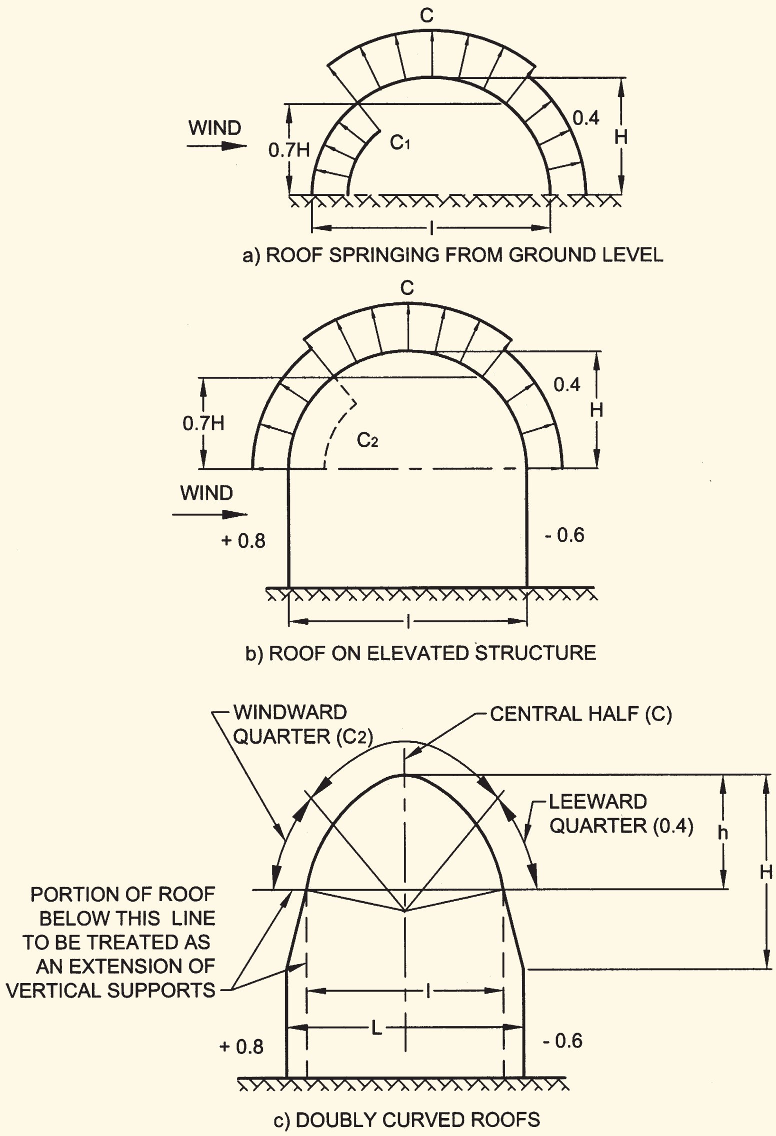
θ	PRESSURE COEFFICIENTS, C_p			
	D	*D′*	*E*	*E′*
0°	–0.7	0.8	–0.6	0.6
45°	–0.4	0.3	–0.2	0.2
90°	–0.1	0.1	–0.1	0.1
180°	–0.4	–1.2	–0.6	–0.3

0° For *f*: C_p top = 1.1; C_p bottom = 0.9.

0° – 180° Tangentially acting friction: $R_{90°} = 0.1\ p_d.bd$.

Table 18 External Pressure Coefficients for Curved Roofs

[*Clause* 4.5.2.2 (e)]



$$\frac{\bar{H}}{l} \ngtr 0.6 \text{ and } \frac{h}{l} > 0.6$$

Values of C, C_1 and C_2

H/l	*C*	C_1	C_2
0.1	–0.8	+0.1	–0.8
0.2	–0.9	+0.3	–0.7
0.3	–1.0	+0.4	–0.3
0.4	–1.1	+0.6	+0.4
0.5	–1.2	+0.7	+0.7

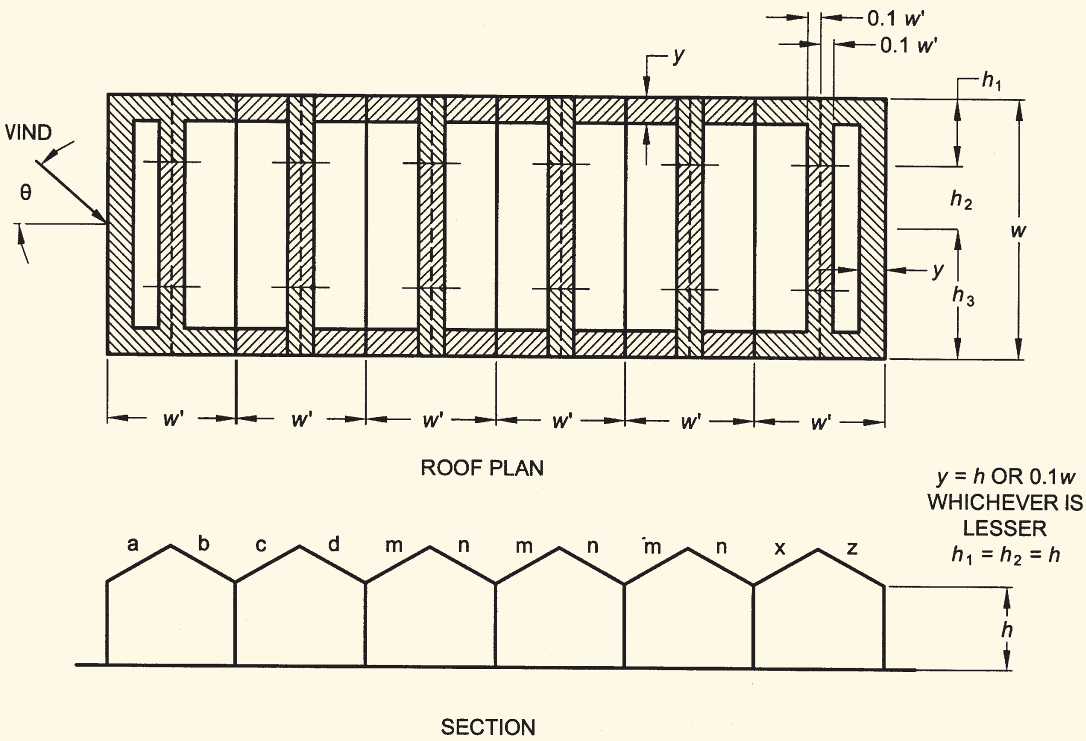
NOTE — When the wind is blowing normal to the gable ends, C_{pe} may be taken as equal to –0.7 for the full width of the roof over a length of *l*/2 from the gable ends and –0.5 for the remaining portion.

f) *Pitched and Saw-Tooth Roofs of Multi-span Buildings* — For pitched and saw-tooth roofs of multi-span buildings, the external average pressure coefficients and pressure concentration coefficients shall be as given in Tables 19 and 20 respectively, provided that all spans shall be equal and the height to the eaves shall not exceed the span.

NOTE — Evidence on multi-span buildings is fragmentary. Any departure given in Tables 19 and 20 should be investigated separately.

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

[*Clause* 4.5.2.2 (f)]



Roof Angle degrees	Wind Angle degrees	First Span	First Intermediate Span			Other Intermediate Span		End Span		Local Coefficient	
α	θ	*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		

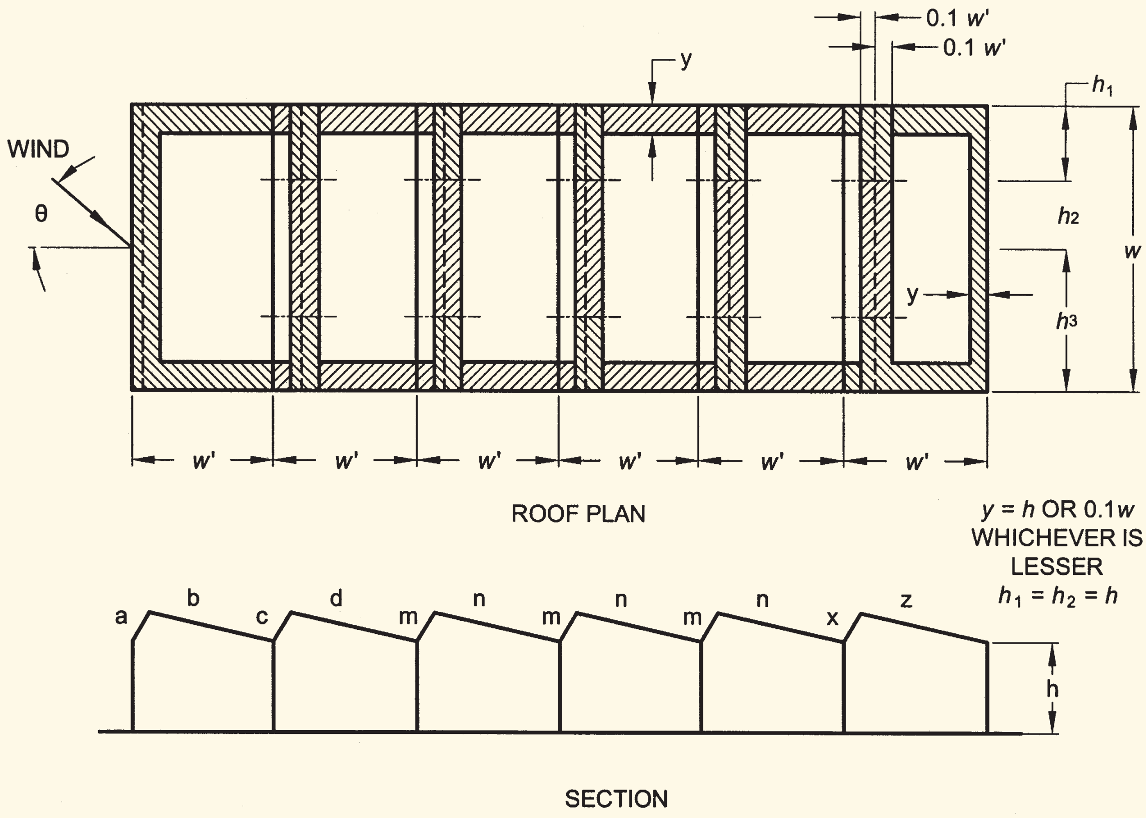
Roof Angle degrees	Wind Angle degrees	Distance		
α	θ	h_1	h_2	h_3
Up to 45	90	–0.8	–0.6	–0.2

Frictional drag: when wind angle $\theta = 0°$ horizontal forces due to frictional drag are allowed for in the above values; when wind angle $\theta = 90°$ allow for frictional drag in accordance with **4.5.3.1**.

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

Table 20 External Pressure Coefficients (C_{pe}) for Saw-Tooth Roofs of Multispan Buildings (All Spans Equal) with $h > w'$

[*Clause* 4.5.2.2 (f)]



Wind Angle degrees θ	First Span		First Intermediate Span		Other Intermediate Span		End Span		Local Coefficient	
	a	*b*	*c*	*d*	*m*	*n*	*x*	*z*	(hatched ▧)	(hatched ▨)
0	+0.6	–0.7	–0.7	–0.4	–0.3	–0.2	–0.1	–0.3	–2.0	–1.5
180	–0.5	–0.3	–0.3	–0.3	–0.4	–0.6	–0.6	–0.1		

Wind Angle degrees θ	Distance h_1	h_2	h_3
90	–0.8	–0.6	–0.2
270	Similarly, but handed		

Frictional drag: when wind angle $\theta = 0^\circ$ horizontal forces due to frictional drag are allowed for in the above values; when wind angle $\theta = 90^\circ$ allow for frictional drag in accordance with **4.5.3.1**.

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

g) *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-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:

1) 1.25, if the overhanging slopes; downwards;
2) 1.0, if the overhanging is horizontal; and
3) 0.75, if the overhanging slopes upwards.

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

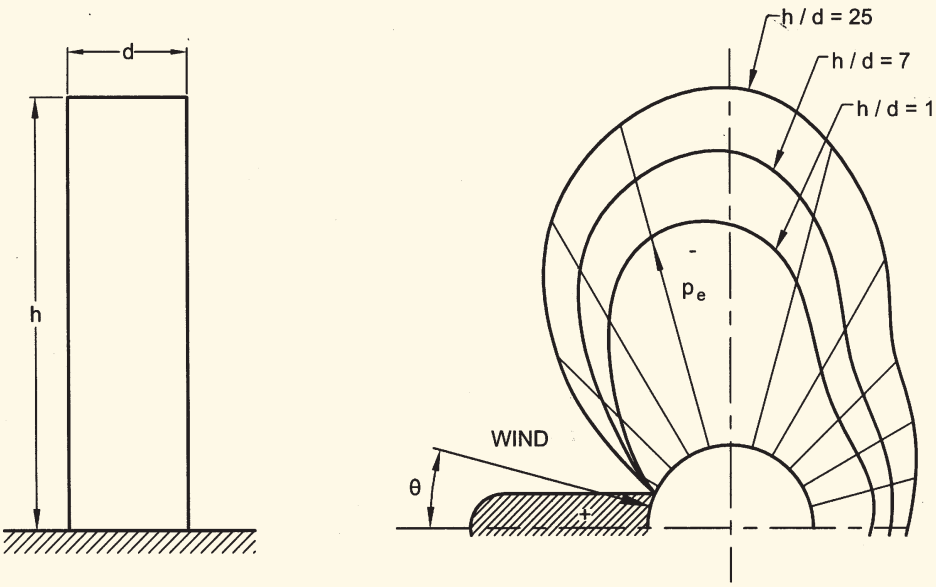
h) *Cylindrical Structures* — For the purpose of calculating the wind pressure distribution around a cylindrical structure of circular cross-section, the value of external pressure coefficients given in Table 21 may be used provided that the Raynolds number is greater than 10 000. They may be used for wind blowing normal to the axes of cylinders having axis normal to the ground plane (that is, chimneys and silos) and cylinders having their axis parallel to the ground plane (that is, horizontal tanks) provided that the clearance between the tank and the ground is not less than the diameter of the cylinder.

h is the 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.

1) –0.8, where *h*/*D* is not less than 0.3; and
2) –0.5, where *h*/*D* is less than 0.3.

Table 21 External Pressure Distribution Coefficients Around Cylindrical Structures

[*Clause* 4.5.2.2 (h)]



Position of Periphery, θ in degrees	Pressure Coefficient, 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	–0.5
180	–0.6	–0.5	–0.4

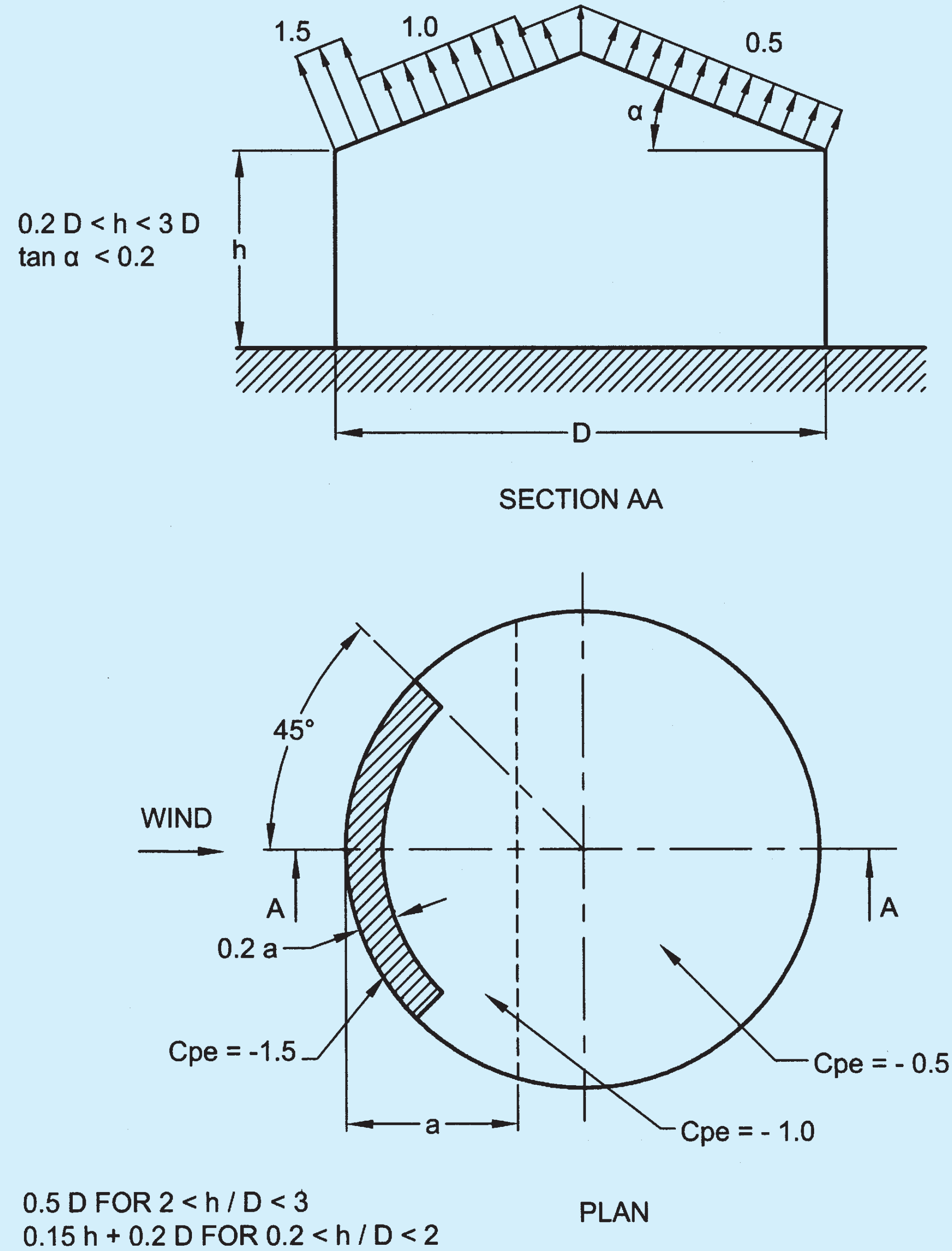
j) *Roofs and Bottom of Cylindrical Related Structure* — The external pressure coefficients for roofs and bottoms of cylindrical elevated structures shall be as given in Table 22 (*see also* Fig. 2).

The total resultant load (P) acting on the roof of the structure is given by the following formula:

$$P = 0.785\, D^2 \,(C_{pi} - C_{pe})\, p_d$$

The resultant of P for roofs lies at 0.1 D from the centre of the roof on the windward side.

k) *Combined Roofs and Roofs with a Sky Light* — The average external pressure coefficients for combined roofs and roofs with a sky light are shown in Table 23.

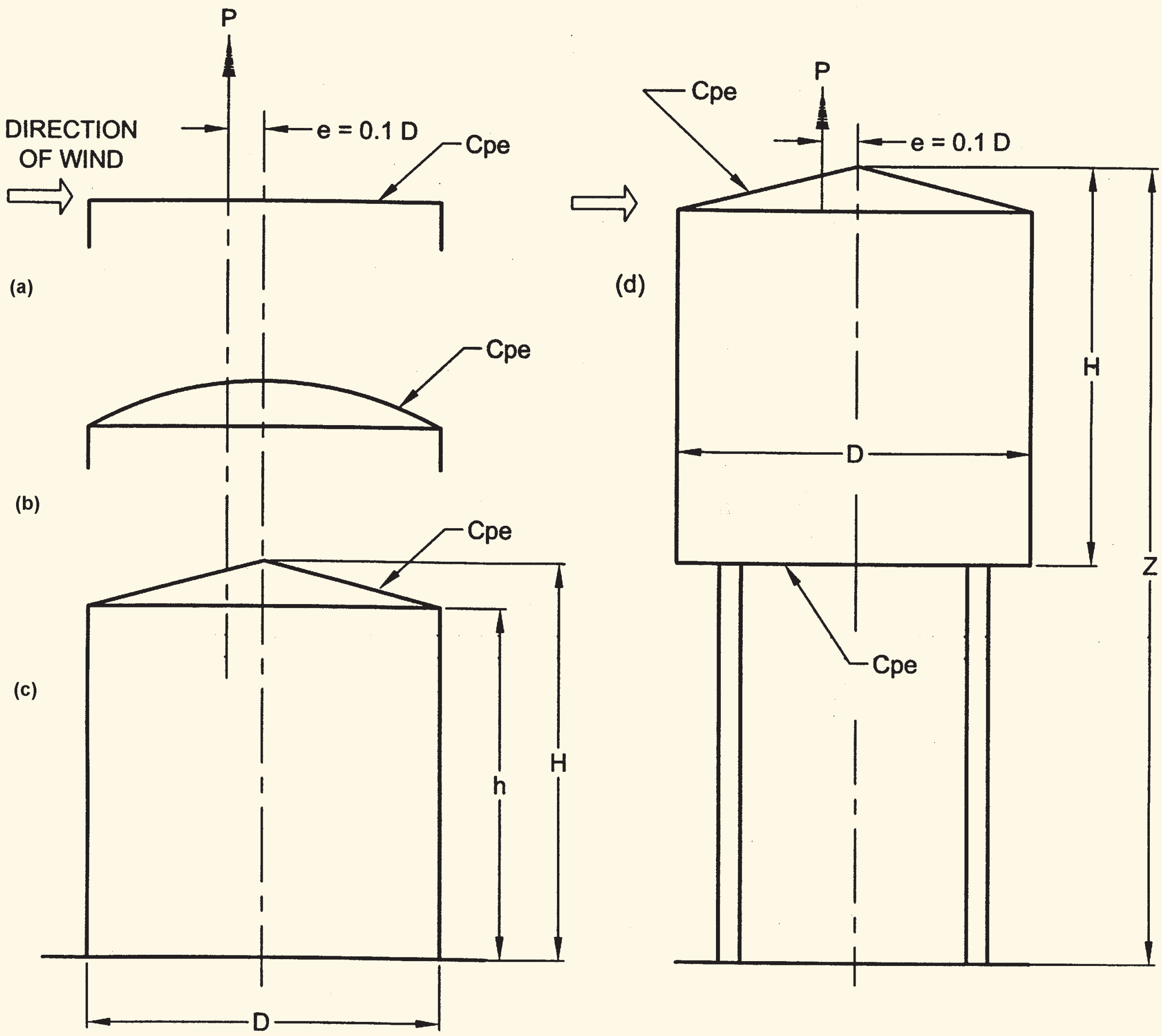


(For force coefficient corresponding to shell portion *see* Table 23)

FIG. 2 EXTERNAL PRESSURE COEFFICIENTS ON THE UPPER ROOF SURFACE OF SINGULAR CIRCULAR STANDING ON THE GROUND

Table 22 External Pressure Coefficients for Roofs and Bottoms of Cylindrical Structures

[*Clause* 4.5.5.2 (j)]



Coefficient 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
1.0	–1.00	1.25	–0.75	–0.7
2.0	–1.00	1.50	–0.75	–0.6

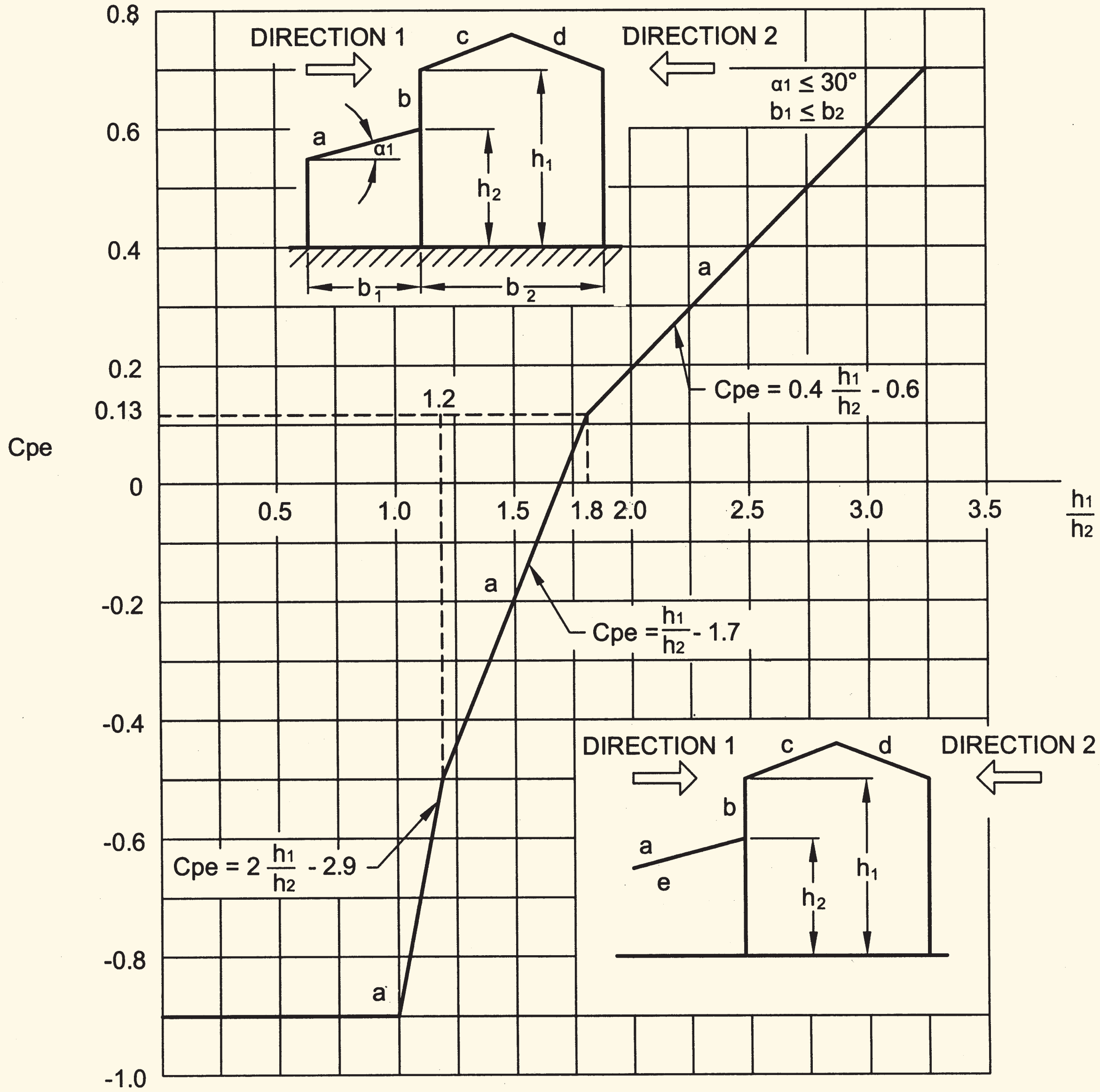
Total force acting on the roof of the structure, $P = 0.785\ D^2\ (C_{pi} - C_{pe})p_d$.

The resultant of P lies eccentrically, $e = 0.1\ D$.

Table 23 External Pressure Coefficients, C_{pe} for Combined Roofs and Roofs with a Sky Light

[*Clause* 4.5.2.2 (k)]

a) Combined Roofs

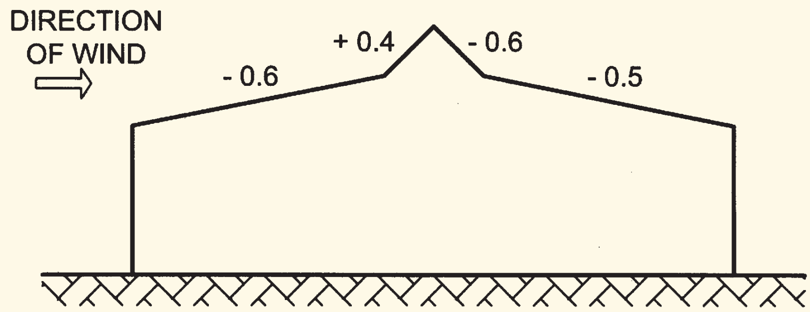


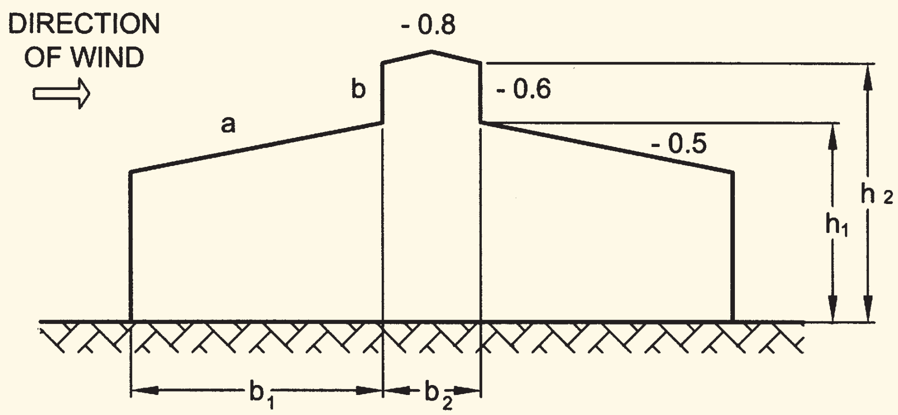
Values of C_{pe}

Portion	Direction 1	Direction 2
a	From the Diagram	
	$C_{pe} = -0.5$, $h_1/h_2 \leq 1.5$	–0.4
b	$C_{pe} = -0.7$, $h_1/h_2 > 1.5$	
c and *d*	*See* Table 5	
e	*See* **4.5.2.2** (g)	

Table 23 — *Concluded*

b) Roofs with a Sky Light





	$b_1 > b_2$		$b_1 \leq b_2$
Portion	*a*	*b*	*a* & *b*
C_{pe}	-0.6	+0.7	*See* Table for Combined Roofs

m) *Grandstands* — The pressure coefficients on the roof (top and bottom) and rear wall of a typical grandstand roof, which is open on three sides, is given in Table 24. 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.

n) *Upper Surface of Round Silos and Tanks* — The pressure coefficients on the upper surface of round silos and tanks standing on ground shall be as given in Fig. 2.

p) *Spheres* — The external pressure coefficients for spheres shall be as given in Table 25.

4.5.2.3 *Internal pressure coefficients*

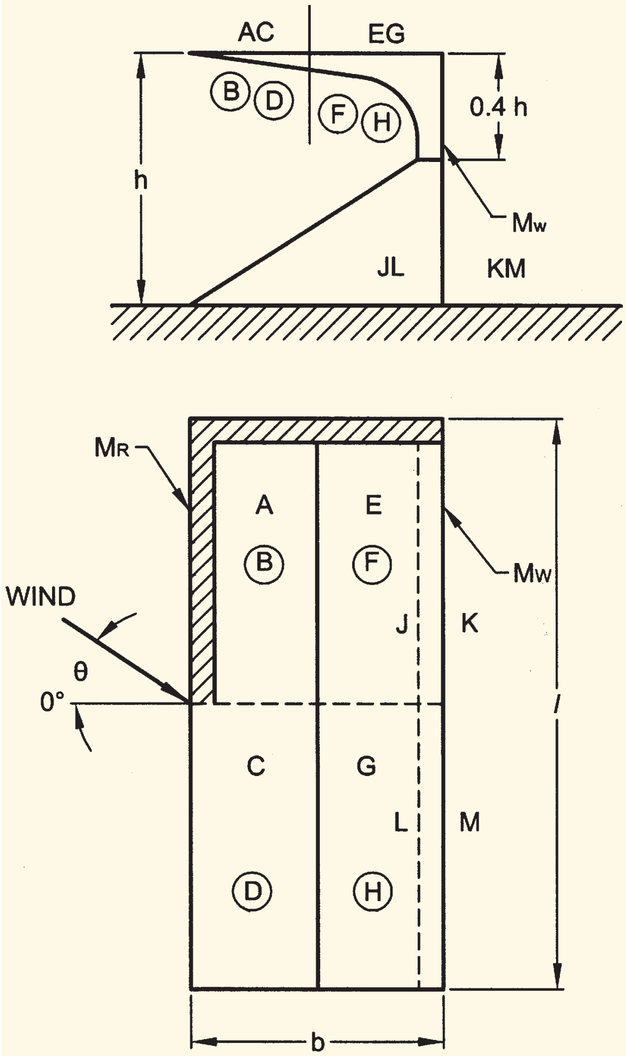
Internal air pressure in a building depends upon the degree of permeability of the cladding to the flow of air. The internal air pressure may be positive or negative depending on the direction of flow of air in relation to the openings in the buildings.

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 areas but where there are no large openings, it is necessary to consider the possibility of the internal pressure being positive or negative.

Table 24 Pressure Coefficients at Top and Bottom Roof of Grandstands Open Three Sides (Roof = 5°)

[*Clause* 4.5.2.2 (m)]

($h:b:l$ = 0.8: 1: 2.2)



(Shaded area to scale)

Front and Back of Wall

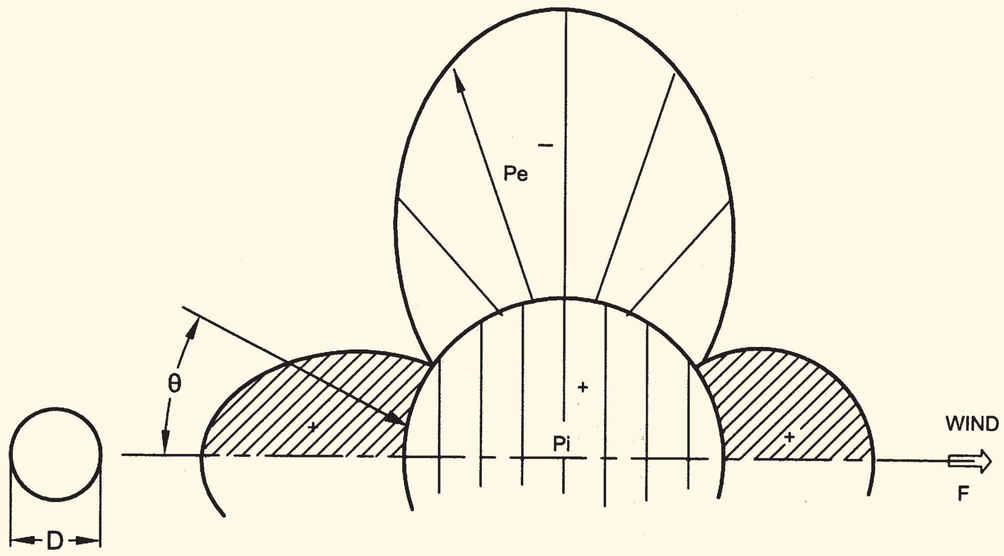
θ	J	K	L	M
0°	+0.9	–0.5	+0.9	–0.5
45°	+0.8	–0.6	+0.4	–0.4
135°	–1.1	+0.6	–1.0	+0.4
180°	–0.3	+0.9	–0.3	+0.9
60°	$M_w - C_p$ of $K = -1.0$			
60°	$M_w - C_p$ of $J = +1.0$			

Top and Bottom of 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°	$M_R - C_p$ (top) = –2.0							
45°	$M_R - C_p$ (bottom) = +1.0							

Table 25 External Pressure Distribution Coefficients Around Spherical Structures

[*Clause* 4.5.2.2 (p)]



Position of Periphery, θ in degrees	C_{pe}	Remarks
0	+1.0	
15	+0.9	C_f = 0.5 for $DV_z < 7$
30	+0.5	= 0.2 for $DV_z \geq 7$
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	

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 the external pressure will at once indicate the proper sign of the internal pressure coefficient to be taken for design.

NOTE — The terms normal permeability relates to the flow of air commonly afforded by the claddings not only through the 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 the area of the walls having the openings.

b) *Building with medium and large openings* — Buildings with medium and large openings may also exhibit either positive or negative internal pressure depending upon the direction of wind. Buildings with medium openings between about 5 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 members shall be adopted. Buildings 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 on the members shall be adopted.

Buildings with one open side or openings exceeding 20 percent of wall area may be assumed to be subjected to internal positive pressure or suction similar to those for buildings with large openings. A few examples of buildings with one sided openings are shown in Fig. 3 indicating values of internal pressure coefficients with respect to direction of wind.

c) In buildings with roofs but no walls, the roofs will be subjected to pressure from both inside and outside, and the recommendations shall be as given in **4.5.2.2**.

4.5.3 *Force Coefficients*

The value of force coefficients 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 by design wind pressure, p_d give the total wind load on that particular building or structure.

$$F = C_f A_e p_d$$

where F is the force acting in a direction specified in the respective tables and C_f is the 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

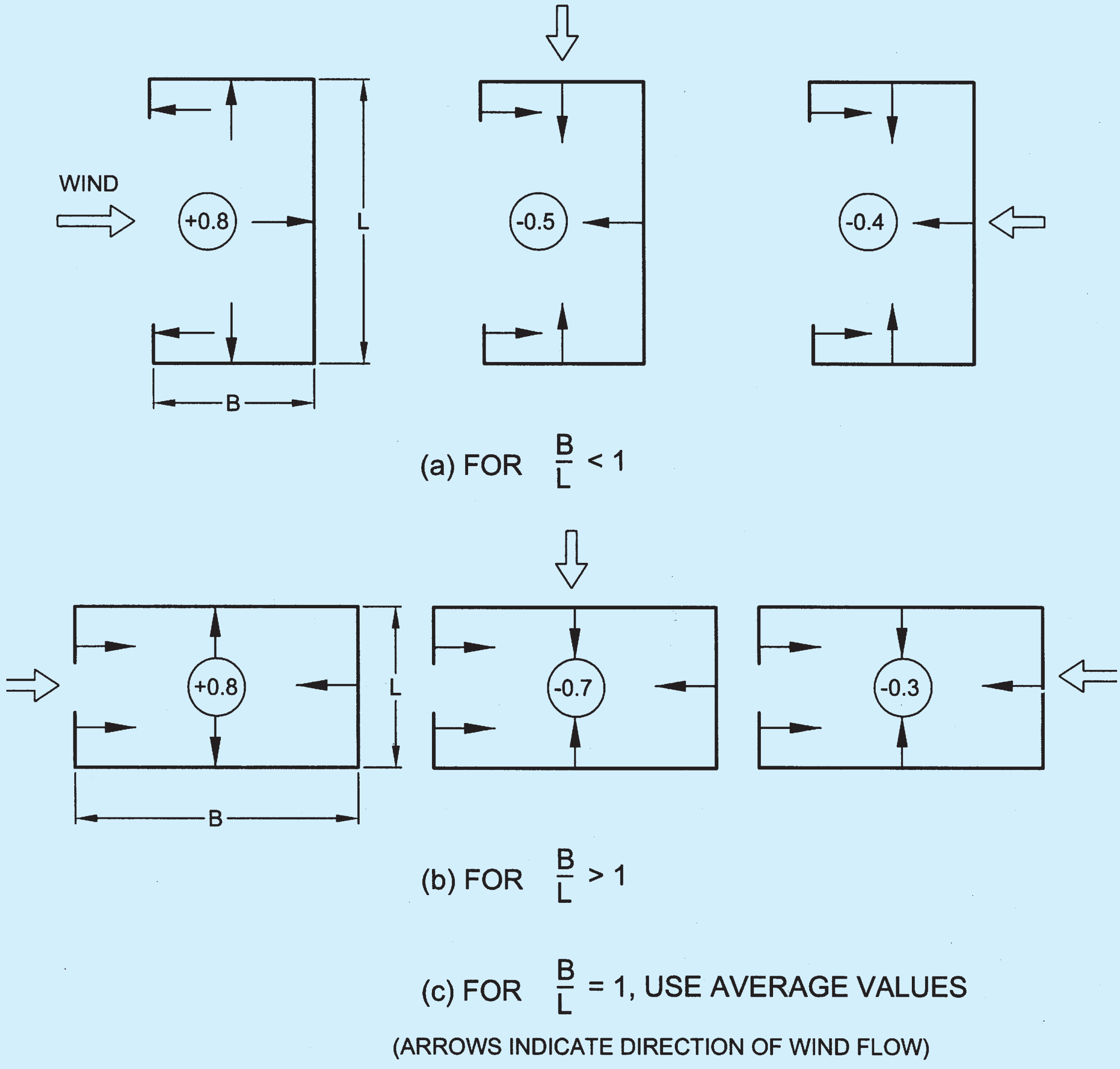


FIG. 3 LARGE OPENING IN BUILDINGS (VALUES OF COEFFICIENT OF INTERNAL PRESSURE) (WITH TOP CLOSED)

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

4.5.3.1 *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 **4.5.2**. For rectangular clad buildings, this addition is necessary only where the ratio *d*/*h* or *d*/*b* is greater than 4. The frictional drag force, F' in the direction or the wind given by the following formulae:

$$\text{If } h \leq b,\ F' = C'_f\,(d-4h)\,bp_d + C'_f\,(d-4h)\,2hp_d$$

$$\text{or If } h > b,\ F' = C'_f\,(d-4b)\,bp_d + C'_f\,(d-4b)\,2hp_d$$

The first term in each case gives the drag on the roof and the second on the walls. The value of C'_f has the following values:

C'_f = 0.01 for smooth surfaces without corrugations or ribs across the wind direction;

C'_f = 0.02 for surfaces with corrugations or ribs across the wind direction;

C'_f = 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.

4.5.3.2 *Force coefficients for clad buildings*

a) *Clad buildings of uniform section* — 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 26.

b) *Buildings of circular shapes* — Force coefficients for buildings of circular cross-section shall be as given in Table 27 (*see* Fig. 5 and Annex F).

c) *Low walls and hoardings* — Force coefficients for low walls and hoardings less than 15 m high shall be as given in Table 27 provided the height shall be measured from the ground to the top of the walls or hoarding, and provided that for walls or hoardings above the ground the clearance between the wall or hoarding and the ground shall be not less than 0.25 times the vertical dimension of the wall or hoarding.

   To allow for oblique winds the design shall also be checked for the net pressure normal to the surface varying linearly from a maximum of 1.7 C_f at the up wind edge to 0.44 C_f at the down wind 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 the shielding effects of one element or another.

d) *Solid circular shapes mounted on a surface* — The force coefficients for solid circular shapes mounted on a surface shall be as given in Fig. 6.

4.6 Dynamic Effects

4.6.1 *General*

Flexible slender structures and structural elements shall be investigated to ascertain the importance of wind induced oscillations for excitations along and across the direction of wind.

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.

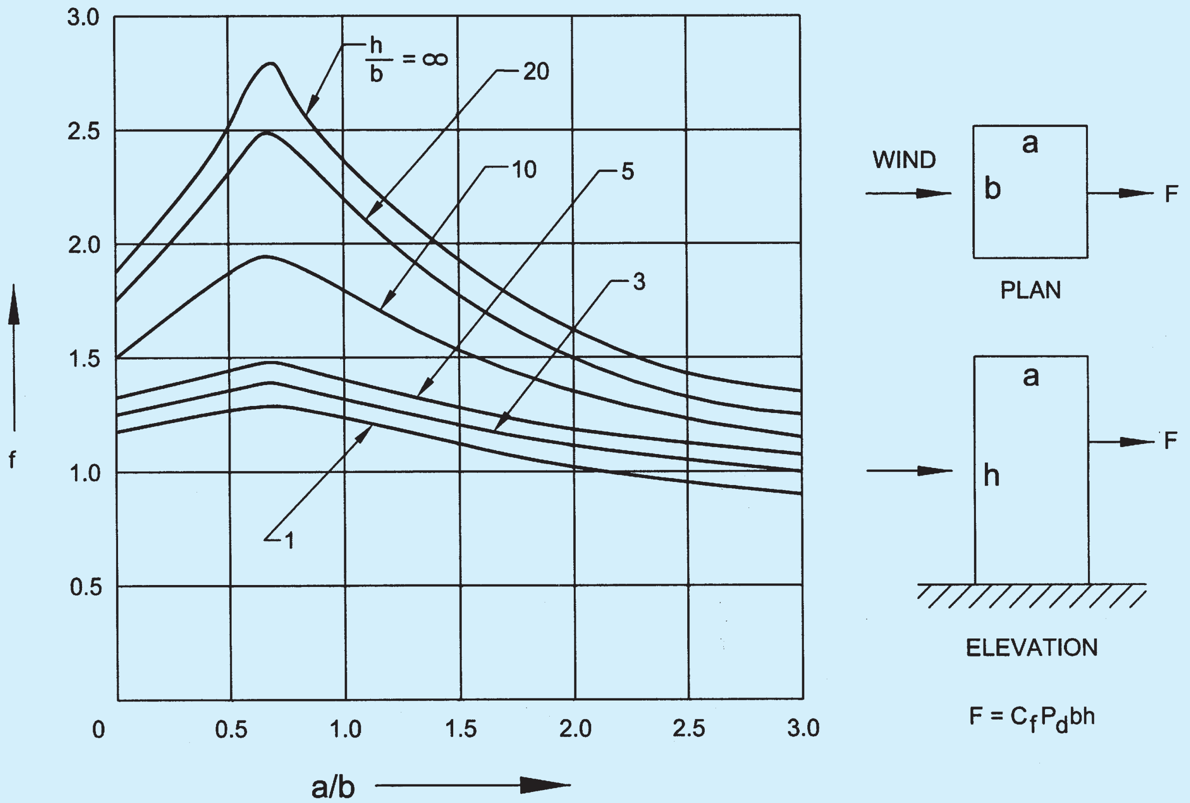
NOTES

1 The fundamental natural period (T_a), in seconds, of a moment-resisting frame building without brick infil panels and of all other buildings including with brick infil panels may be estimated in accordance with **5.4.6**.

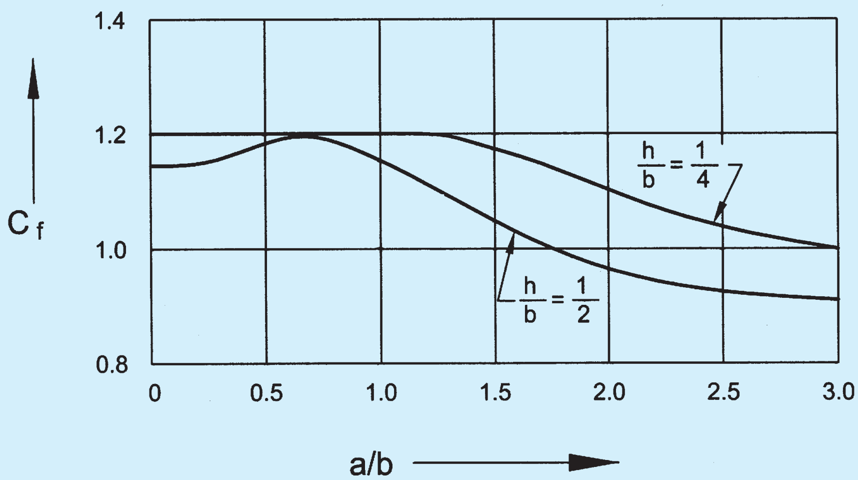
2 If preliminary studies indicate that wind-induced oscillations are likely to be significant, investigations should be persued with the aid of analytical methods or, if necessary, by means of wind tunnel tests on models.

3 Cross wind motions may be due to the lateral gustiness of the wind, unsteady wake flow (for example, vortex shedding), negative aerodynamic damping or to a combination of these effects. These cross-wind motions can become critical in the design of tall building structures.

4 Motions in the direction of the wind (also known as buffeting) are caused by fluctuating wind force associated with gusts. The excitations depend on the gust energy available at the resonant frequency.



a) Values of C_f *versus* a / b for h / b $\geq$ 1



b) Values of C_f *versus* a / b for h / b < 1

FIG. 4 FORCE COEFFICIENT FOR RECTANGULAR CLAD BUILDING IN UNIFORM FLOW

Table 26 Force Coefficients C_r for Clad Buildings of Uniform Section (Acting in the Direction of Wind)

[*Clause* 4.5.3.2 (a)]

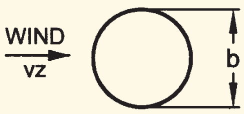
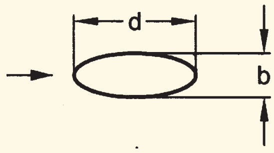
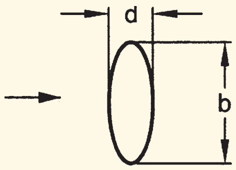
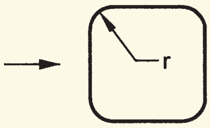
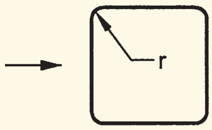
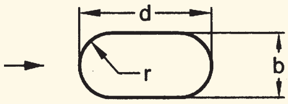
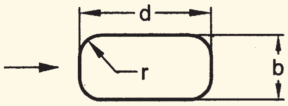
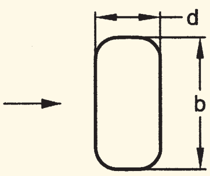
Plan Shape		$V_z b$ m^2/s	C_f for Height/Breadth Ratio Up to 1/2	1	2	5	10	20	∞
 (*see also* Annex E)	All surfaces Rough or with projections	< 6 ≥ 6	0.7	0.7	0.7	0.8	0.9	1.0	1.2
	Smooth	≥6	0.5	0.5	0.5	0.5	0.5	0.6	0.6
	Ellipse b/d = 1/2	< 10	0.5	0.5	0.5	0.5	0.6	0.6	0.7
		≥ 10	0.2	0.2	0.2	0.2	0.2	0.2	0.2
	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
	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
	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
	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
	b/d = 1/2 r/b = 1/6	All values	0.5	0.5	0.5	0.5	0.6	0.6	0.7
	b/d = 2 r/b = 1/12	All values	0.9	0.9	1.0	1.1	1.2	1.5	1.9

Table 26 — *Continued*

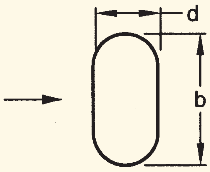
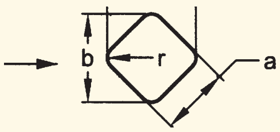
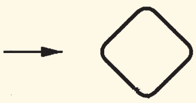
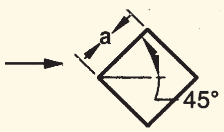
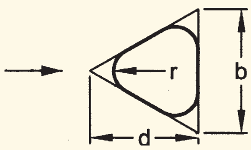
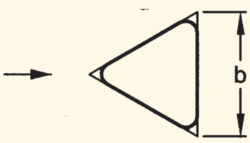
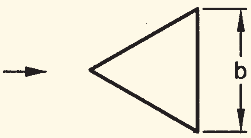
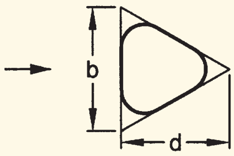
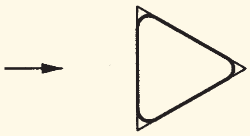
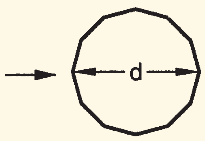
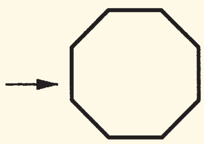
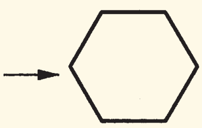
Plan Shape		$V_z b$ m^2/s	C_f for Height/Breadth Ratio Up to 1/2	1	2	5	10	20	∞
	All surfaces b/d = 2 r/b = 1/4	< 6	0.7	0.8	0.8	0.9	1.0	1.2	1.6
		≥ 6	0.5	0.5	0.5	0.5	0.5	0.6	0.6
	r/a = 1/3	< 10	0.8	0.8	0.9	1.0	1.1	1.3	1.5
		≥ 10	0.5	0.5	0.5	0.5	0.5	0.6	0.6
	r/a = 1/12	All values	0.9	0.9	0.9	1.1	1.2	1.3	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

Table 26 — *Concluded*

Plan Shape		$V_z b$ m^2/s	C_f for Height/Breadth Ratio Up to 1/2	1	2	5	10	20	∞
	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
	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

NOTE — Structures that, because of their size and the design wind velocity, are in the supercritical flow regime 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 this table *V* b is used as an indication of the airflow regime.

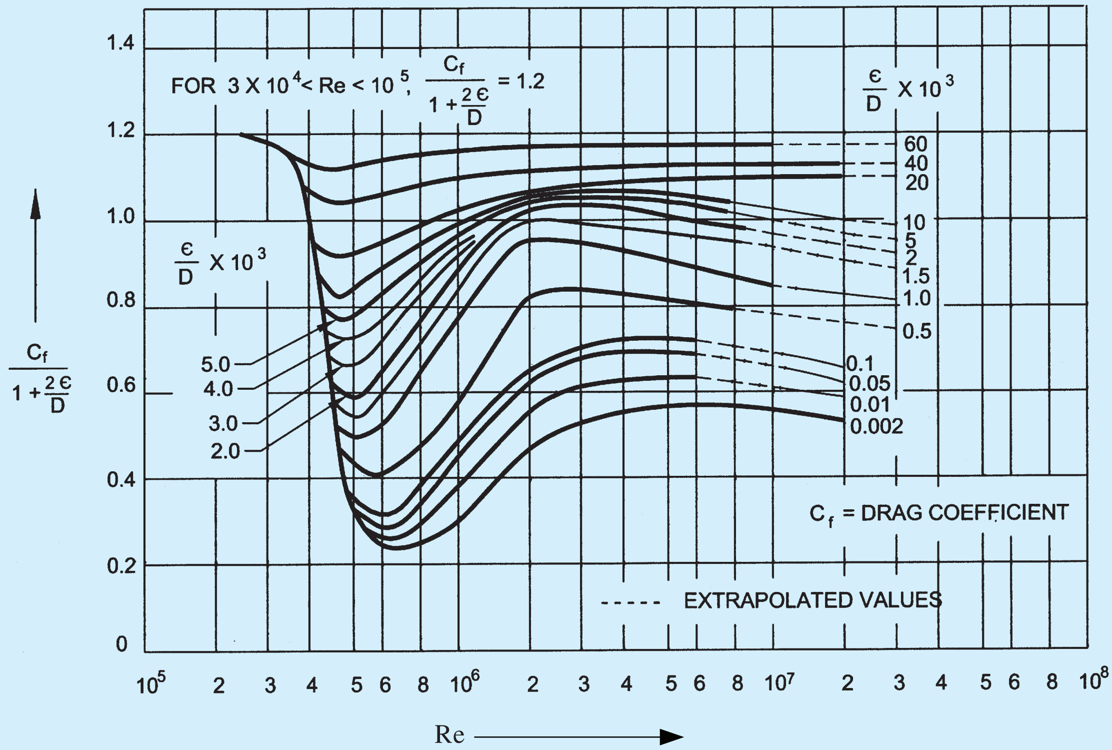
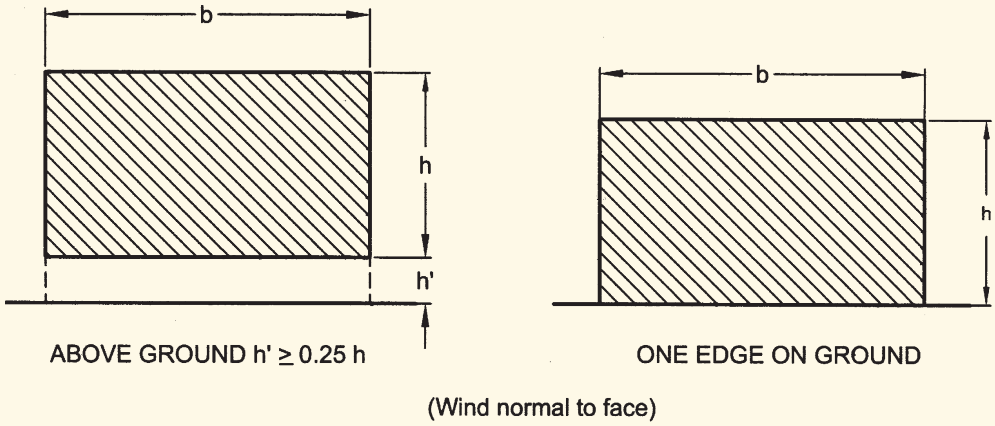


FIG. 5 VARIATION OF $\frac{C_f}{1+\frac{2\varepsilon}{D}}$ WITH $Re < 3 \times 10^4$ FOR CIRCULAR SECTIONS

Table 27 Force Coefficients for Low Walls or Hoardings (<15 m High)

[*Clause* 4.5.3.2 (b)]



Width to Height Ratio, *b/h*		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	1.7
60	120	1.8
80 or more	160 more 20	2.0

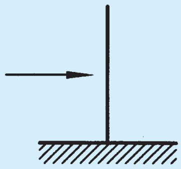
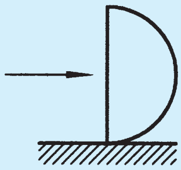
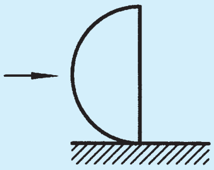
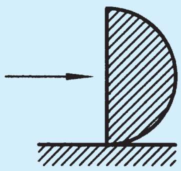
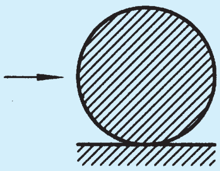
SIDE ELEVATION	DESCRIPTION OF SHAPE	C_f
	CIRCULAR DISC	1.2
	HEMISPHERICAL BOWL	1.4
	HEMISPHERICAL BOWL	0.4
	HEMISPHERICAL SOLID	-1.2
	SPHERICAL SOLID	0.5 FOR $V D < 7$ 0.2 FOR $V D \geq 7$

FIG. 6 FORCE COEFFICIENTS FOR SOLID SHAPES MOUNTED ON A SURFACE

5 The wake shed from an upstream body may intensify motions in the direction of the wind, and may also effect crosswind motions.

6 The designer must be aware of the following three forms of wind induced motion which are characterized by increasing amplitude of oscillation with 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:

   i) All structures with non-circular cross-sections, such as triangular, square, polygons, as well as angles, crosses and T-sections.

   ii) Twisted cables and cables with ice encrustations.

b) *Flutter* — Flutter is unstable oscillatory motion of a structure due to coupling between aerodynamic force and the elastic deformation of the structure. Perhaps the most common form is the oscillatory motion due to combined bending and torsion. Although oscillatory motions in each degree of freedom may be dampled, 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 motion. Such energy transfer takes place when the natural frequencies of the 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 the depth of a structure or structural member parallel to wind stream and t is the least lateral dimension of a member) 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 of 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 serious wind excited oscillations require careful investigation. It is to be noted that wind induced oscillations may occur at wind speeds lower than the static design wind speed for the location.

8 Analytical methods for determining dynamic response of structures to wind loading can be found in the following publications:

a) Engineering Science Data, Wind Engineering sub-series (4 volumes), London, ESDU International.

b) 'Wind Engineering in the Eighties'. Construction Industry Research and Information Association, 1981, London.

c) 'Wind Effects on Structures' by E Simiu and R.H. Scanlan. Johan Wiley and Sons, New York, 1978.

d) Supplement to the National Building Code of Canada, 1980. NRCC, No. 17724. National Research Council of Canada, Ottawa, 1980.

e) Wind Forces on Structures by Peter Sachs. Pergamon Press.

f) Flow Induced Vibration by Robert D. Clevins. Von Nostrand Reinfold Co.

9 In assessing wind loads due to such dynamic phenomenon as galloping, flutter and ovalling, if the required information is not available either in the references of Note 8 or other literature, specialist advice shall be sought, including experiments on models in wind tunnels.

4.6.2 *Motions Due to Vortex Shedding*

4.6.2.1 *Slender structures* — For a structure, the shedding frequency, η shall be determined by the following formula:

$$\eta = \frac{SV_d}{b}$$

where

S = Strouhal number,

V_d = design wind velocity, and

b = The breadth of a structure or structural members in the horizontal plane normal to the wind direction.

a) *Circular Structures* — For structures circular in cross-section:

S = 0.20 for bV_z not greater than 7, and

S = 0.25 for bV_z greater than 7.

b) *Rectangular Structures* — For structures of rectangular cross-section:

S = 0.15 for all values of bV_z.

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 velocities. In such cases, further analysis should be carried out on the basis of references given in Note 8 of **4.6.1**.

2 Unlined welded steel chimney stacks and similar structures are prone to excitation 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 20 b apart, where b is the dimension of the structure normal to the wind.

4 The formulae given in **4.6.2.1** (a) and **4.6.2.1** (b) are valid for infinitely long cylindrical structures. The value of S 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 width is less than 2.0.

4.7 Gust Factor (GF) or Gust Effectiveness Factor (GEF) Method

4.7.1 *Application*

Only the method of calculating load along wind or drag load by using gust factor method is given in the section since methods for calculating load across-wind or other components are not fully matured for all types of structures. However, it is permissible for a designer to use gust factor method to calculate all components of load on a structure using any available theory. However, such a theory must take into account the random nature of atmospheric wind speed.

NOTE — It may be noted that investigations for various types of wind induced oscillations out lined in **4.6** are in no way related to the use of gust factor method given in **4.7**. Although study of **4.6** is needed for using gust factor method.

4.7.2 *Hourly Mean Wind*

Use of the existing theories of gust factor method require a knowledge of the maximum of the wind speeds averaged over one hour at a particular site. Hourly mean wind speeds at different heights over different terrains is given in Table 28.

NOTE — It must also be recognized that the ratio of hourly mean wind (HMW) to peak gust (PG) given in Table 28 may not be obtainable in India since extreme wind occurs mainly due to cyclones and thunderstorms, unlike in UK and Canada where the mechanism is fully developed pressure system. However Table 28 may be followed at present for the estimation of the hourly mean wind speed till more reliable values become available.

Table 28 Hourly Mean Wind Speed Factor k_2 in Different Terrains for Different Heights

(*Clause* 4.7.2)

Height m	Terrain			
	Category 1	Category 2	Category 3	Category 4
(1)	(2)	(3)	(4)	(5)
10	0.78	0.67	0.50	0.24
15	0.82	0.72	0.55	0.24
20	0.85	0.75	0.59	0.24
30	0.88	0.79	0.64	0.34
50	0.93	0.85	0.70	0.45
100	0.99	0.92	0.79	0.57
150	1.03	0.96	0.84	0.64
200	1.06	1.00	0.88	0.68
250	1.08	1.02	0.91	0.72
300	1.09	1.04	0.93	0.74
350	1.11	1.06	0.95	0.77
400	1.12	1.07	0.97	0.79
450	1.13	1.08	0.98	0.81
500	1.14	1.09	0.99	0.82

4.7.2.1 *Variation of hourly mean wind speed with height*

The variation of hourly mean wind speed with height shall be calculated as follows:

$$\bar{V}_z = V_b k_1 \bar{k}_2 k_3$$

where

$\bar{V}_z$ = hourly mean wind speed in m/s at height z,

V_b = regional basic wind speed in m/s (*see* Fig. 1),

k_1 = probability factor (Table 4),

$\bar{k}_2$ = terrain and height factor (Table 28), and

k_3 = topography factor (**4.4.3.3**).

4.7.3 *Along Wind Load*

Along wind load on a structure on a strip area (A_e) at any height (Z) is given by:

$$F_z = C_f A_e \bar{p}_z G$$

where

F_z = along wind load on the structure at any height Z corresponding to strip area A_e,

C_f = force coefficient for the building,

A_e = effective frontal area considered for the structure at height Z.

$\bar{p}_z$ = design pressure at height Z due to mean hourly wind obtained as 0.6 $\bar{V}_z^2$ (N/m^2), and

G = gust factor $\frac{\text{peak load}}{\text{mean load}}$ and is given by:

$$G = 1 + g_f r \sqrt{\left[B\,(1+\phi)^2 + \frac{SE}{\beta} \right]}$$

where

g_f = peak factor defined as the ratio of the expected peak value to the root mean value of a fluctuating load, and

r = a roughness factor which is dependent on the size of the structure in relation to the ground roughness.

The value of '$g_f r$' is given in Fig. 7.

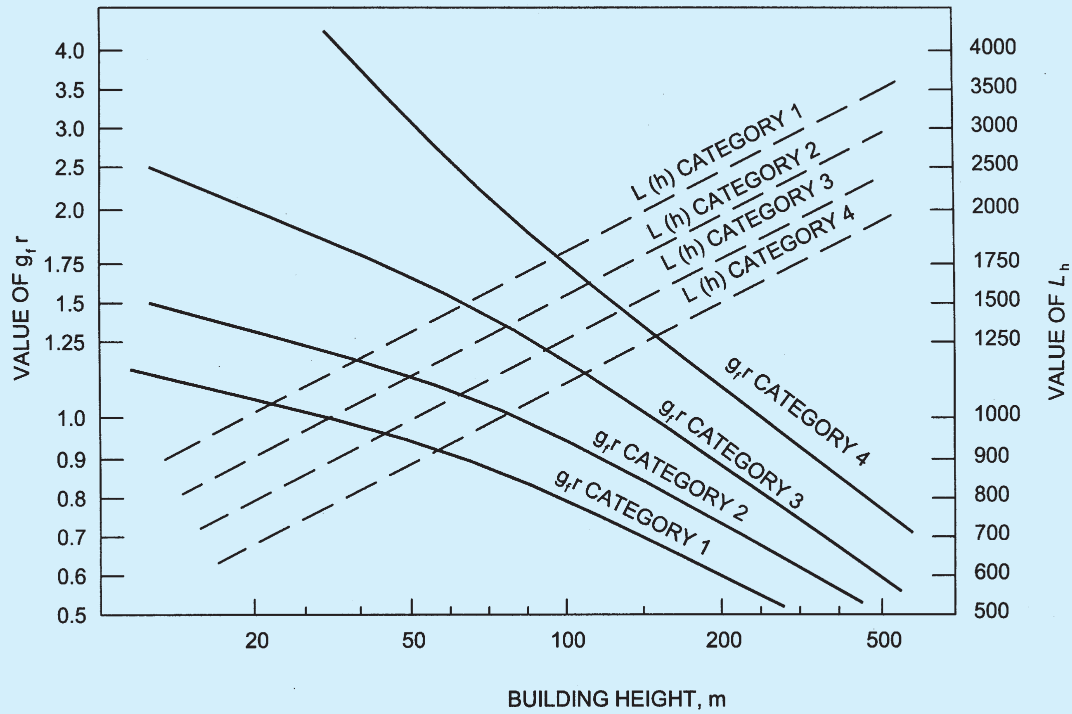


FIG. 7 VALUES OF $g_f r$ AND L_h

B is a background factor indicating a measure of the slowly varying component of the fluctuating wind load and is obtained from Fig. 8.

$\frac{SE}{\beta}$ is a measure of the resonant component of the fluctuating wind load.

E is a measure of the available energy in the wind stream at the natural frequency of the structure (*see* Fig. 9).

S is size reduction factor (*see* Fig. 10).

β is the damping coefficient (as a fraction of critical damping) of the structure (*see* Table 29).

$\phi = \frac{g_r \sqrt{B}}{4}$ and is to be accounted only for buildings less than 75 m high in terrain category 4 and for buildings less than 25 high in terrain category 3, and is to be taken as zero in all other cases.

In Fig. 8 and 10,

$$\lambda = \frac{C_y b}{C_z h} \text{ and } f_o = \frac{C_z f_o h}{V_h}$$

where

C_y = lateral correlation constant which may be taken as 10 in the absence of more precise load data;

C_z = longitudinal correlation constant which may be taken as 12 in the absence of more precise load data;

b = breadth of a structure normal to the wind stream;

h = height of a structure;

V_z = hourly mean wind speed at height Z;

f_o = natural frequency of the structure in the fundamental mode; and

L_h = a measure of turbulence length scale (*see* Fig. 7)

Table 29 Suggested Values of Damping Coefficient

(*Clause* 4.7.3)

Nature of Structure	Damping Coefficient, β
(1)	(2)
Welded steel structures	0.010
Bolted steel structures	0.020
Reinforced concrete structures	0.016

The peak acceleration along the wind direction at the top of the structure is given by the following formula:

$$a = (2\pi f_o)^2 \, \bar{x} \, g_f \, r \frac{SE}{\beta}$$

where

$\bar{x}$ = mean deflection at the position where the acceleration is required.

5 SEISMIC LOAD

5.0 This clause deals with assessment of seismic loads on various structures and earthquake resistant design of buildings. (For the purpose of this clause the symbols given at Annex G are applicable).

5.1 Terminology for Earthquake Engineering

5.1.1 For the purpose of this standard, the following definitions shall apply which are applicable generally to all structures:

NOTE — For the definitions of terms pertaining to soil mechanics and soil dynamics references may be made to Part 6 'Structural Design, Section 2 Soils and Foundations'.

5.1.2 *Closely-Spaced Modes*

Closely-spaced modes of a structure are those of its natural modes of vibration whose natural frequencies differ from each other by 10 percent or less of the lower frequency.

5.1.3 *Critical Damping*

The damping beyond which the free vibration motion will not be oscillatory.

5.1.4 *Damping*

The effect of internal friction, imperfect elasticity of material, slipping, sliding etc, in reducing the amplitude of vibration and is expressed as a percentage of critical damping.

5.1.5 *Design Acceleration Spectrum*

Design acceleration spectrum refers to an average smoothened plot of maximum acceleration as a function of frequency or time period of vibration for a specified damping ratio for earthquake excitations at the base of a single degree of freedom system.

5.1.6 *Design Basis Earthquake (DBE)*

It is the earthquake which can reasonably be expected to occur at least once during the design life of the structure.

5.1.7 *Design Horizontal Acceleration Coefficient (A_h)*

It is a horizontal acceleration coefficient that shall be used for design of structures.

5.1.8 *Design Lateral Force*

It is the horizontal seismic force prescribed by this standard, that shall be used to design a structure.

5.1.9 *Ductility*

Ductility of a structure, or its members, is the capacity to undergo large inelastic deformations without significant loss of strength or stiffness.

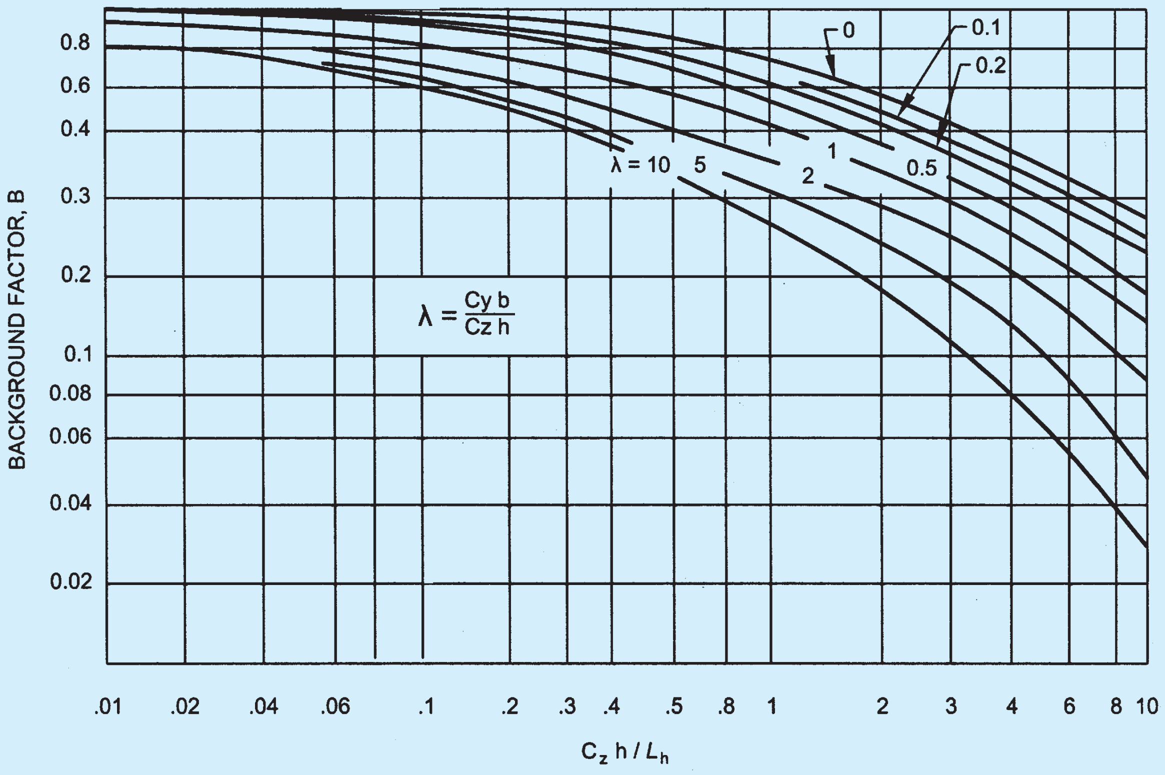


FIG. 8 BACKGROUND FACTOR, *B*

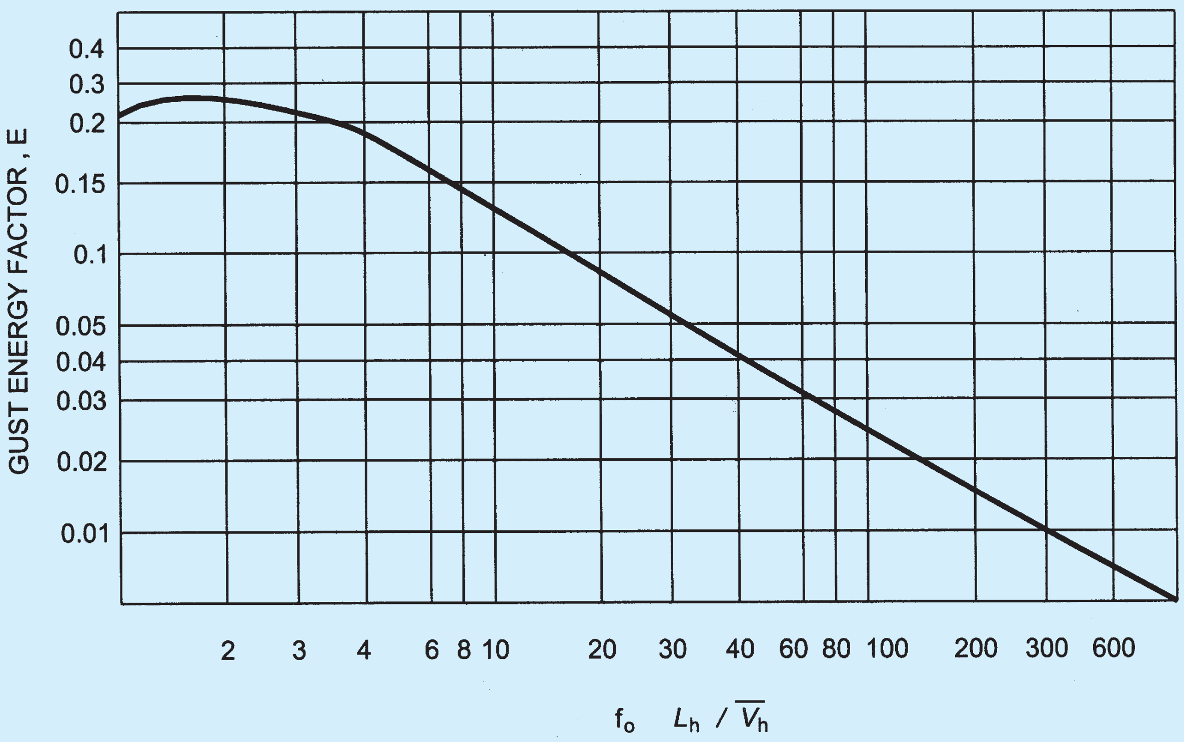


FIG. 9 GUST ENERGY FACTOR, *E*

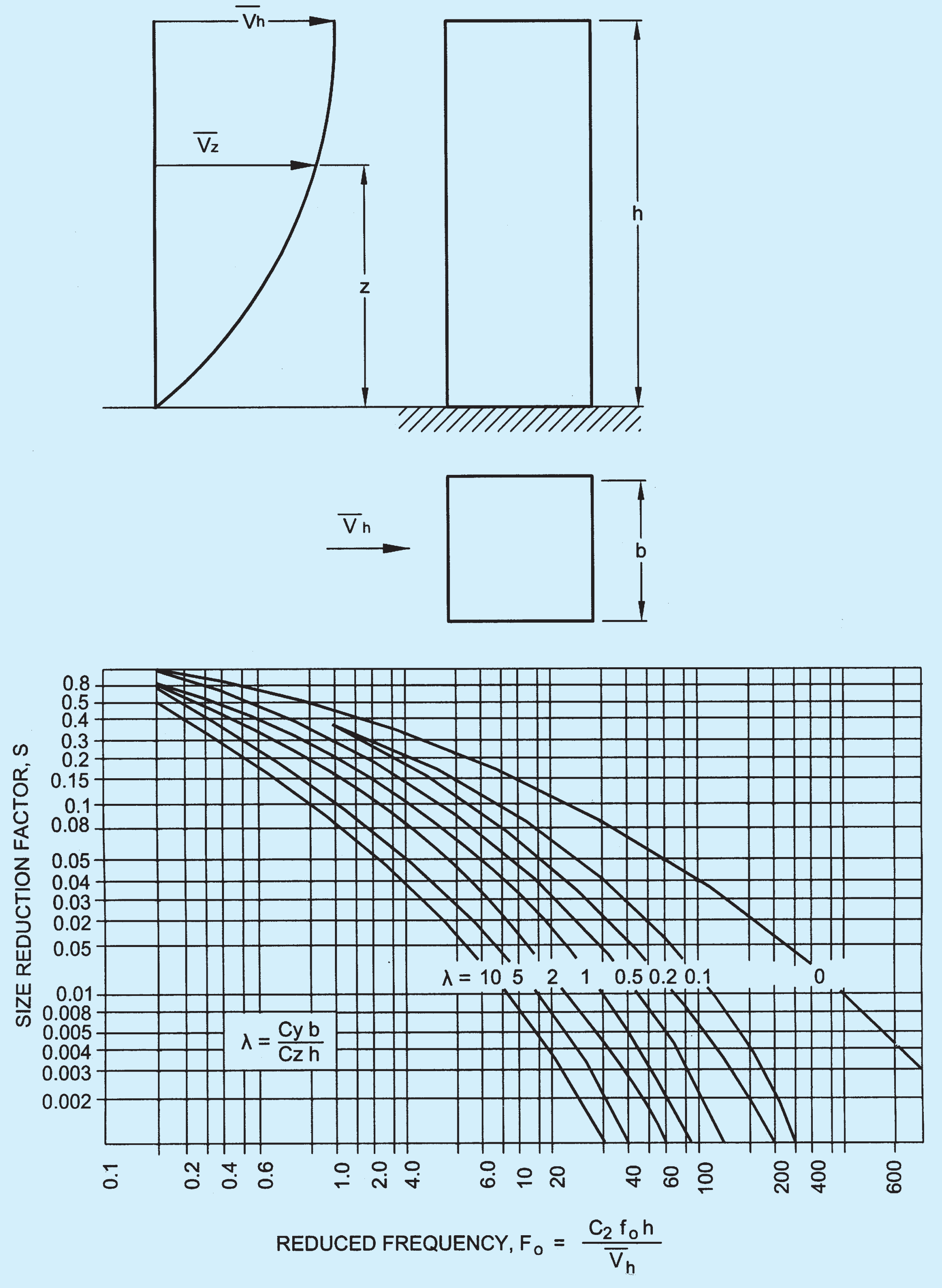


FIG. 10 SIZE REDUCTION FACTOR, *S*

5.1.10 *Epicentre*

The geographical point on the surface of earth vertically above the focus of the earthquake.

5.1.11 *Effective Peak Ground Acceleration (EPGA)*

It is 0.4 times the 5 percent damped average spectral acceleration between period 0.1 to 0.3 s. This shall be taken as zero period acceleration (ZPA).

5.1.12 *Floor Response Spectra*

Floor response spectra is the response spectra for a time history motion of a floor. This floor motion time history is obtained by an analysis of multi-storey building for appropriate material damping values subjected to a specified earthquake motion at the base of structure.

5.1.13 *Focus*

The originating earthquake source of the elastic waves inside the earth which cause shaking of ground due to earthquake.

5.1.14 *Importance Factor (I)*

It is a factor used to obtain the design seismic force depending on the functional use of the structure, characterised by hazardous consequences of its failure, its post-earthquake functional need, historic value, or economic importance.

5.1.15 *Intensity of Earthquake*

The intensity of an earthquake at a place is a measure of the strength of shaking during the earthquake, and is indicated by a number according to the modified Mercalli Scale or M.S.K. scale of Seismic Intensities (*see* Annex H).

5.1.16 *Liquefaction*

Liquefaction is a state in saturated cohesionless soil wherein the effective shear strength is reduced to negligible value for all engineering purpose due to pore pressure caused by vibrations during an earthquake when they approach the total confining pressure. In this condition the soil tends to behave like a fluid mass.

5.1.17 *Lithological Features*

The nature of the geological formation of the earth's crust above bed rock on the basis of such characteristics as colour, structure, mineralogical composition and grain size.

5.1.18 M*agnitude of Earthquake (Richter's Magnitude)*

The magnitude of earthquake is a number, which is a measure of energy released in an earthquake. It is defined as logarithm to the base 10 of the maximum trace amplitude, expressed in microns, which the standard short-period torsion seismometer (with a period of 0.8 s, magnification 2 800 and damping nearly critical) would register due to the earthquake at an epicentral distance of 100 km.

5.1.19 *Maximum Considered Earthquake (MCE)*

The most severe earthquake effects considered by this Code.

5.1.20 *Modal Mass (M_k)*

Modal mass of a structure subjected to horizontal or vertical, as the case may be, ground motion is a part of the total seismic mass of the structure that is effective in *mode k* of vibration. The modal mass for a given mode has a unique value irrespective of scaling of the mode shape.

5.1.21 *Modal Participation Factor (P_k)*

Modal participation factor of mode *k* of vibration is the amount by which mode *k* contributes to the overall vibration of the structure under horizontal and vertical earthquake ground motions. Since the amplitudes of 95 per cent mode shapes can be scaled arbitrarily, the value of this factor depends on the scaling used for mode shapes.

5.1.22 *Modes of Vibration* (*see* **5.1.25**)

5.1.23 *Mode Shape Coefficient (ϕ_{ik})*

When a system is vibrating in normal *mode k*, at any particular instant of time, the amplitude of mass *i* expressed as a ratio of the amplitude of one of the masses of the system, is known as mode shape coefficient (ϕ_{ik}).

5.1.24 *Natural Period (T)*

Natural period of a structure is its time period of undamped free vibration.

5.1.24.1 *Fundamental natural period* (T_1)

It is the first (longest) modal time period of vibration.

5.1.24.2 *Modal natural period* (T_k)

The modal natural period of mode *k* is the time period of vibration in *mode k.*

5.1.25 *Normal Mode*

A system is said to be vibrating in a normal mode when all its masses attain maximum values of displacements and rotations simultaneously, and pass through equilibrium positions simultaneously.

5.1.26 *Response Reduction Factor (R)*

It is the factor by which the actual base shear force, that would be generated if the structure were to remain elastic during its response to the design basis earthquake (DBE) shaking, shall be reduced to obtain the design lateral force.

5.1.27 *Response Spectrum*

The representation of the maximum response of idealized single degree freedom systems having certain period and damping, during earthquake ground motion. The maximum response is plotted against the undamped natural period and for various damping values, and can be expressed in terms of maximum absolute acceleration, maximum relative velocity, or maximum relative displacement.

5.1.28 *Seismic Mass*

It is the seismic weight divided by acceleration due to gravity.

5.1.29 *Seismic Weight (W)*

It is the total dead load plus appropriate amounts of specified imposed load.

5.1.30 *Structural Response Factors (S_a/g)*

It is a factor denoting the acceleration response spectrum of the structure subjected to earthquake ground vibrations, and depends on natural period of vibration and damping of the structure.

5.1.31 *Tectonic Features*

The nature of geological formation of the bed rock in the earth's crust revealing regions characterized by structural features, such as dislocation, distortion, faults, folding, thrusts, volcanoes with their age of formation, which are directly involved in the earth movement or quake resulting in the above consequences.

5.1.32 *Time History Analysis*

It is an analysis of the dynamic response of the structure attach increment of time, when its base is subjected to a specific ground motion time history.

5.1.33 *Zone Factor (Z)*

It is a factor to obtain the design spectrum depending on the perceived maximum seismic risk characterized by maximum considered earthquake (MCE) in the zone in which the structure is located. The basic zone factors included in this standard are reasonable estimate of effective peak ground acceleration.

5.1.34 *Zero Period Acceleration (ZPA)*

It is the value of acceleration response spectrum for period below 0.03 s (frequencies above 33 Hz).

5.2 Terminology for Earthquake Engineering of Buildings

5.2.1 For the purpose of earthquake resistant design of buildings in this standard, the following definitions shall apply:

5.2.2 *Base*

It is the level at which inertia forces generated in the structure are transferred to the foundation, which then transfers these forces to the ground.

5.2.3 *Base Dimensions (d)*

Base dimension of the building along a direction is the dimension at its base, in metres, along that direction.

5.2.4 *Centre of Mass*

The point through which the resultant of the masses of a system acts. This point corresponds to the centre of gravity of masses of system.

5.2.5 *Centre of Stiffness*

The point through which the resultant of the restoring forces of a system acts.

5.2.6 *Design Eccentricity (e_{di})*

It is the value of eccentricity to be used at floor *i* in torsion calculations for design.

5.2.7 *Design Seismic Base Shear (V_B)*

It is the total design lateral force at the base of a structure.

5.2.8 *Diaphragm*

It is a horizontal, or nearly horizontal system, which transmits lateral forces to the vertical resisting elements, for example, reinforced concrete floors and horizontal bracing systems.

5.2.9 *Dual System*

Buildings with dual system consist of shear walls (or braced frames) and moment resisting frames such that:

a) the two systems are designed to resist the total design lateral force in proportion to their lateral stiffness considering the interaction of the dual system at all floor levels; and
b) the moment resisting frames are designed to independently resist at least 25 percent of the design base shear.

5.2.10 *Height of Floor (h_i)*

It is the difference in levels between the base of the building and that of floor *i*.

5.2.11 *Height of Structure (h)*

It is the difference in levels, in metres, between its base and its highest level.

5.2.12 *Horizontal Bracing System*

It is a horizontal truss system that serves the same function as a diaphragm.

5.2.13 *Joint*

It is the portion of the column that is common to other members, for example, beams, framing into it.

5.2.14 *Lateral Force Resisting Element*

It is part of the structural system assigned to resist lateral forces.

5.2.15 *Moment-Resisting Frame*

It is a frame in which members and joints are capable of resisting forces primarily by flexure.

5.2.15.1 *Ordinary moment-resisting frame*

It is a moment-resisting frame not meeting special detailing requirements for ductile behaviour.

5.2.15.2 *Special moment-resisting frame*

It is a moment-resisting frame specially detailed to provide ductile behaviour and comply with the requirements given in IS 4326 or IS 13920 or SP 6(6).

5.2.16 *Number of Storeys (n)*

Number of storeys of a building is the number of levels above the base. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But, it includes the basement storeys, when they are not so connected.

5.2.17 *Principal Axes*

Principal axes of a building are generally two mutually perpendicular horizontal directions in plan of a building along which the geometry of the building is oriented.

5.2.18 $P-\Delta$ *Effect*

It is the secondary effect on shears and moments of frame members due to action of the vertical loads, interacting with the lateral displacement of building resulting from seismic forces.

5.2.19 *Shear Wall*

It is a wall designed to resist lateral forces acting in its own plane.

5.2.20 *Soft Storey*

It is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above.

5.2.21 *Static Eccentricity (e_{si})*

It is the distance between centre of mass and centre of rigidity of floor *i*.

5.2.22 *Storey*

It is the space between two adjacent floors.

5.2.23 *Storey Drift*

It is the displacement of one level relative to the other level above or below.

5.2.24 *Storey Shear (V_i)*

It is the sum of design lateral forces at all levels above the storey under consideration.

5.2.25 *Weak Storey*

It is one in which the storey lateral strength is less than 80 percent of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction.

5.3 General Principles and Design Criteria

5.3.1 *General Principles*

5.3.1.1 *Ground motion*

The characteristics (intensity, duration, etc) of seismic ground vibrations expected at any location depends upon the magnitude of earthquake, its depth of focus, distance from the epicentre, characteristics of the path through which the seismic waves travel, and the soil strata on which the structure stands. The random earthquake ground motions, which cause the structure to vibrate, can be resolved in any three mutually perpendicular directions. The predominant direction of ground vibration is usually horizontal.

Earthquake-generated vertical inertia forces are to be considered in design unless checked and proven by specimen calculations to be not significant. Vertical acceleration should be considered in structures with large spans, those in which stability is a criterion for design, or for overall stability analysis of structures. Reduction in gravity force due to vertical component of ground motions can be particularly detrimental in cases of prestressed horizontal members and of cantilevered members. Hence, special attention should be paid to the effect of vertical component of the ground motion on prestressed or cantilevered beams, girders and slabs.

5.3.1.2 The response of a structure to ground vibrations is a function of the nature of foundation soil, materials, form, size and mode of construction of structures; and the duration and characteristics of ground motion. This standard specifies design forces for structures standing on rocks or soils which do not settle, liquefy or slide due to loss of strength during ground vibrations.

5.3.1.3 The design approach adopted in this standard is to ensure that structures possess at least a minimum strength to withstand minor earthquakes (<DBE), which occur frequently, without damage; resist moderate earthquakes (DBE) without significant structural damage though some non-structural damage may occur; and aims that structures withstand a major earthquake (MCE) without collapse. Actual forces that appear on structures during earthquakes are much greater than the design forces specified in this Code. However, ductility, arising from inelastic material behaviour and detailing, and overstrength, arising from the additional reserve strength in structures over and above the design strength, are relied upon to account for this difference in actual and design lateral loads.

Reinforced and prestressed concrete members shall be suitably designed to ensure that premature failure due to shear or bond does not occur, subject to the provisions of Part 6 'Structural Design, Section 5 Concrete'. Provisions for appropriate ductile detailing of reinforced concrete members shall be in accordance with good practice [6-1(4)].

In steel structures, members and their connections should be so proportioned that high ductility is obtained, vide SP 6(6), avoiding premature failure due to elastic or inelastic buckling of any type.

The specified earthquake loads are based upon post-elastic energy dissipation in the structure and because of this fact, the provision of this Code for design, detailing and construction shall be satisfied even for structures and members for which load combinations that do not contain the earthquake effect indicate larger demands than combinations including earthquake.

5.3.1.4 *Soil-structure interaction*

The soil-structure interaction refers to the effects of the supporting foundation medium on the motion of structure. The soil-structure interaction may not be considered in the seismic analysis for structures supported on rock or rock-like material.

5.3.1.5 The design lateral force specified in this Code shall be considered in each of the two orthogonal horizontal directions of the structure. For structures which have lateral force resisting elements in the two orthogonal directions only, the design lateral force shall be considered along one direction at a time, and not in both directions simultaneously. Structures, having lateral force resisting elements (for example, frames, shear walls) in directions other than the two orthogonal directions, shall be analysed considering the load combinations specified in **5.3.3.2**.

Where both horizontal and vertical seismic forces are taken into account, load combinations specified in **5.3.3.3** shall be considered.

5.3.1.6 Equipment and other systems, which are supported at various floor levels of the structure, will be subjected to motions corresponding to vibration at their support points. In important cases, it may be necessary to obtain floor response spectra for design of equipment supports. For detail reference be made to good practice [6-1(5)]

5.3.1.7 *Additions to existing structures*

Additions shall be made to existing structures only as follows:

a) An addition that is structurally independent from an existing structures shall be designed and constructed in accordance with the seismic requirements for new structures.
b) An addition that is not structurally independent from an existing structure shall be designed and constructed such that the entire structure conforms to the seismic force resistance requirements for new structures unless the following three conditions are complied with:
    1) The addition shall comply with the requirements for new structures;
    2) The addition shall not increase the seismic forces in any structural elements of the existing structure by more than 5 per cent unless the capacity of the element subject to the increased force is still in compliance with this standard; and
    3) The addition shall not decrease the seismic resistance of any structural element of the existing structure unless reduced resistance is equal to or greater than that required for new structures.

5.3.1.8 *Change in occupancy*

When a change of occupancy results in a structure being reclassified to a higher importance factor (I), the structure shall conform to the seismic requirements for a new structure with the higher importance factor.

5.3.2 *Assumptions*

The following assumptions shall be made in the earthquake-resistant design of structures:

a) Earthquake causes impulsive ground motions, which are complex and irregular in character, changing in period and amplitude each lasting for a small duration. Therefore, resonance of the type as visualized under steady-state sinusoidal excitations, will not occur as it would need time to build up such amplitudes.

    NOTE — However, there are exceptions where resonance-like conditions have been seen to occur between long distance waves and tall structures founded on deep soft soils.

b) Earthquake is not likely to occur simultaneously with wind or maximum flood or maximum sea waves.

c) The value of elastic modulus of materials, wherever required, may be taken as for static analysis unless a more definite value is available for use in such condition (*see* Part 6 'Structural Design, Section 5 Concrete and Section 6 Steel'.

5.3.3 *Load Combination and Increase in Permissible Stresses*

5.3.3.1 *Load combinations*

When earthquake forces are considered on a structure, these shall be combined as per **5.3.3.1.1** and **5.3.3.1.2** where the terms *DL*, *IL* and *EL* stand for the response quantities due to dead load, imposed and designated earthquake load respectively.

5.3.3.1.1 *Load factors for plastic design of steel structures*

In the plastic design of steel structures, the following load combinations shall be accounted for:

a) 1.7 ($DL + IL$)
b) 1.7 ($DL \pm EL$)
c) 1.3 ($DL + IL \pm EL$)

5.3.3.1.2 *Partial safety factors for limit state design of reinforced concrete and prestressed concrete structures*

In the limit state design of reinforced and prestressed concrete structures, the following load combinations shall be accounted for:

a) 1.5 ($DL + IL$)
b) 1.2 ($DL + IL \pm EL$)
c) 1.5 ($DL \pm EL$)
d) 0.9 $DL \pm 1.5\ EL$

5.3.3.2 *Design horizontal earthquake load*

5.3.3.2.1 When the lateral load resisting elements are oriented along orthogonal horizontal direction, the structure shall be designed for the effects due to full design earthquake load in one horizontal direction at time.

5.3.3.2.2 When the lateral load resisting elements are not oriented along the orthogonal horizontal directions, the structure shall be designed for the effects due to full design earthquake load in one horizontal direction plus 30 percent of the design earthquake load in the other direction.

NOTE — For instance, the building should be designed for ($\pm EL_x \pm 0.3\ EL_y$) as well as ($\pm 0.3\ EL_x \pm EL_y$), where *x* and *y* are two orthogonal horizontal direction, *EL* in **5.3.3.1.1** and **5.3.3.1.2** shall be replaced by ($EL_x \pm 0.3\ EL_y$) or ($EL_y \pm 0.3\ EL_x$).

5.3.3.3 *Design vertical earthquake load*

When effects due to vertical earthquake loads are to be considered, the design vertical force shall be calculated in accordance with **5.3.4.5**.

5.3.3.4 *Combination for two or three component motion*

5.3.3.4.1 When responses from the three earthquake components are to be considered, the responses due to each components may be combined using the assumption that when the maximum response from one component occurs, the responses from the other two component are 30 percent of their maximum. All possible combinations of the three components (EL_x, EL_y and EL_z) including variations in sign (plus or minus) shall be considered. Thus, the response due earthquake force (*EL*) is the maximum of the following three cases:

a) $\pm EL_x \pm 0.3\ EL_Y \pm 0.3\ EL_z$
b) $\pm EL_y \pm 0.3\ EL_x \pm 0.3\ EL_z$
c) $\pm EL_z \pm 0.3\ EL_x \pm 0.3\ EL_y$

Where *x* and *y* are two orthogonal directions and z is vertical direction.

5.3.3.4.2 As an alternative to the procedure in **5.3.3.4.1**, the response (*EL*) due to the combined effect of the three components can be obtained on the basis of 'square root of the sum of the square (SRSS)' that is

$$EL = \sqrt{(EL_x)^2 + (EL_y)^2 + (EL_z)^2}$$

NOTE — The combination procedure of **5.3.3.4.1** and **5.3.3.4.2** apply to the same response quantity (say, moment in a column about its major axis, or storey shear in a frame) due to different components of the ground motion.

5.3.3.4.3 When two component motions (say one horizontal and one vertical, or only two horizontal) are combined, the equations in **5.3.3.4.1** and **5.3.3.4.2** should be modified by deleting the term representing the response due to the component of motion not being considered.

5.3.3.5 *Increase in permissible stresses*

5.3.3.5.1 *Increase in permissible stresses in materials*

When earthquake forces are considered along with other normal design forces, the permissible stresses in material, in the elastic method of design, may be increased by one-third. However, for steels having a definite yield stress, the stress be limited to the yield stress; for steels without a definite yield point, the stress will be limited to 80 percent of the ultimate strength or 0.2 percent proof stress, whichever is smaller; and that in prestressed concrete members, the tensile stress in the extreme fibres of the concrete may be permitted so as not to exceed two-thirds of the modulus of rupture of concrete.

5.3.3.5.2 *Increase in allowable pressure in soils*

When earthquake forces are included, the allowable bearing pressure in soils shall be increased as per Table 1, depending upon type of foundation of the structure and the type of soil.

In soil deposits consisting of submerged loose sands and soils falling under classification SP with standard penetration N values less than 15 in seismic Zones III, IV, V and less than 10 in seismic Zone II, the vibration caused by earthquake may cause liquefaction or excessive total and differential settlements. Such sites should preferably be avoided while locating new settlements or important projects. Otherwise, this aspect of the problem needs to be investigated and appropriate methods of compaction or stabilization adopted to achieve suitable N values as indicated in Note 3 under Table 30. Alternatively, deep pile foundation may be provided and taken to depths well into the layer which is not likely to liquefy. Marine clays and other sensitive clays are also known to liquefy due to collapse of soil structure and will need special treatment according to site condition.

NOTE — Specialist literature may be referred for determining liquefaction potential of a site.

Table 30 Percentage of Permissible Increase in Allowable Bearing Pressure or Resistance of Soils

(*Clause* 5.3.3.5.2)

Sl No.	Foundation	Type of Soil Mainly Constituting the Foundation		
		Type I Rock or Hard Soil: Well graded gravel and sand gravel mixtures with or without clay binder, and clayey sands poorly graded or sand clay mixtures (GB, CW, SB, SW and SC)[1] having[2] above 30, where N is the standard penetration value	Type II Medium Soils All soils with N between 10 an 30, and poorly graded sands or gravelly sands with little or no fines (SP[1]) with $N > 15$	Type III Soft Soils: All soils other than SP[1] with $N < 10$
(1)	(2)	(3)	(4)	(5)
i)	Piles passing through any soil but resting on solid type I	50	50	50
ii)	Piles not covered under item (i)	—	25	25
iii)	Raft foundations	50	50	50
iv)	Combined isolated RCC footing with tie beams	50	25	—
v)	Isolated RCC footing without tie beams, or unreinforced strip foundations	50	25	—
vi)	Well foundations	50	25	25

NOTES

1 The allowable bearing pressure shall be determined in accordance with good practice [6-1(6)].

2 If any increase in bearing pressure has already been permitted for forces other than seismic forces, the total increase in allowable bearing pressure when seismic force is also included shall not exceed the limits specified above.

3 Desirable minimum field values of N — If soils of smaller N-values are met, compacting may be adopted to achieve these values or deep pile foundations going to stronger strata should be used.

4 The values of N (corrected values) are at the founding level and the allowable bearing pressure shall be determined in accordance with good practice [6-1(6)].

Seismic Zone Level	*Depth Below Ground* m	*N-Values*	*Remark*
III, IV and V	≤ 5	15	For values of depths between 5 m and 10 m, linear interpolation is recommended.
	≥ 10	25	
II (for important structures only)	≤ 5	15	
	≥ 10	20	

5 The piles should be designed for lateral loads neglecting lateral resistance of soil layers liable to liquefy.

6 Accepted standard [6-1(7)] and good practice [6-1(8)] may also be referred.

7 Isolated R.C.C. footing without tie beams, or unreinforced strip foundation shall not be permitted in soft soils with N<10.

[1] *See* accepted standard [6-1(7)].

[2] *See* good practice [6-1(8)].

5.3.4 *Design Spectrum*

5.3.4.1 For the purpose of determining seismic forces, the country is classified into four seismic zones as shown in Fig. 11.

5.3.4.2 The design horizontal seismic coefficient A_h for a structure shall be determined by the following expression:

$$A_h = \frac{ZI\ S_a}{2\ Rg}$$

Provided that for any structure with $T \geq 0.1$ sec, the value of A_h will not be less than $Z/2$ whatever be the value of I/R

where

Z = Zone factor given in Table 31, is for the maximum considered earthquake (MCE) and service life of structure in a zone. The factor 2 in the denominator of $\geq$ is used so as to reduce the maximum considered earthquake (MCE) zone factor to the factor for design basis earthquake (DBE).
Zone factor for some important towns are given at Annex J.

I = Importance factor, depending upon the functional use of the structures, characterised by hazardous consequences of its failure, post-earthquake functional needs, historical value, or economic importance (Table 35).

R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. However, the ratio (I/R) shall not be greater than 1.0 (Table 36). The values of R for buildings are given in Table 36.

S_a/g = Average response acceleration coefficient for rock or soil sites as given by Fig. 12 and Table 32 based on appropriate natural periods and damping of the structure. These curves represent free field ground motion.

NOTE — For various types of structures, the values of Importance Factor I, Response Reduction Factor R, and damping values are given in the respective parts of this standard. The method (empirical or otherwise) to calculate the natural periods of the structure to be adopted for evaluating S_a/g is also given in the respective parts of this Code.

Table 31 Zone Factor, *Z*
(*Clause* 5.3.4.2)

Seismic Zone	**II**	**III**	**IV**	**V**
Seismic Intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

5.3.4.3 Where a number of modes are to be considered for dynamic analysis, the value of A_h as defined in **5.3.4.2** for each mode shall be determined using the natural period of vibration of that mode.

5.3.4.4 For underground structures and foundations at depths of 30 m or below, the design horizontal acceleration spectrum value shall be taken as half the value obtained from **5.3.4.2**. For structures and foundations placed between the ground level and 30 m depth, the design horizontal acceleration spectrum value shall be linearly interpolated between A_h and 0.5 A_h, where A_h is as specified in **5.3.4.2**.

5.3.4.5 The design acceleration spectrum for vertical motions, when required, may be taken as two-thirds of the design horizontal acceleration spectrum specified in **5.3.4.2**.

Figure 12 shows the proposed 5 percent spectra for rocky and soils sites and Table 32 gives the multiplying factors for obtaining spectral values for various other dampings.

For Rocky, or hard soil sites

$$\frac{S_a}{g} = \begin{cases} 1+15\,T; & 0.00 \leq T \leq 0.10 \\ 2.50 & 0.10 \leq T \leq 0.40 \\ 1.00/T & 0.40 \leq T \leq 4.00 \end{cases}$$

For Medium soil sites

$$\frac{S_a}{g} = \begin{cases} 1+15\,T; & 0.00 \leq T \leq 0.10 \\ 2.50 & 0.10 \leq T \leq 0.55 \\ 1.36/T & 0.55 \leq T \leq 4.00 \end{cases}$$

For Soft soil sites

$$\frac{S_a}{g} = \begin{cases} 1+15\,T; & 0.00 \leq T \leq 0.10 \\ 2.50 & 0.10 \leq T \leq 0.67 \\ 1.67/T & 0.67 \leq T \leq 4.00 \end{cases}$$

Table 32 Multiplying Factors for Obtaining Values for Other Damping
(*Clause* 5.3.4.2)

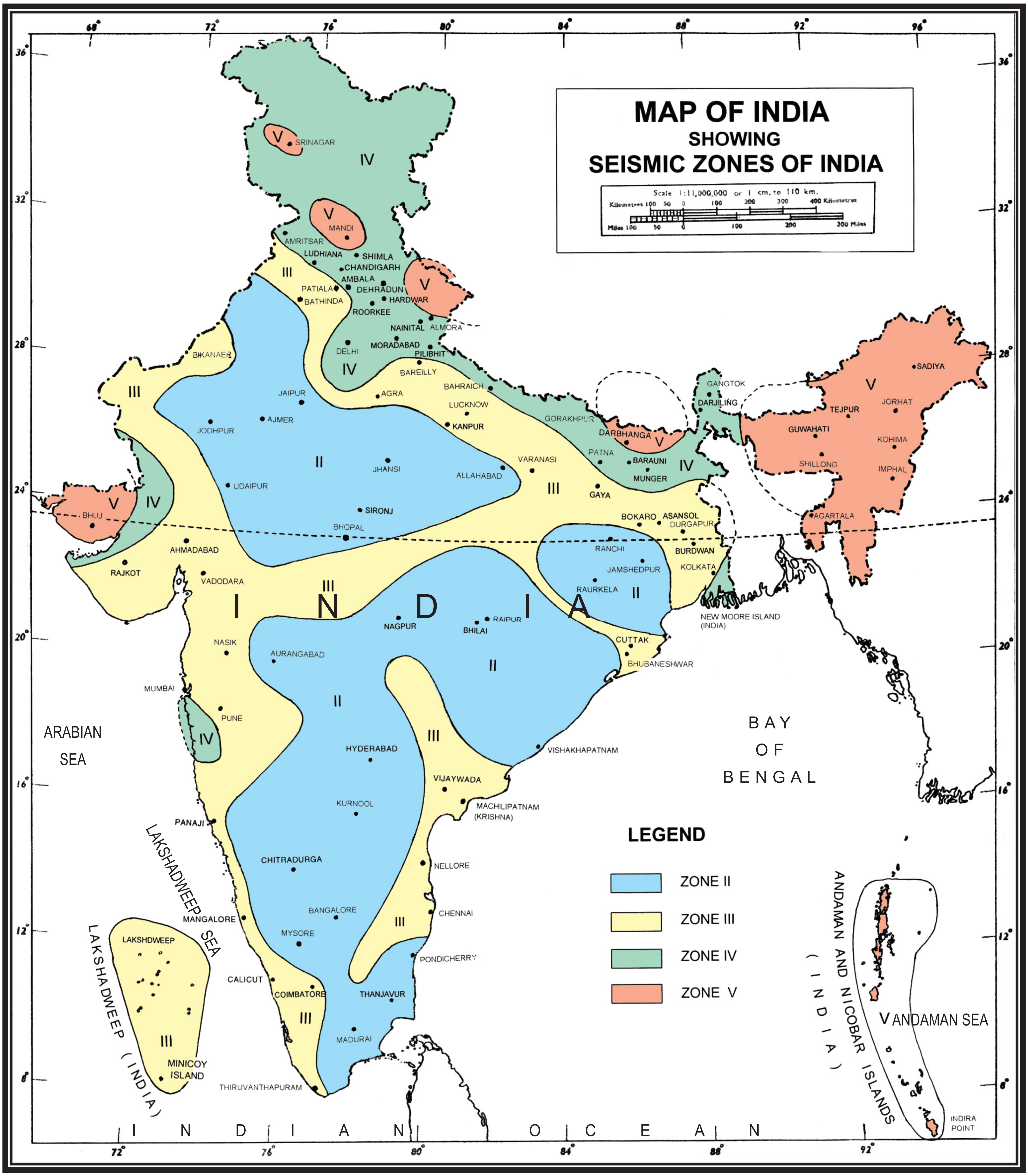
Damping percent	0	2	5	7	10	15	20	25	30
Factors	3.20	1.40	1.00	0.90	0.80	0.70	0.60	0.55	0.50

5.3.4.6 In case design spectrum is specifically prepared for a structure at a particular project site, the same may be used for design at the discretion of the project authorities.

5.4 Buildings

5.4.1 *Regular and Irregular Configuration*

To perform well in an earthquake, a building should possess four main attributes, namely, simple and regular configuration, and adequate lateral strength, stiffness and ductility. Buildings having simple regular in plan as well as in elevation, suffer much less damage



Based upon Survey of India Outline Map printed in 1993.

The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.
The boundary of Meghalaya shown on this map is as interpreted from the North-Eastern Areas (Reorganisation) Act, 1971, but has yet to be verified.
Responsibility for correctness of internal details shown on the map rests with the publisher.
The state boundaries between Uttaranchal & Uttar Pradesh, Bihar & Jharkhand and Chhatisgarh & Madhya Pradesh have not been verified by Governments concerned.

NOTE — Towns falling at the boundary of zones demarcation line between two zones shall be considered in higher zone.

FIG. 11 SEISMIC ZONES

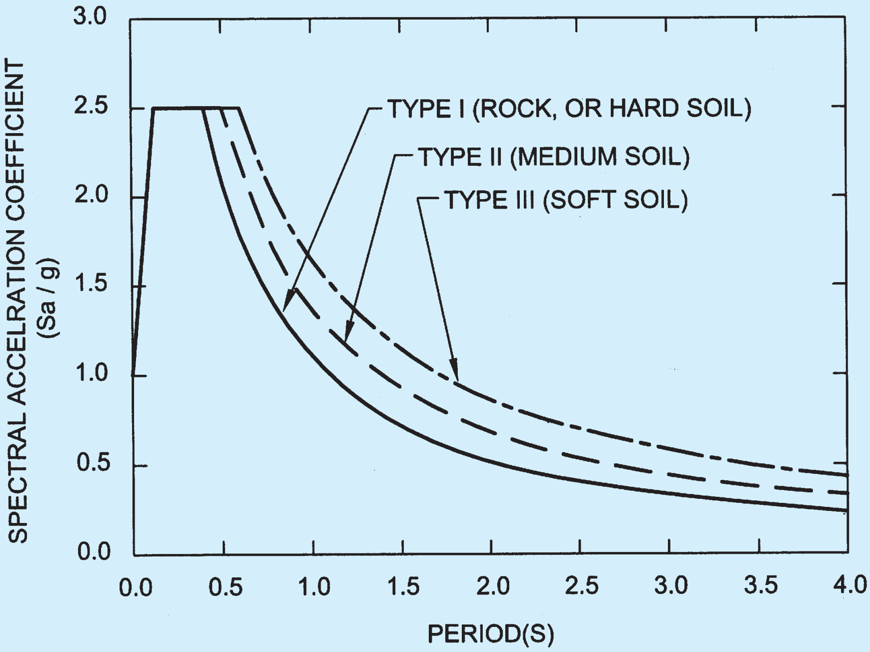


FIG. 12 RESPONSE SPECTRA FOR ROCK AND SOIL SIGHT FOR 5 PERCENT DAMPING

than buildings with irregular configurations. A building shall be considered as irregular for the purposes of this standard, if at least one of the conditions given in Tables 33 and 34 is applicable.

Table 33 Definitions of Irregular Buildings — Plan Irregularities (Fig. 13)

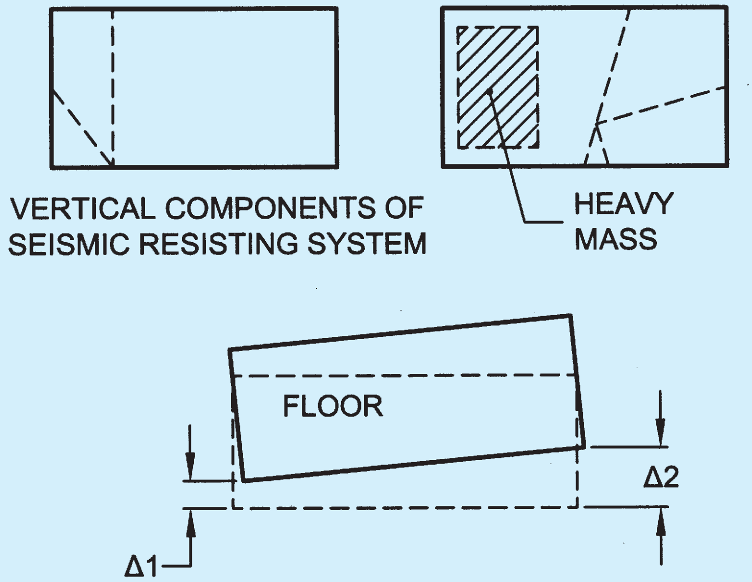
(*Clause* 5.4.1)

Sl No.	Irregularity Type and Description
i)	*Torsion Irregularity* To be considered when floor diaphragms are rigid in their own plan in relation to the vertical structural elements that resist the lateral forces. Torsional irregularity to be considered to exist when the maximum storey drift, computed with design eccentricity, at one end of the structures transverse to an axis is more than 1.2 times the average of the storey drifts at the two ends of the structure.
ii)	*Re-entrant Corners* Plan configurations of a structure and its lateral force resisting system contain re-entrant corners, where both projections of the structure beyond the re-entrant corner are greater than 15 percent of its plan dimension in the given direction.
iii)	*Diaphragm Discontinuity* Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next.
iv)	*Out- of- Plane Offsets* Discontinuities in a lateral force resistance path, such as out-of-plane offsets of vertical elements.
v)	*Non-parallel Systems* The vertical elements resisting the lateral force are not parallel to or symmetric about the major orthogonal axes or the lateral force resisting elements.

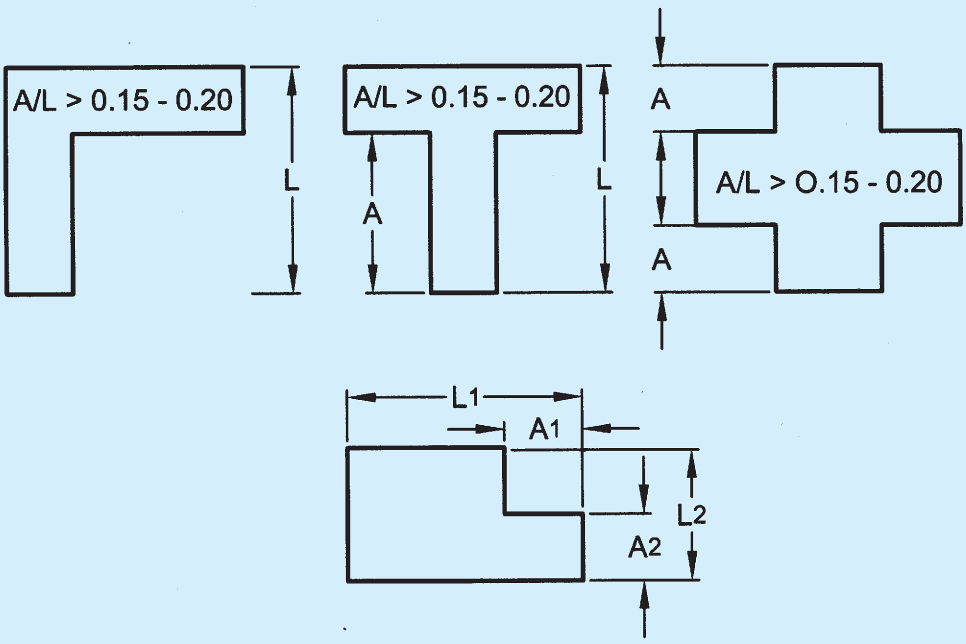
Table 34 Definition of Irregular Buildings — Vertical Irregularities (Fig. 14)

(*Clause* 5.4.1)

Sl No.	Irregularity Type and Description
i)	a) *Stiffness Irregularity — Soft Storey* A soft storey is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above. b) *Stiffness Irregularity — Extreme Soft Storey* An extreme soft storey is one in which the lateral stiffness is less than 60 percent of that in the storey above or less than 70 percent of the average stiffness of the three storeys above. For example, buildings on STILTS will fall under this category.
ii)	*Mass Irregularity* Mass irregularity shall be considered to exist where the seismic weight of any storey is more than 200 percent of that of its adjacent storeys. The irregularity need not be considered in case of roofs.
iii)	*Vertical Geometric Irregularity* Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force resisting system in any storey is more than 150 percent of that in its adjacent storey.
iv)	*In-Plane Discontinuity in Vertical Elements Resisting Lateral Force* An in-plane offset of the lateral force resisting elements greater than the length of those elements.
v)	*Discontinuity in Capacity — Weak Strorey* A weak storey is one in which the storey lateral strength is less than 80 percent of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction.



13 A TORSIONAL IRREGULARITY



13 B RE - ENTRANT CORNER

FIG. 13 PLAN IRREGULARITIES — *Continued*

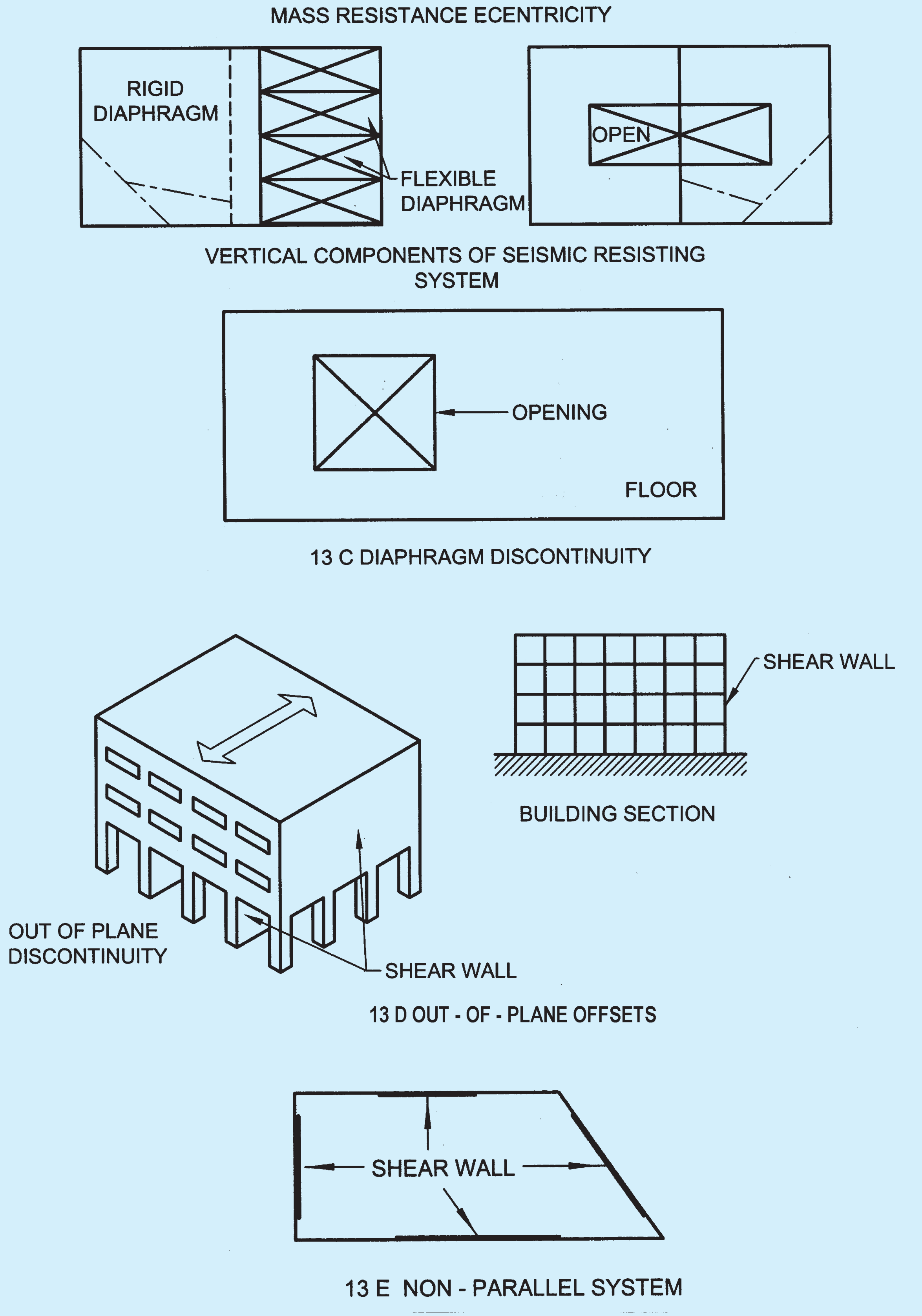
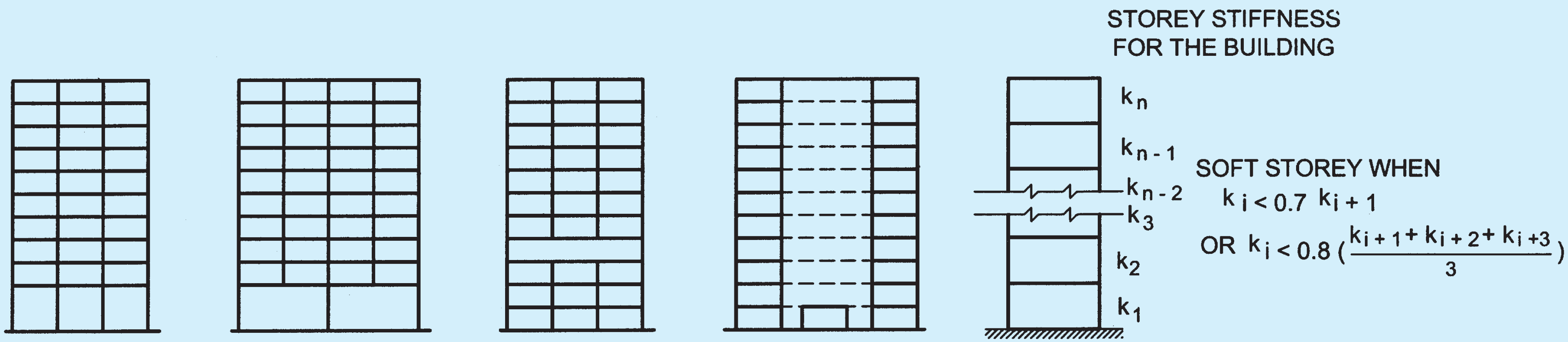
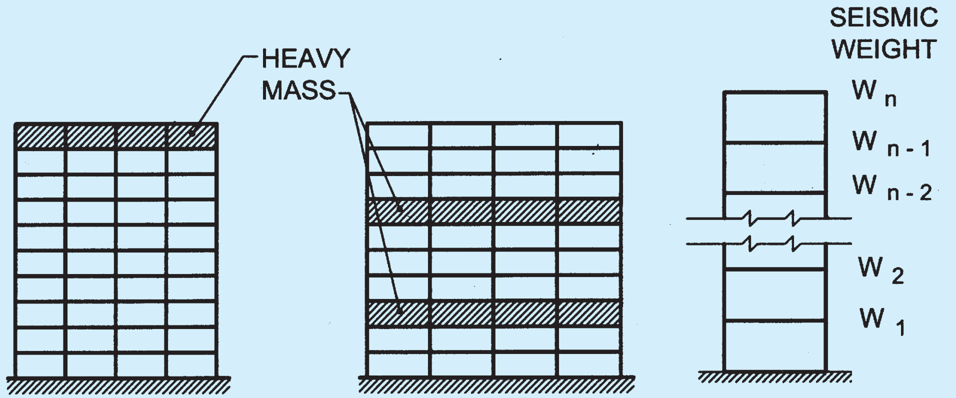


FIG. 13 PLAN IRREGULARITIES



14 A STIFFNESS IRREGULARITY

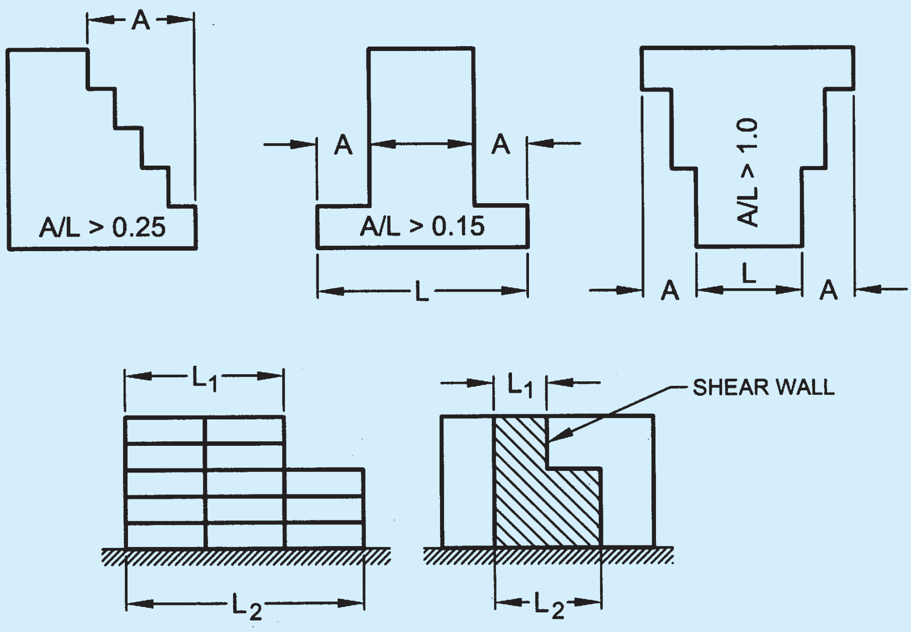


MASS RATIO

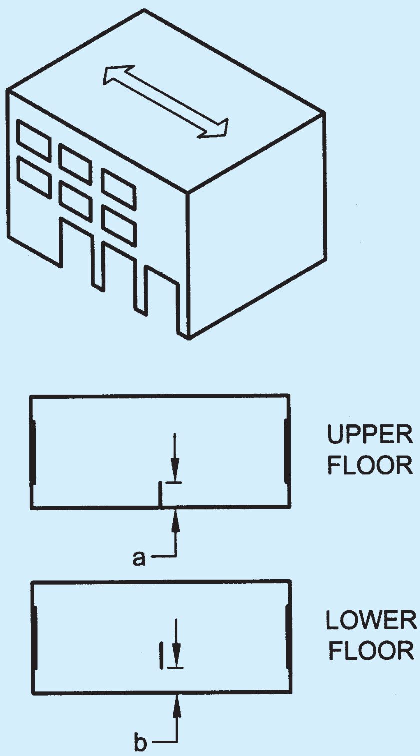
MASS IRREGULARITY
WHEN, $W_i > 2.0\ W_{i-1}$
or $W_i > 2.0\ W_{i+1}$

14 B MASS IRREGULARITY

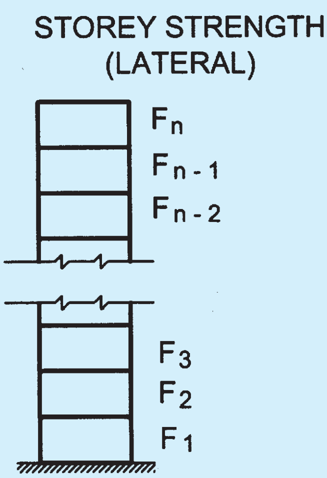
FIG. 14 VERTICAL IRREGULARITIES — *Continued*



14 C VERTICAL GEOMETRIC IRREGULARITY, WHEN $L_2 > 1.5\ L_1$



14 D IN - PLANE DISCONTINUITY VERTICAL ELEMENTS RESISTING LATERAL FORCE, WHEN b > a



14 E WEAK STOREY, WHEN $F_i < 0.8\ F_i + 1$

Fig. 14 Vertical Irregularities

5.4.2 *Importance Factor I and Response Reduction Factor R*

The minimum value of importance factor, *I*, for different building systems shall be as given in Table 35. The response reduction factor, *R*, for different building systems shall be as given in Table 36.

5.4.3 *Design Imposed Loads for Earthquake Force Calculation*

5.4.3.1 For various loading classes as specified in IS 875 (Part 2), the earthquake force shall be calculated for the full dead load plus the percentage of imposed load as given in Table 37.

5.4.3.2 For calculating the design seismic forces of the structure, the imposed load on roof need not be considered.

5.4.3.3 The percentage of imposed loads given in **5.3.3.1** and **5.3.3.2** shall also be used for 'Whole frame loaded' condition in the load combinations specified in **5.3.3.1.1** and **5.3.3.1.2** where the gravity loads are combined with the earthquake loads [that is in load combinations (a) in **5.3.3.1.1**, and (b) in **5.3.3.1.2**]. No further reduction in the imposed load will be used as envisaged in **3** for number of storeys above the one under consideration or for large spans of beams or floors.

5.4.3.4 The proportions of imposed load indicated above for calculating the lateral design forces for earthquakes are applicable to average conditions. Where the probable loads at the time of earthquake are more accurately assessed, the designer may alter the proportions indicated or even replace the entire

Table 35 Importance Factors, *I*

(*Clauses* 5.3.4.2 *and* 5.4.2)

Sl No.	Structure	Importance Factor
i)	Important service and community buildings, such as hospitals; schools; monumental structures; emergency buildings like telephone exchange, television stations, radio stations, railway stations, fire station buildings; large community halls like cinemas, assembly halls and subway stations, power stations.	1.5
ii)	All other buildings.	1.0

NOTES

1 The design engineer may choose values of importance factor *I* greater than those mentioned above.

2 Buildings not covered in Sl No. (i) and (ii) above may be designed for higher value of *I*, depending on economy, strategy considerations like multi-storey buildings having several residential units.

3 This does not apply to temporary structures like excavations, scaffolding etc of short duration.

Table 36 Response Reduction Factor[1] *R*, for Building Systems

(*Clauses* 5.3.4.2 *and* 5.4.2)

Sl No.	Lateral Load Resisting System	R
	Building Frame Systems	
i)	Ordinary RC Moment-Resisting Frame (OMRF)[2]	3.0
ii)	Special RC Moment-Resisting Frame (SMRF)[3]	5.0
iii)	Steel Frame with	
	a) Concentric Braces	4.0
	b) Eccentric Braces	5.0
iv)	Steel Moment Resisting Frame designed as per SP: 6(6)	5.0
	Building with Shear Walls [4]	
v)	Load Bearing Masonry Wall Buildings [5]	
	a) Unreinforced	1.5
	b) Reinforced with horizontal RC Bands	2.5
	c) Reinforced with horizontal RC bands and vertical bars at corners of rooms and jambs of openings	3.0
vi)	Ordinary Reinforced Concrete Shear Walls [6]	3.0
vii)	Ductile Shear Walls [7]	4.0
	Buildings with Dual Systems [8]	
viii)	Ordinary Shear Wall with OMRF	3.0
ix)	Ordinary Shear Wall with SMRF	4.0
x)	Ductile Shear Wall with OMRF	4.5
xi)	Ductile Shear Wall with SMRF	5.0

[1] The above values of response reduction factors are to be used for buildings with lateral load resisting elements, and not just for the lateral load resisting elements built in isolation.

[2] OMRF are those designed and detailed as per IS 456 or IS 800 but not meeting ductile detailing requirement as per IS 13920.

[3] SMRF defined in **4.15.2**.

[4] Buildings with shear walls also include buildings having shear walls and frames, but where

a) frames are not designed to carry lateral loads, or

b) frames are designed to carry lateral loads but do not fulfil the requirements of 'dual systems'.

[5] Reinforcement should be as per IS 4326.

[6] Prohibited in Zones IV and V.

[7] Ductile shear walls are those designed and detailed as per IS 13920

[8] Buildings with dual systems consist of shear walls (or braced frames) and moment resisting frames such that

a) the two systems are designed to resist the total design force in proportion to their lateral stiffness considering the interaction of the dual system at all floor levels; and

b) the moment resisting frames are designed to independently resist at least 25 percent of the design seismic base shear.

Table 37 Percentage of Imposed Load to be Considered in Seismic Weight Calculation

(*Clause* 5.4.3.1)

Imposed Uniformity Distributed Floor Loads (kN/sq.m)	Percentage of Imposed Load
Up to and including 3.0	25
Above 3.0	50

imposed load proportions by the actual assessed load. In such cases, where the imposed load is not assessed as per **5.4.3.1** and **5.4.3.2** only that part of imposed load, which possesses mass, shall be considered. Lateral design force for earthquakes shall not be calculated on contribution of impact effects from imposed loads.

5.4.3.5 Other loads apart from those given above (for example, snow and permanent equipment) shall be considered as appropriate.

5.4.4 *Seismic Weight*

5.4.4.1 *Seismic weight of floors*

The seismic weight of each floor is its full dead load plus appropriate amount of imposed load, as specified in **5.4.3.1** and **5.4.3.2**. While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be equally distributed to the floors above and below the storey.

5.4.4.2 *Seismic weight of building*

The seismic weight of the whole building is the sum of the seismic weights of all the floors.

5.4.4.3 Any weight supported in between storeys shall be distributed to the floors above and below in inverse proportion to its distance from the floors.

5.4.5 *Design Lateral Force*

5.4.5.1 Buildings and portions thereof shall be designed and constructed, to resist the effects of design lateral force specified in **5.4.5.3** as a minimum.

5.4.5.2 The design lateral force shall first be computed for the building as a whole. This design lateral force shall then be distributed to the various floor levels. The overall design seismic force thus obtained at each floor level, shall then be distributed to individual lateral load resisting elements depending on the floor diaphragm action.

5.4.5.3 *Design seismic base shear* — The total design lateral force or seismic base shear (V_B) along any principal direction shall be determined by the following expression:

$$V_B = A_h W$$

where

- A_h = Design horizontal acceleration spectrum value as per **5.3.4.2**, using the fundamental natural period as per **5.4.6** in the considered direction of vibration; and
- W = Seismic weight of the building as per **5.4.4.2**.

5.4.6 *Fundamental Natural Period*

5.4.6.1 The approximate fundamental natural period of vibration (T_a), in seconds, of a moment-resisting frame building without brick infill panels may be estimated by the empirical expression:

$$T_a = 0.075\ h^{0.75} \qquad \text{for RC Frame Building}$$

$$= 0.085\ h^{0.75} \qquad \text{for Steel Frame Building}$$

where

- h = Height of building, in metres. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But, it includes the basement storeys, when they are not so connected.

5.4.6.2 The approximate fundamental natural period of vibration (T_a), in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

$$T_a = \frac{0.09\ h}{\sqrt{d}}$$

where

- h = Height of building, in metres, as defined in **5.4.6.1**; and
- d = Base dimension of the building at the plinth level, in metres; and along the considered direction of the lateral force.

5.4.7 *Distribution of Design Force*

5.4.7.1 *Vertical distribution of base shear to different floor levels*

The design base shear (V_B) computed in **5.4.5.3** shall be distributed along the height of the building as per the following expression:

$$Q_i = V_B \frac{W_i\ h_i^2}{\sum_{j=1}^{n} W_j\ h_j^2}$$

where

- Q_i = Design lateral force at floor *i*,
- W_i = Seismic weight of floor *i*,
- h_i = Height of floor *i* measured from base, and
- n = Number of storeys in the building is the number of levels at which the masses are located.

5.4.7.2 *Distribution of horizontal design lateral force to different lateral force resisting elements*

5.4.7.2.1 In case of buildings whose floors are capable of providing rigid horizontal diaphragm action, the total shear in any horizontal plane shall be distributed to the various vertical elements of lateral force resisting system, assuming the floors to be infinitely rigid in the horizontal plane.

5.4.7.2.2 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.

NOTES

1 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.

2 Reinforced concrete monolithic slab-beam floors or those consisting of Prefabricated/Precast elements with topping reinforced screed can be taken a rigid diaphragms.

5.4.8 *Dynamic Analysis*

5.4.8.1 Dynamic analysis shall be performed to obtain the design seismic force, and its distribution to different levels along the height of the building and to the various lateral load resisting elements, for the following buildings:

a) *Regular buildings* — Those greater than 40 m in height in Zones IV and V, and those greater than 90 m in height in Zones II and III. Modelling as per **5.4.8.4.5** can be used.

b) *Irregular buildings (as defined in* **5.4.1***)* — All framed buildings higher than 12 m in Zones IV and V, and those greater than 40 m in height in Zones II and III.

The analytical model for dynamic analysis of buildings with unusual configuration should be such that it adequately models the types of irregularities present in the building configuration. Buildings with plan irregularities, as defined in Table 33 (as per **5.4.1**), cannot be modelled for dynamic analysis by the method given in **5.4.8.4.5**

NOTE — For irregular buildings, lesser than 40 m in height in Zones II and III, dynamic analysis, even though not mandatory, is recommended.

5.4.8.2 Dynamic analysis may be performed either by the Time History Method or by the Response Spectrum Method. However, in either method, the design base shear (V_B) shall be compared with a base shear (V_B) calculated using a fundamental period T_a, where T_a is as per **5.4.6**. Where V_B is less than V_B, all the response quantities (for example, member forces, displacements, storey forces, storey shears and base reactions) shall be multiplied by $\bar{V}_B / V_B$).

5.4.8.2.1 The value of damping for buildings may be taken as 2 and 5 percent of the critical, for the purposes of dynamic analysis of steel and reinforced concrete buildings, respectively.

5.4.8.3 *Time history method*

Time history method of analysis, when used, shall be based on an appropriate ground motion and shall be performed using accepted principles of dynamics.

5.4.8.4 *Response spectrum method*

Response spectrum method of analysis shall be performed using the design spectrum specified by **5.3.4.2**, or by a site-specific design spectrum mentioned in **5.3.4.6**.

5.4.8.4.1 *Free vibration analysis*

Undamped free vibration analysis of the entire building shall be performed as per established methods of mechanics using the appropriate masses and elastic stiffness of the structural system, to obtain natural periods (T) and mode shapes $\{\emptyset\}$ of those of its modes of vibration that need to be considered as per **5.4.8.4.2**.

5.4.8.4.2 *Modes to be considered*

The number of modes to be used in the analysis should be such that the sum total of modal masses of all modes considered is at least 90 percent of the total seismic mass and missing mass correction beyond 33 percent. If modes with natural frequency beyond 33 Hz are to be considered, modal combination shall be carried out only for modes up to 33 Hz. The effect of higher modes shall be included by considering missing mass correction following well established procedures.

5.4.8.4.3 *Analysis of building subjected to design forces*

The building may be analysed by accepted principles of mechanics for the design forces considered as static forces.

5.4.8.4.4 *Modal Combination*

The peak response quantities (for example, member forces, displacements, storey forces, storey shears and base reactions) shall be combined as per Complete Quadratic Combination (CQC) method.

$$\lambda = \sqrt{\sum_{i=1}^{r} \sum_{j=1}^{r} \lambda_i \ \rho_{ij} \ \lambda_j}$$

where

r = Number of modes being considered,

λ_i = Response quantity in i th mode,

ρ_{ij} = Cross-modal coefficient i (including sign), and

λ_j = Response quantity in mode j (including sign).

$$\rho_{ij} = \frac{8\varsigma^2 \ (1+\beta) \ \beta^{1.5}}{(1-\beta^2)^2 + 4\varsigma^2 \beta(1+\beta)^2}$$

ς = Modal damping ratio (in fraction) as specified in **5.4.8.2.1**,

β = Frequency ratio = ω_j/ω_i

ω_i = circular frequency in *i*th mode, and

ω_j = circular frequency in *j*th mode.

Alternatively, the peak response quantities may be combined as follows:

a) If the building does not have closely-spaced modes, then the peak response quantity (λ) due to all modes considered shall be obtained as

$$\lambda = \sqrt{\sum_{K=1}^{r} (\lambda_K)^2}$$

where

λ_k = Absolute value of quantity in mode *k*.

b) If the building has a few closely-spaced modes (*see* **5.3.3.2**), then the peak response quantity (λ^*) due to these modes shall be obtained as

$$\lambda^* = \sum_{c} \lambda_{c}$$

where the summation is for the closely-spaced modes only. This peak response quantity due to the closely spaced modes (λ^*) is then combined with those of the remaining well-separated modes by the method described in **5.4.8.4.4** (a).

5.4.8.4.5 Buildings with regular, or nominally irregular, plan configurations may be modelled as a system of masses lumped at the floor levels with each mass having one degree of freedom, that of lateral displacement in the direction under consideration. In such a case, the following expressions shall hold in the computation of the various quantities:

a) *Modal Mass* — The modal mass (M_k) of mode *k* is given by

$$M_K = \frac{\left[\sum_{i=1}^{N} W_i \ \phi_{ik} \right]^2}{g \sum_{i=1}^{N} W_i \ (\phi_{ik})^2}$$

where

g = Acceleration due to gravity,

ϕ_{ik} = Mode shape coefficient at floor *i* in mode *k*, and

W_i = Seismic weight of floor *i*.

b) *Modal Participation Factors* — The modal participation factor (P_K) of mode *k* is given by

$$P_K = \frac{\sum_{i=1}^{n} W_i \ \phi_{ik}}{\sum_{i=1}^{n} W_i \ (\phi_{ik})^2}$$

c) *Design lateral Force at Each Floor in Each Mode* — The peak lateral force (Q_{ik}) at floor *i* in mode *k* is given by

$$Q_{ik} = A_K \ \phi_{ik} \ P_K \ W_i$$

where

A_K = Design horizontal acceleration spectrum value as per **5.3.4.2** using the natural period of vibration (T_k) of mode *k*.

c) *Storey Shear Forces in Each Mode* — The peak shear force (V_{iK}) acting in storey *i* in mode *k* is given by

$$V_{iK} = \sum_{j=i+1}^{n} Q_{iK}$$

d) *Storey Shear Forces due to All Modes Considered* — The peak storey shear force (V_i) in storey *i* due to all modes considered is obtained by combining those due to each mode in accordance with **5.4.8.4.4**.

e) *Lateral Forces at Each Storey due to All Modes Considered* — The design lateral forces, F_{roof} and F_i, at roof and at floor *i*:

$$F_{roof} = V_{roof} \text{ and}$$

$$F_i = V_i - V_{i+1}$$

5.4.9 *Torsion*

5.4.9.1 Provision shall be made in all buildings for increase in shear forces on the lateral force resisting elements resulting from the horizontal torsional moment arising due to eccentricity between the centre of mass and centre of rigidity. The design forces calculated as in **5.4.8.4.5** are to be applied at the centre of mass appropriately displaced so as to cause design eccentricity (**5.4.9.2**) between the displaced centre of mass and centre of rigidity. However, negative torsional shear shall be neglected.

5.4.9.2 The design eccentricity, e_{di} to be used at floor i shall be taken as

$$e_{di} = \begin{cases} 1.5 \ e_{si} + 0.05 \ b_i \\ \text{or} \ \ e_{si} - 0.05 \ b_i \end{cases}$$

whichever of these gives the more severe effect in the shear of any frame where

e_{di} = static eccentricity at floor i defined as the distance between centre of mass and centre of rigidity and

b_i = floor plan dimension of floor i, perpendicular to the direction of force.

NOTE — The factor 1.5 represents dynamic amplification factor, while the factor 0.05 represents the extent of accidental eccentricity.

5.4.9.3 In case of highly irregular buildings analysed according to **5.4.8.4.5**, additive shears will be superimposed for a statically applied eccentricity of $\pm 0.05b_i$ with respect to the centre of rigidity.

5.4.10 *Buildings with Soft Storey*

5.4.10.1 In case buildings with a flexible storey, such as the ground storey consisting of open spaces for parking that is stilt buildings, special arrangement needs to be made to increase the lateral strength and stiffness of the soft/open storey.

5.4.10.2 Dynamic analysis of building is carried out including the strength and stiffness effects of infills and inelastic deformations in the members, particularly, those in the soft storey, and the members designed accordingly.

5.4.10.3 Alternatively, the following design criteria are to be adopted after carrying out the earthquake analysis, neglecting the effect of infill walls in other storeys:

a) the columns and beams of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads specified in the other relevant clauses; or,

b) besides the columns designed and detailed for the calculated shears and moments, shear walls placed symmetrically in both directions of the building as far away from the centre of the building as feasible; to be designed exclusively for 1.5 times the lateral shear force calculated as before.

5.4.11 *Deformations*

5.4.11.1 *Storey drift limitation*

The storey drift in any storey due to the minimum specified design lateral force, with partial load factor of 1.0, shall not exceed 0.004 times the storey height.

For the purposes of displacement requirements only (that is, in **5.4.11.1**, **5.4.11.2** and **5.4.11.3** only), it is permissible to use seismic force obtained from the computed fundamental period (T) of the building without the lower bound limit on design seismic force specified in **5.4.8.2**.

There shall be no drift limit for single storey building which has been designed to accommodate storey drift.

5.4.11.2 *Deformation compatibility of non-seismic members*

For building located in seismic Zones IV and V, it shall be ensured that the structural components, that are not a part of the seismic force resisting system in the direction under consideration, do not lose their vertical load-carrying capacity under the induced moments resulting from storey deformations equal to R times the storey displacements calculated as per **5.4.11.1,** where R is specified in Table 36.

NOTE — For instance, consider a flat-slab building in which lateral load resistance is provided by shear walls. Since the lateral load resistance of the slab-column system is small, these are often designed only for the gravity loads, while all the seismic force is resisted by the shear walls. Even though the slabs and columns are not required to share the lateral forces, these deform with rest of the structure under seismic force. The concern is that under such deformations, the slab-column system should not lose its vertical load capacity.

5.4.11.3 *Separation between adjacent units*

Two adjacent buildings, or two adjacent units of the same building with separation joint in between shall be separated by a distance equal to the amount R times the sum of the calculated storey displacements as per **5.4.11.1** of each of them, to avoid damaging contact when the two units deflect towards each other. When floor levels of two similar adjacent units or buildings are at the same elevation levels, factor R in this requirement may be replaced by $R/2$.

5.4.12 *Miscellaneous*

5.4.12.1 *Foundations*

The use of foundations vulnerable to significant differential settlement due to ground shaking shall be avoided for structures in seismic Zones III, IV and V. In seismic Zones IV and V, individual spread footings or pile caps shall be interconnected with ties, (*see* **5.2.3.4.1** of IS 4326) except when individual spread footings are directly supported on rock. All ties shall be capable of carrying, in tension and in compression, an axial force equal to $A_h/4$ times the larger of the column or pile cap load, in addition to the otherwise computed forces. Here, A_h is as per **5.3.4.2**.

5.4.12.2 *Cantilever projections*

5.4.12.2.1 *Vertical projections*

Tower, tanks, parapets, smoke stacks (chimneys) and other vertical cantilever projections attached to buildings and projecting above the roof, shall be designed and checked for stability for five times the design horizontal seismic coefficient A_h specified in **5.3.4.2.** In the analysis of the building, the weight of these projecting elements will be lumped with the roof weight.

5.4.12.2.2 *Horizontal projection*

All horizontal projections like cornices and balconies shall be designed and checked for stability for five times the design vertical coefficient specified in **5.3.4.5**.

5.4.12.2.3 The increased design forces specified in **5.4.11.2.1** and **5.4.12.2.2** are only for designing the projecting parts and their connections with the main structures. For the design of the main structure, such increase need not be considered.

5.4.12.3 *Compound walls*

Compound walls shall be designed for the design horizontal coefficient A_h with Importance Factor $I = 1.0$ specified in **6.4.2**

5.4.12.4 *Connections between parts*

All parts of the building, except between the separation sections, shall be tied together to act as integrated single unit. All connections between different parts, such as, beams to columns and columns to their footings, should be made capable of transmitting a force, in all possible directions, of magnitude (Q_i/W_i) times but not less than 0.05 times the weight of the smaller part or the total of dead and imposed load reaction. Frictional resistance shall not be relied upon for fulfilling these requirements.

6 SNOW LOAD

6.1 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 parts of India are subjected to snow fall.

In India, part of Jammu and Kashmir (Baramulah District, Srinagar District, Anantnag District and Ladakh District); Punjab and Himachal Pradesh (Chamba, Kulu Kinnaur District, Mahasu District, Mandi District, Sirmur District and Simla District); and Uttaranchal (Dehra Dun District, Tehri Garhwal District, Almora District and Nainital District) experience snow fall of varying depths two or three times in a year.

6.2 Notations

(Dimensionless) — Nominal values of the shape coefficients, taking tin account snow draft, sliding snow, etc, with subscripts, if necessary.

l_i (metres) — Horizontal dimension with numerical subscripts, if necessary.

h_i (metres) — Vertical dimensions with numerical subscripts, if necessary.

β_i (degrees) — Roof slope.

s_o (pascals) — Snow load on ground.

s_i (pascals) — Snow load on roofs.

6.3 Snow Load in Roof(s)

6.3.1 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:

$$s = \mu\, s_o$$

where

s = Design snow load in Pa on plan area of roof,

μ = Shape coefficient (*see* **5.4**), 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 code are advised to contact either Snow and Avalanches Study Establishment (Defence Research and Development Organization), Manali (HP) or Indian Meteorological Department (IMD), Pune in the absence of any specific information for any location.

6.4 Shape Coefficients

6.4.1 *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.

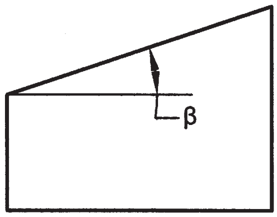
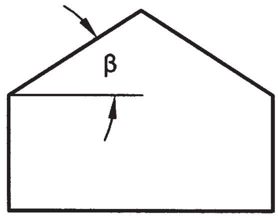
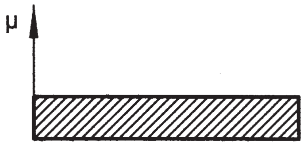
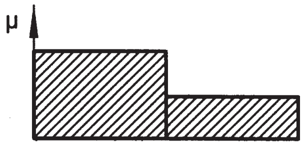
In most regions, snow-falls are accompanied or followed by winds. The winds will re-distribute the snow, and on some roofs especially multilevel roofs, the accumulated drift load may reach a multiple of the ground load. Roofs which are sheltered by other buildings, vegetation, etc, may collect more snow load than the ground level. The phenomenon is of the same nature as that illustrated for multi-level roofs in **6.4.2.4**.

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 **6.4.2**. 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.

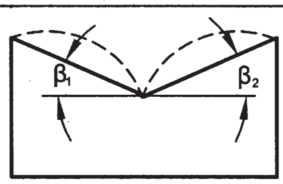
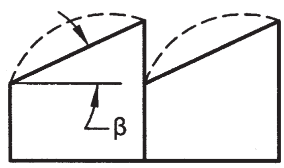
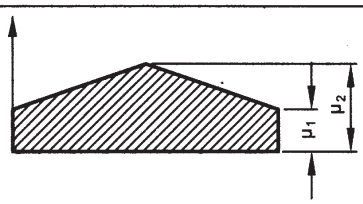
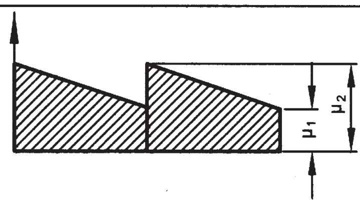
The distribution of snow in the direction parallel to the caves is assumed to be uniform.

6.4.2 *Shape Coefficients for Selected Types of Roofs*

6.4.2.1

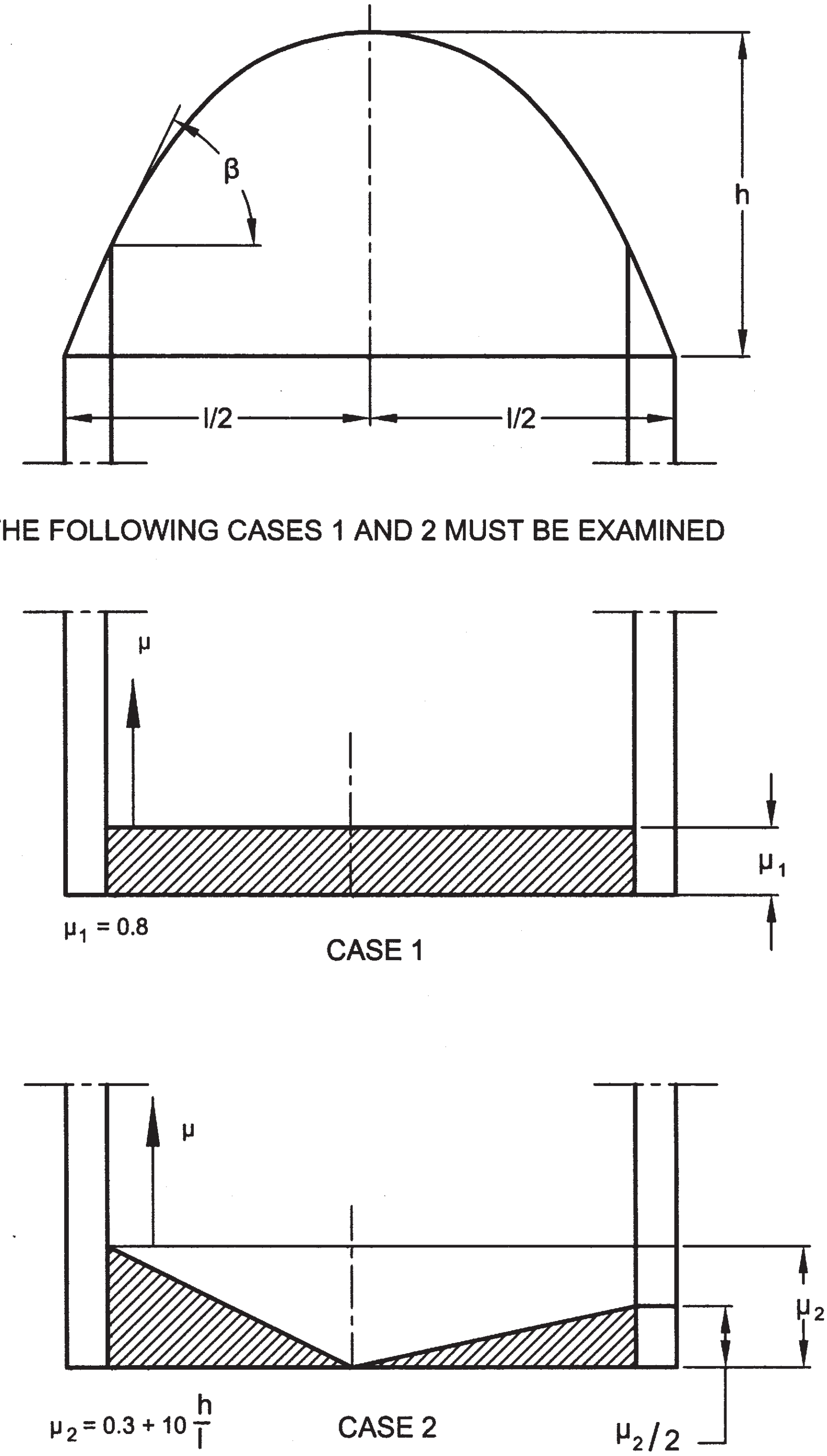
	Simple Flat and Monopitch Roofs	Simple Pitched Roofs (Positive Roof Slope)[1]
		
		
$0° < \beta \leq 30°$	$\mu_1 = 0.8$	$\mu_2 = \mu_1 = 0.8$
$15° < \beta \leq 30°$		$\mu_2 = 0.8 + 0.4\left(\frac{\beta - 15}{15}\right)$ $\mu_1 = 0.8$
$30° < \beta \leq 60°$	$\mu_1 = 0.8\left(\frac{60 - \beta}{30}\right)$	$\mu_2 = 1.2\left(\frac{60 - \beta}{30}\right)$ $\mu_1 = 0.8\left(\frac{60 - \beta}{30}\right)$
$\beta > 60°$	$\mu_1 = 0$	$\mu_2 = \mu_1 = 0$

6.4.2.2

	Simple or Multiple Pitched Roofs (Negative Roof Slope)	Two-Span or Multispan Roofs
	 $\beta = \frac{\beta_1 + \beta_2}{2}$	
		
$0° < \beta < 30°$	$\mu_1 = 0.8\left(\frac{30 + \beta}{30}\right)$ $\mu_1 = 0.8$	$\mu_1 = 0.8\left(\frac{30 + \beta}{30}\right)$ $\mu_1 = 0.8$
$30° < \beta < 60°$	$\mu_2 = 1.6$ $\mu_1 = 0.8\left(\frac{60 - \beta}{30}\right)$	$\mu_2 = 1.6$ $\mu_1 = 0.8\left(\frac{60 - \beta}{30}\right)$
$\beta > 60°$	$\mu_2 = 1.6$ $\mu_1 = 0$	$\mu_2 = 1.6$ $\mu_1 = 0$

[1] For asymmetrical simple pitched roofs, each side of the roof shall be treated as one half of corresponding symmetrical roofs.

6.4.2.3 *Simple curved roofs*

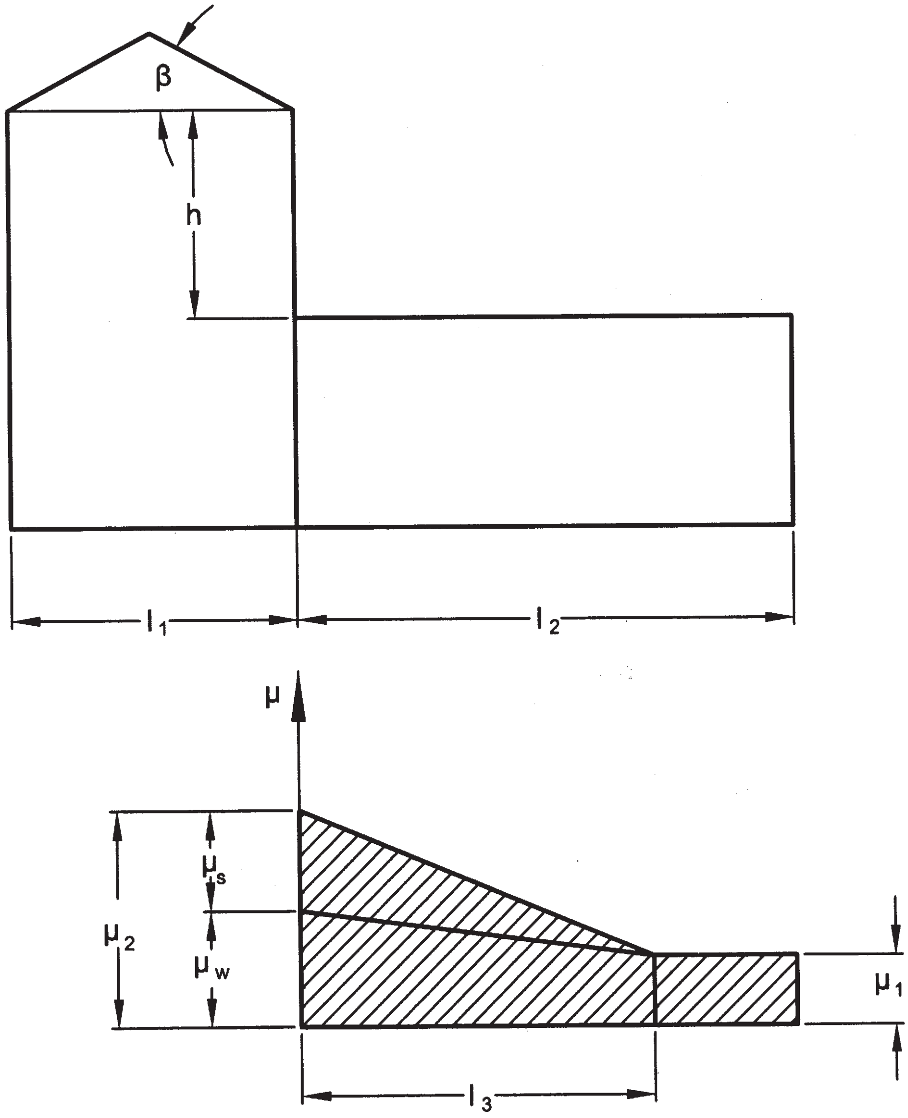


Restriction:

$m_2 \leq 2.3$

$m = 0$ if $b > 60°$

6.4.2.4 *Multi-level roofs*[1)]



$\mu_1 = 0.8$

$\mu_2 = \mu_s + \mu_w$

where

μ_s = due to sliding

μ_w = due to wind

$l_3 = 2h$[2)] but is restricted as follows:

$$5 \text{ m} \leq l_3 \leq 15 \text{ m}$$

$$\mu_w = \frac{l_1 + l_2}{2h} \leq \frac{kh}{s_o}$$

with the restriction $0.8 \leq \mu_w \leq 4.0$

where

h is in metres

s_o is in kilopascals (kilonewtons per square metre)

$k = 2$ kN/m^2

$\beta \geq 15°$: 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[1)], and is distributed linearly as shown in the figure.

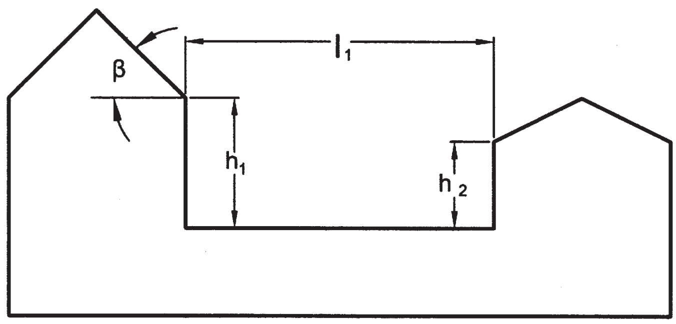
$$\beta \leq 15° : \mu_s = 0$$

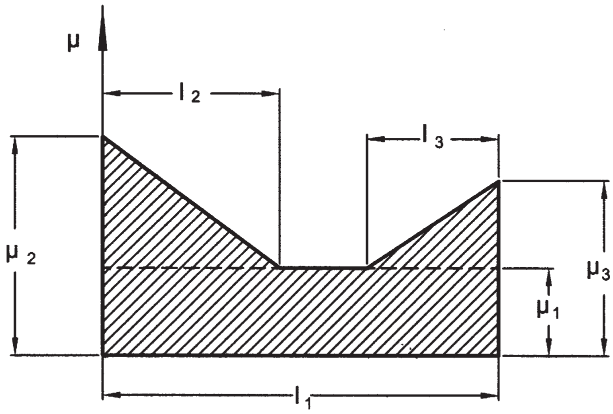
1) A more extensive formula for μ_w is described in Annex A.

2) If $l_2 < l_3$, the coefficient μ is determined by interpolation between μ_1 and μ_2.

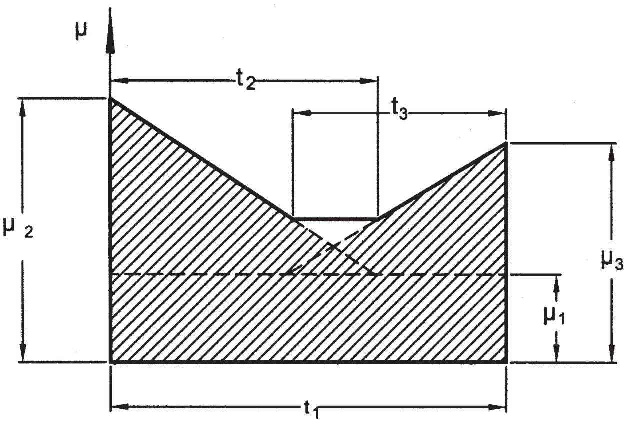
1) The load on the upper roof is calculated according to **6.4.2.1** or **6.4.2.2**.

6.4.2.5 *Complex multi-level roofs*





$l_2 + l_3 < l_1$



$l_2 + l_3 > l_1$

$l_2 = 2h_1 : l_3 = 2h_2 : \mu_1 = 0.8$

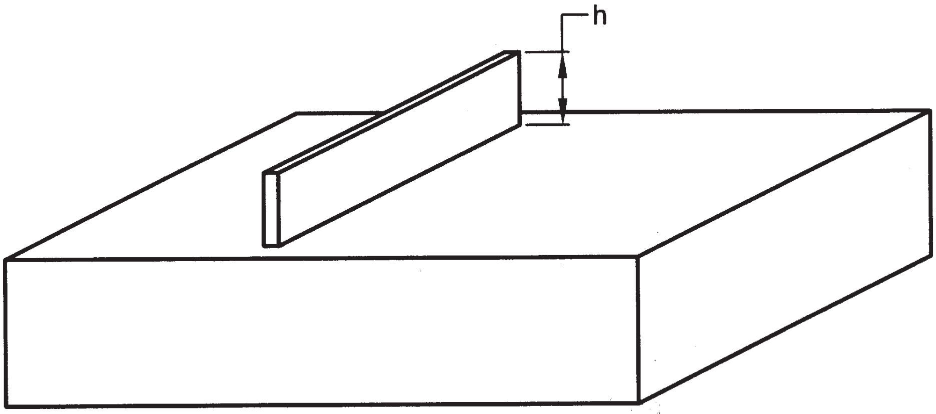
Restriction:

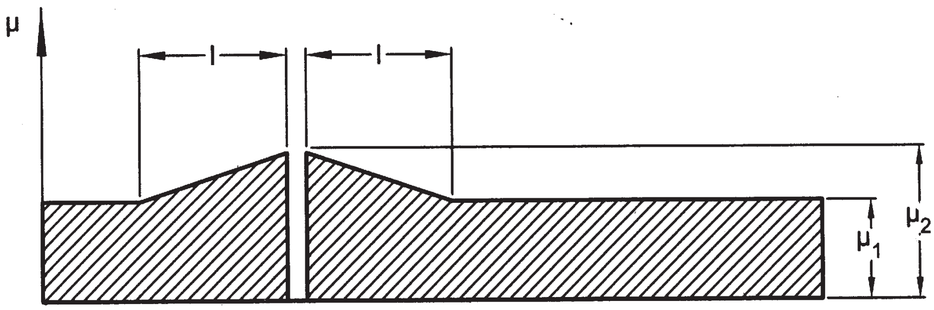
5 m $< l_2 \leq$ 15 m;

5 m $< l_3 \leq$ 15 m;

μ_2 and $\mu_3 = (\mu_s + \mu_w)$, are calculated according to **6.4.2.1**, **6.4.2.2** and **6.4.2.4**.

6.4.2.6 *Roofs with local projections and obstructions*





$$\mu_2 = \frac{kh}{s_o}$$

h = is in metres

s_o = is in kilopascals (kilonewtons per square metre)

k = 2 kN/m^2

μ = 0.8

Restrictions:

$0.8 \leq \mu_w \leq 2.0$

5 m ≤ 1≤ 15 m

6.4.3 *Shape Coefficients in Areas Exposed to Wind*

The shape coefficients given in **6.4.2** and Annex K 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

b) The roof does not have any significant projections, such as, parapet walls which may prevent snow from being blow 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 these areas area:

a) Winter calm valleys in the mountains where sometimes layer after layer of snow accumulates on roofs without any appreciable removal of snow by wind; and

b) Areas (that is, high temperature) where the maximum snow load may be the result of single snowstorm, occasionally without appreciable wind removal.

In such area, the determination of the shape coefficients shall be based on local experience with due regard to the likelihood of wind drifting and sliding.

7 SPECIAL LOADS

7.1 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 judgement (*see also* **3.6**)

7.2 Temperature Effects

7.2.1 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 [6-1(10)] or design the structure to carry additional stresses due to temperature effects as appropriate to the problem.

7.2.1.1 The temperature range varies for different regions and under different diurinal and seasonal conditions. The absolute maximum and minimum temperature which may be expected in different localities in Annex B of Part 6 'Structural Design, Section 6 Steel' respectively. These figures may be used for guidance in assessing the maximum variations of temperature.

7.2.1.2 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.

7.2.1.3 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.

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.

7.3 Hydrostatic and Soil Pressure

7.3.1 In the design of structures of parts or 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 blow the free water surface, the lateral earth pressure shall be evaluated for weight of soil diminished by buoyancy and the full hydrostatic pressure.

7.3.1.1 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.

7.3.2 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. 15).

Actual Width of Member	*Ratio of Effective Width to Actual Width*
Less than 0.5 m	3.0
Beyond 0.5 m and up to 1 m	3.0 to 2.0
Beyond 1 m	2.0

The relieving pressure of soil in front of the structural member concerned may generally not be taken into account.

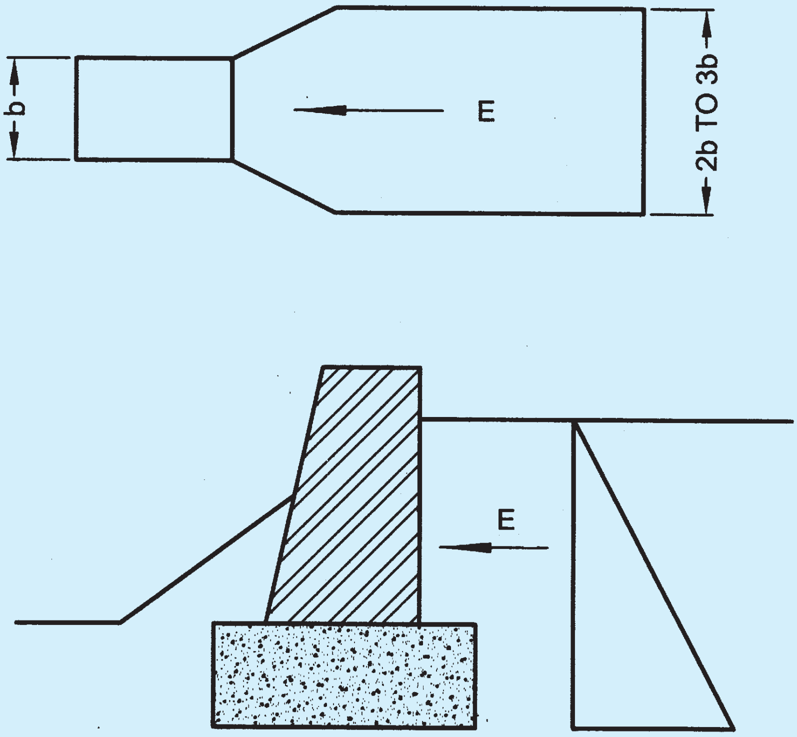


FIG. 15 SKETCH SHOWING EFFECTIVE WIDTH OF PILAR FOR CALCULATING SOIL PRESSURE

7.3.3 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.

7.4 Fatigue

7.4.1 *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 propogate, if loading is continuous.

Where there is such loading cycles, 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.

7.4.2 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.

7.5 Structural Safety During Construction

7.5.1 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.

7.6 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 unfavouable external circumstances).

In most cases, accidental loads only develop under a combination of several unfavourable occurrence. 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.

7.6.1 *Impact and Collisions*

7.6.1.1 *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 **7.6.1.2** and **7.6.1.3**.

7.6.1.2 *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:

m_1 = 400 kg, C_1 = 10 000 kN/m, the vehicle is compressed.

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.

7.6.1.3 *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 **7.6.1.2**.

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.

7.6.1.4 *Crane impact load on buffer stop*

The basic horizontal load P_y (tones), 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 travelling 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, medium-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

g = Acceleration due to gravity (9.81 m/s^2);

P_h = Crane bridge weight (t);

P_t = Crab bridge weight (t);

Q = Crane loading capacity (t);

k = Coefficient, assumed equal to zero for cranes with flexible load suspension and to one for cranes with rigid suspension;

L_k = Crane span (m); and

l = Nearness of crab (m).

7.6.2 *Explosions*

7.6.2.1 *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 **7.6.2.2**.

7.6.2.2 *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).

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.

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 (3-4 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.

If account is taken of the time curve of the action, the schematic correspondence between pressure and time is assumed (Fig. 16), 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. 17.

2 Figure 16 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 16 is given purely as guide and probability of occurrence of an explosion should be checked in each case using appropriate values.

7.6.3 *Vertical Load on Air Raid Shelters*

7.6.3.1 *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:

Buildings up to 2 storeys	28 kN/m^2
Buildings with 3-4 storeys	34 kN/m^2
Buildings with more than 4 storeys	41 kN/m^2
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.

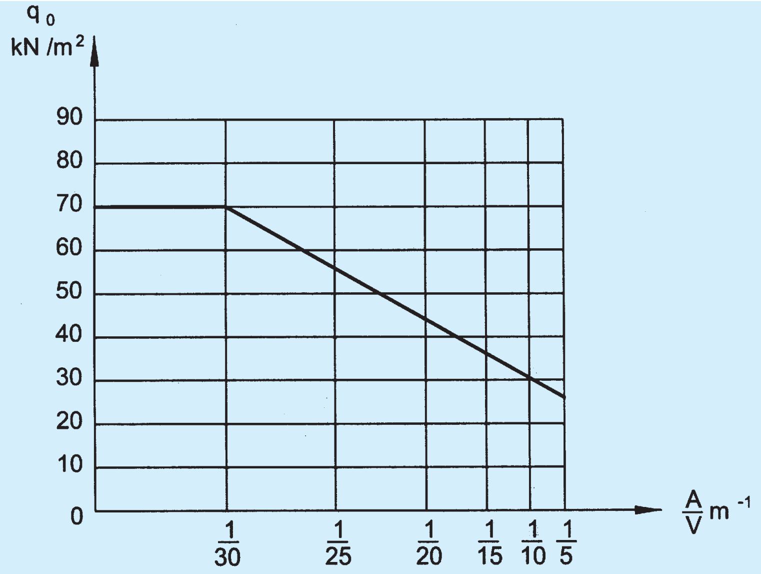


FIG. 16 SKETCH SHOWING RELATION BETWEEN PRESSURE AND TIME

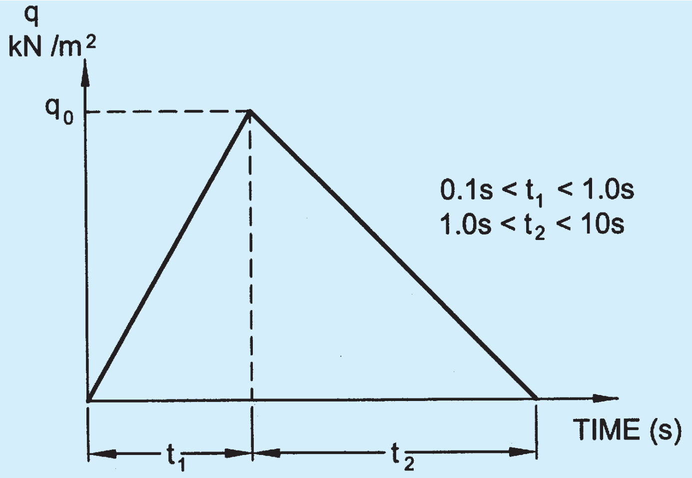


FIG. 17 SKETCH SHOWING TIME INTERVAL AND PRESSURE

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.

7.6.4 *Fire*

7.6.4.1 *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.

7.6.4.2 *Thermal effects during fire*

The thermal effect during fire may be determined from one of the following methods:

a) the time-temperature curve and the required fire resistance (minutes), and
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 per m^2 floor), and
t_b = Required fire resistance (minutes) [*see* 6-1(11)].

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.

7.7 Vibrations

For general details on loads due to vibrations, reference may be made to Annex L.

7.8 Other Loads

Other loads not included in the 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.

7.9 For additional information regarding loads, forces and effects about cyclone resistant buildings and landslide control aspects, reference may be made to good practices [6-1(12)] and 6-1(13)] respectively.

8 LOAD COMBINATIONS

8.1 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.

8.2 Land Combinations

Keeping the aspect specified in **8.1**, 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.

1) *DL*
2) *DL* + *IL*
3) *DL* + *WL*
4) *DL* + *EL*
5) *DL* + *TL*
6) *DL* + *IL* + *WL*
7) *DL* + *IL* + *EL*
8) *DL* + *IL* + *TL*
9) *DL* + *WL* + *TL*
10) *DL* + *EL* + *TL*
11) *DL* + *IL* + *WL*+*TL*
12) *DL* + *IL* + *EL*+*TL*

(*DL* = dead load, *IL* = imposed load, *WL* = wind load, *EL* = earthquake load and *TL* = temperature load).

NOTES

1 When snow load is present on roof's, 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 stale 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.

5 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.

6 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**.

Reduced imposed load specified in **3** for the design of supporting structures should not be applied in combination with earthquake forces.

7 Other loads and accidental load combination not included should be dealt with appropriately.

8 Crane load combinations are covered in **3.6.4**.

9 MULTI-HAZARD RISK IN VARIOUS DISTRICTS OF INDIA

9.1 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.

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.

9.2 Multi-Hazard Prone Areas

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 prossibility 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.

9.3 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 district 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 the such multi-hazard situation.

ANNEX A

[*Clause* 3.3.2.1(b)]

ILLUSTRATIVE EXAMPLE SHOWING REDUCTION OF UNIFORMLY DISTRIBUTED IMPOSED FLOOR LOADS IN MULTI-STOREYED BUILDINGS FOR DESIGN OF COLUMNS

A-1 The total imposed loads from different floor levels (including the roof) combing on the central column of a multi-storeyed building (with mixed occupancy) is shown in Fig. 20. 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.

A-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. 18.

FLOOR No. FROM TOP INCLUDING ROOF	ACTUAL FLOOR LOAD COMING ON DIFFERENT FLOORS kN	LOADS FOR WHICH COLUMNS ARE TO BE DESIGNED 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

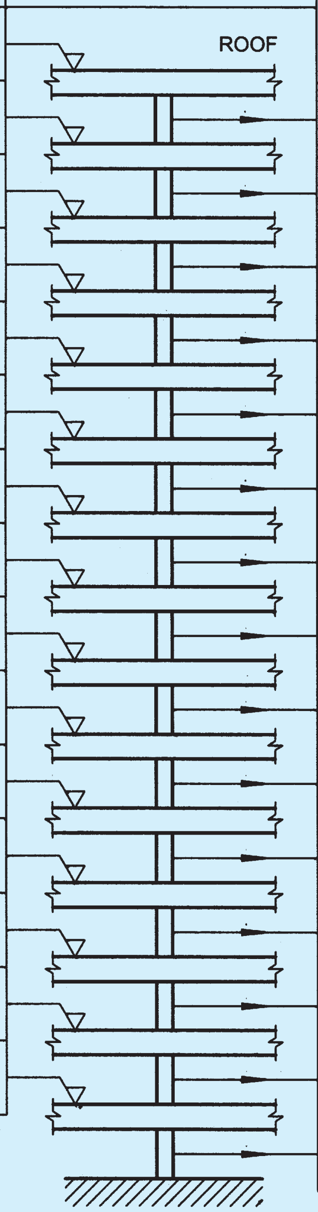


FIG. 18

ANNEX B

(*Clause* 4.2)

NOTATIONS

A = Surface area of a structure or part of a structure
A_e = Effective frontal area
A_z = An area at height Z
b = Breadth of a structure or structural member normal to be wind stream in the horizontal plane
C_f = Force coefficient/drag 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
d = Depth of a structure or structural member parallel to wind stream
D = Diameter of cylinder
F = Force normal to the surface
F_n = Normal force
F_t = Transverse force
F' = Frictional force
h = Height of structure above mean ground level
h_x = The height of development of a velocity profile at a distance x down wind from a change in terrain category
$\left.\begin{matrix} k_1 \\ k_2 \\ k_3 \end{matrix}\right\}$ = Multiplication factors
K = Multiplication factor
l = Length of the member or greater horizontal dimension of a building
p_d = Design wind pressure
p_z = Design wind pressure at height Z
p_e = External pressure
p_i = Internal pressure
R_e = Reynolds number
S = Strouhal number
V_b = Regional basic wind speed
$\bar{V}_b$ = Mean hourly wind speed corresponding to 10 m height
V_z = Design wind velocity at height Z
$\bar{V}_z$ = Hourly mean wind speed at height Z
w = Lesser horizontal dimension of a building or a structural member
w' = Bay within multi-bay buildings
x = Distance down wind from a change in terrain category
θ = Wind angle from a given axis
α = Inclination of the roof to the horizontal
ϕ = Solidity ratio
Z = A height or distance above the ground
ε = Average height of the surface roughness

ANNEX C

(*Clause* 4.4.2)

BASIC WIND SPEED A 10 m HEIGHT FOR SOME IMPORTANT CITIES/TOWNS

City/Town	*Basic Wind Speed* m/s	*City/Town*	*Basic Wind Speed* m/s
Agra	47	Barauni	47
Ahmedabad	39	Bareilly	47
Ajmer	47	Bhatinda	47
Almora	47	Bhilai	39
Amritsar	47	Bhopal	39
Asansol	47	Bhubaneshwar	50
Aurangabad	39	Bhuj	50
Bahraich	47	Bikaner	47
Bangalore	33	Bokaro	47

City/Town	*Basic Wind Speed* m/s	*City/Town*	*Basic Wind Speed* m/s
Calicut	39	Mangalore	39
Chandigarh	47	Moradabad	47
Chennai	50	Mumbai	44
Coimbatore	39	Mysore	33
Cuttack	50	Nagpur	44
Darbhanga	55	Nainital	47
Darjeeling	47	Nasik	39
Dehra Dun	47	Nellore	50
Delhi	47	Panjim	39
Durgapur	47	Patiala	47
Gangtok	47	Patna	47
Guwahati	50	Pondicherry	50
Gaya	39	Portblair	44
Gorakhpur	47	Pune	39
Hyderabad	44	Raipur	39
Imphal	47	Rajkot	39
Jabalpur	47	Ranchi	39
Jaipur	47	Roorkee	39
Jamshedpur	47	Rourkela	39
Jhansi	47	Shimla	39
Jodhpur	47	Srinagar	39
Kanpur	47	Surat	44
Kohima	44	Tiruchchirappalli	47
Kolkata	50	Thiruvananthpuram	39
Kurnool	39	Udaipur	47
Lakshadweep	39	Vadodara	44
Lucknow	47	Varanasi	47
Ludhiana	47	Vijayawada	50
Madurai	39	Vishakhapatnam	50
Mandi	39		

ANNEX D

[*Clause* 4.4.3.2(d)]

CHANGES IN TERRAIN CATEGORIES

D-1 LOW TO HIGH NUMBER

D-1.1 In cases of transitions from a low category number (corresponding to a low terrain roughness) to a high 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.

D-2 HIGH TO LOW NUMBER

D-2.1 In cases of transitions 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 velocities shall be taken as the lesser of the following:

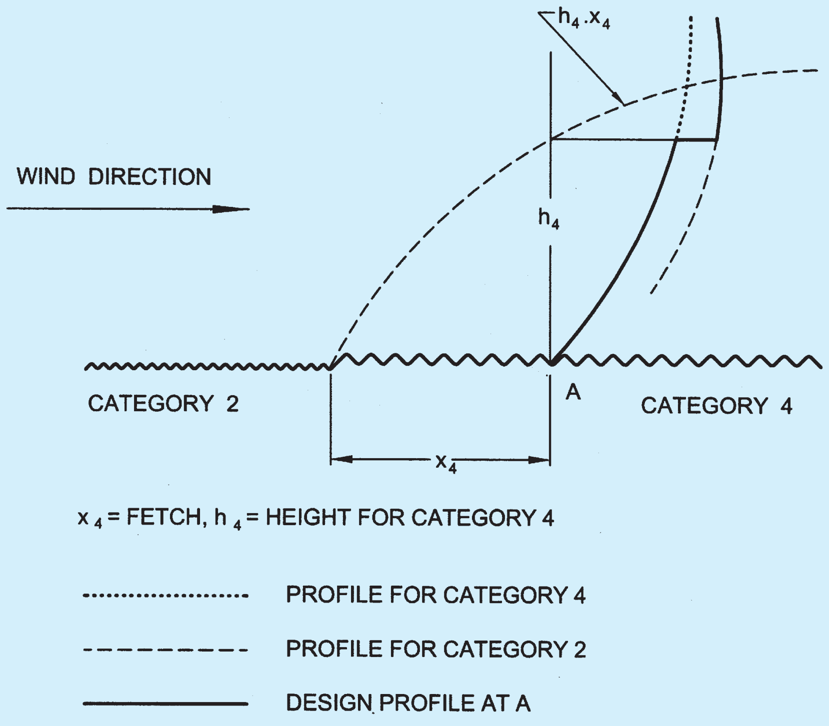
   1) that determined in accordance with the less rough terrain; and
   2) the velocity at height h_x as determined in relation to the rougher terrain.

NOTE — Examples of the determination of velocity profiles in the vicinity of a change in terrain category are shown in Fig. 19 (a) and (b).

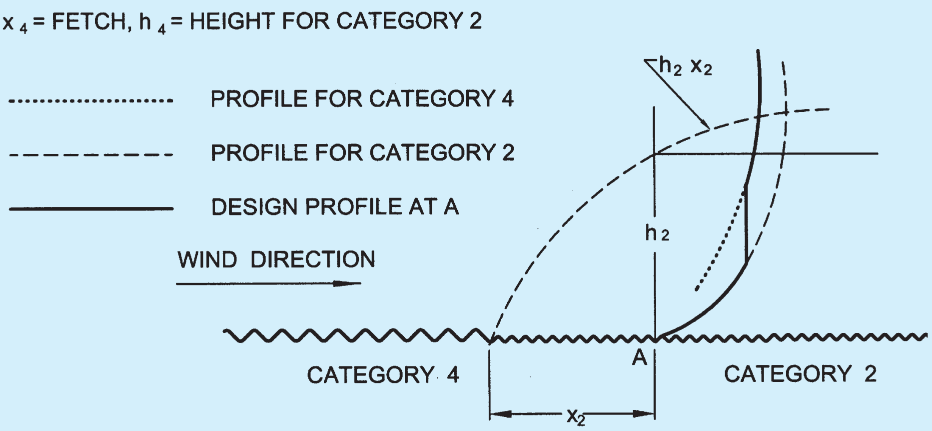
D-3 MORE THAN ONE CATEGORY

D-3.1 Terrain changes involving more than one category shall be treated in similar fashion to that described in **A-1** and **A-2**.

NOTE — Examples involving three terrain categories are shown in Fig. 19(c).

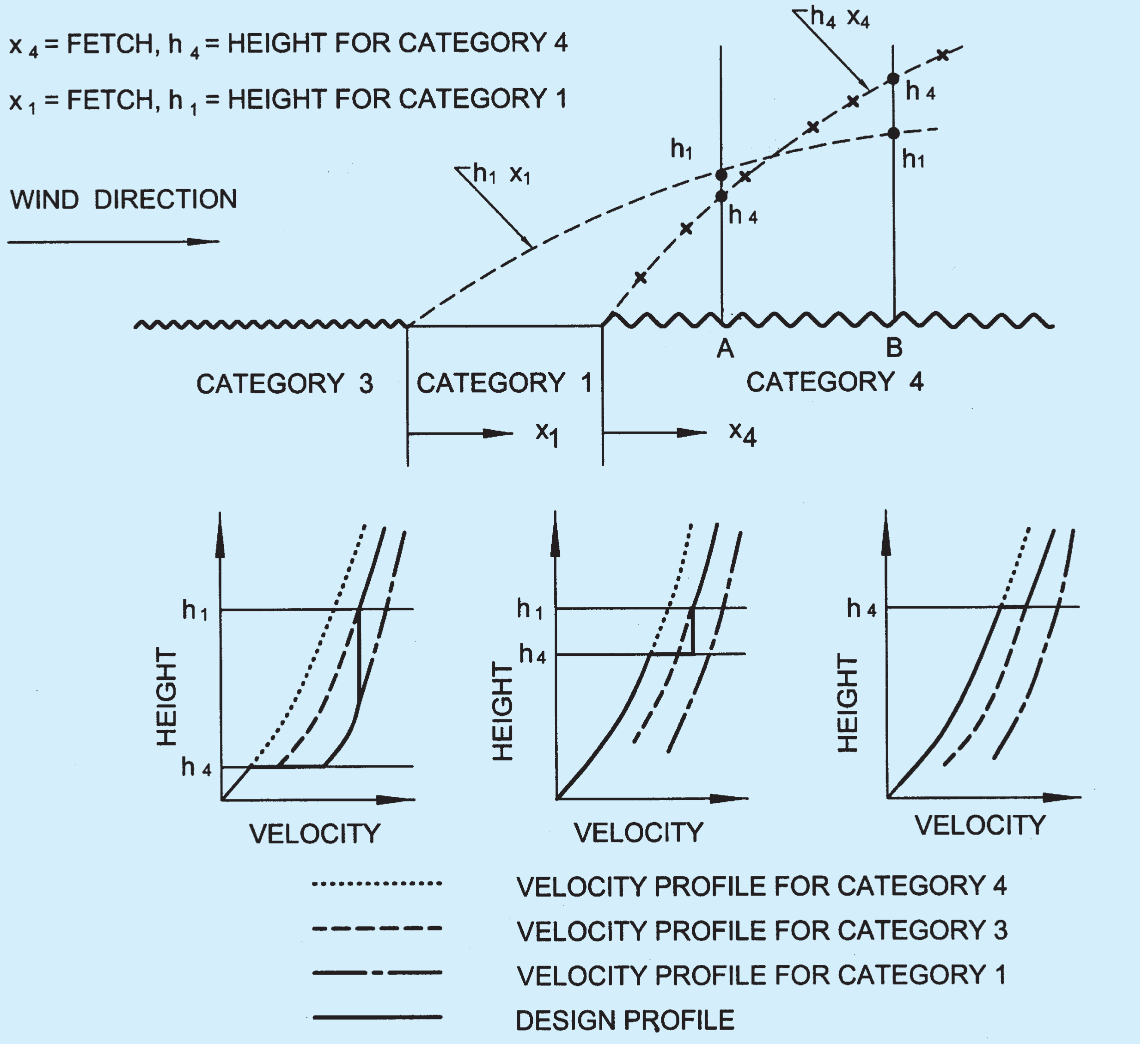


a) DETERMINATION OF VELOCITY PROFILE NEAR A CHANGE IN TERRAIN CATEGORY
(Less rough to more rough)



b) DETERMINATION OF VELOCITY PROFILE NEAR A CHANGE IN TERRAIN CATEGOR
(More rough to less rough)

FIG. 19 VELOCITY PROFILES IN THE VICINITY OF A CHANGE IN TERRAIN CATEGORY — *Continued*



c) DETERMINATION OF DESIGN PROFILE INVOLVING MORE THAN ONE CHANGE TERRAIN CATEGORY

FIG. 19 VELOCITY PROFILES IN THE VICINITY OF A CHANGE IN TERRAIN CATEGORY

ANNEX E

(*Clause* 4.4.3.3)

EFFECT OF A CLIFF OR ESCARPMENT ON THE EQUIVALENT HEIGHT ABOVE GROUND (k_3 FACTOR)

E-1 The influence of the topographic feature is considered to extend 1.5 L_e is the effective horizontal length of the hill depending on slope as indicated below (*see* Fig. 20).

Slope	L_e
$3° < \theta \leq 17°$	L
$> 17°$	$\frac{Z}{0.3}$

where L is the actual length of the upwind slope in the wind direction, Z is the effective height of the feature, and θ is the upwind slope in the wind direction.

If the zone downwind from the crest of the feature is relatively flat, ($\theta < 3°$) for a distance exceeding L_e, then the feature should be treated as an escarpment. If not then the feature should be treated as a hill or ridge. Examples of typical features are given in Fig. 20.

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 L and the upwind slope θ of the feature.

E-2 TOPOGRAPHY FACTOR, k_3

The topography factor k_3 is given by the following:

$$k_3 = 1 + C\,s$$

where C has the following values:

Slope	C
$3° < \theta \le 17°$	$1.2\left(\frac{Z}{L}\right)$
$> 17°$	0.36

s is a factor derived in accordance with **E-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.

E-2.1 The factor s should be determined from:

a) Figure 21 for cliffs and escarpments, and

b) Figure 22 for hills and ridges.

NOTE — Where the downwind slope of a hill or ridge is greater 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 from Fig. 22 may be used as upper bound values.

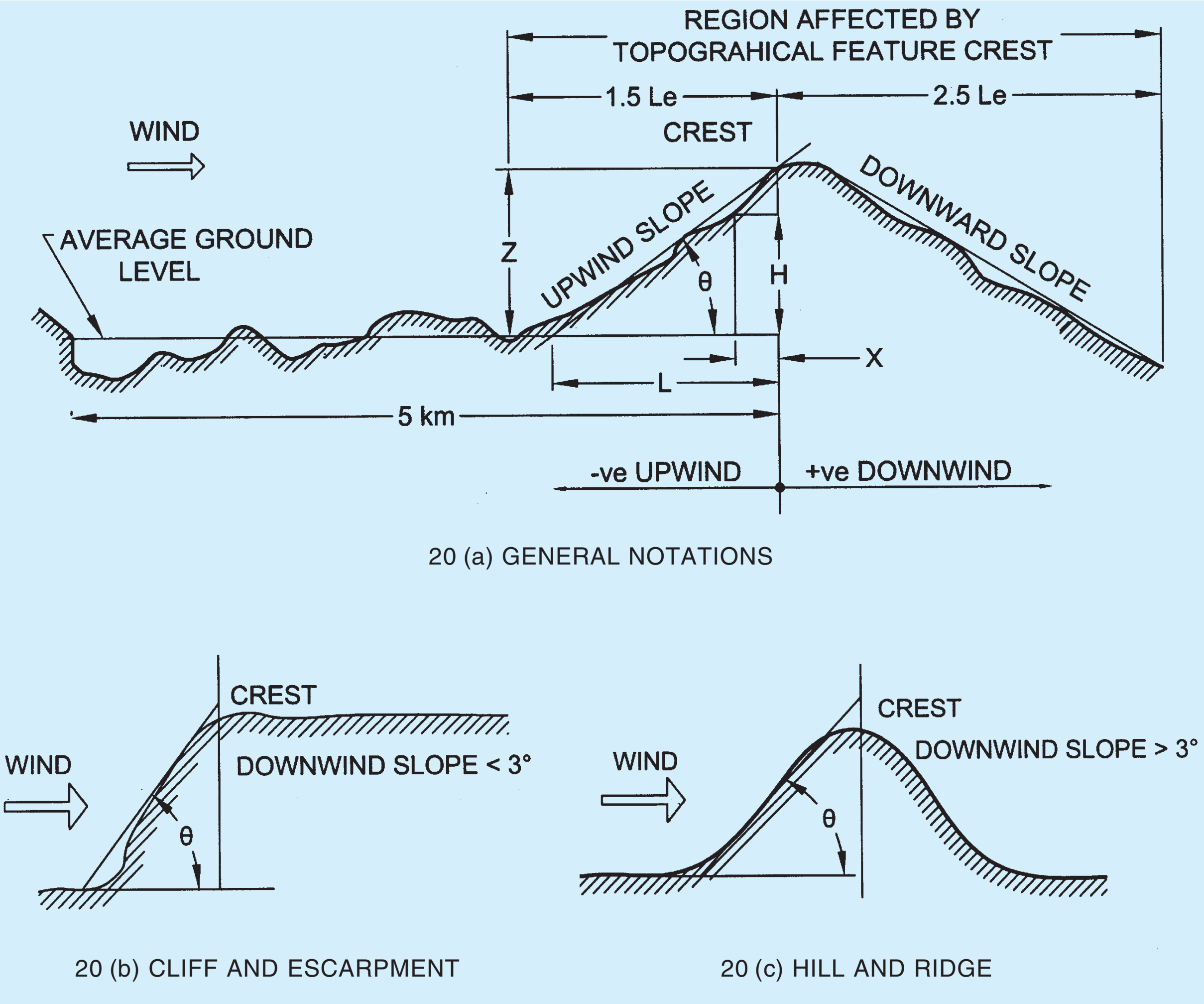


FIG. 20 DEFINITION OF TOPOGRAPHICAL DIMENSIONS

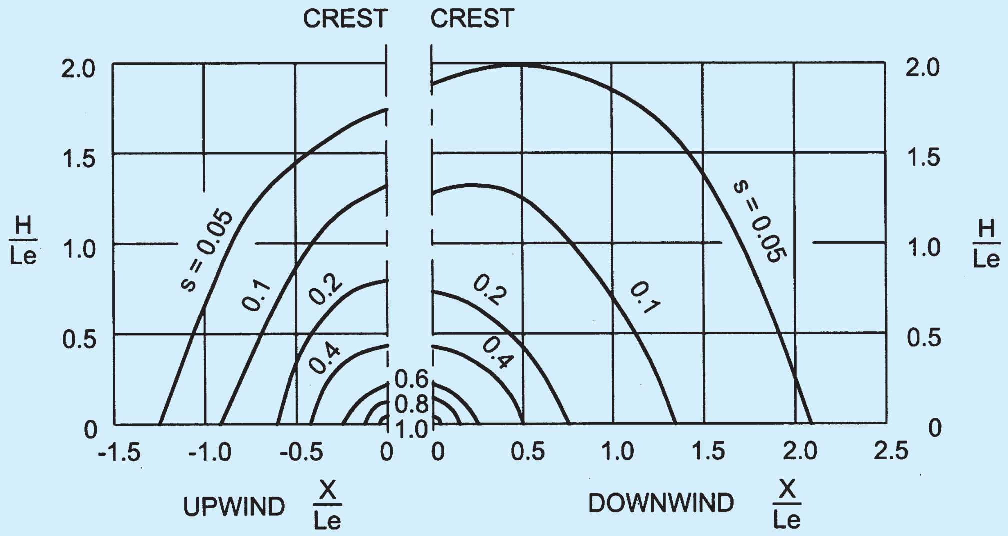


FIG. 21 FACTOR s FOR CLIFF AND ESCARPMENT

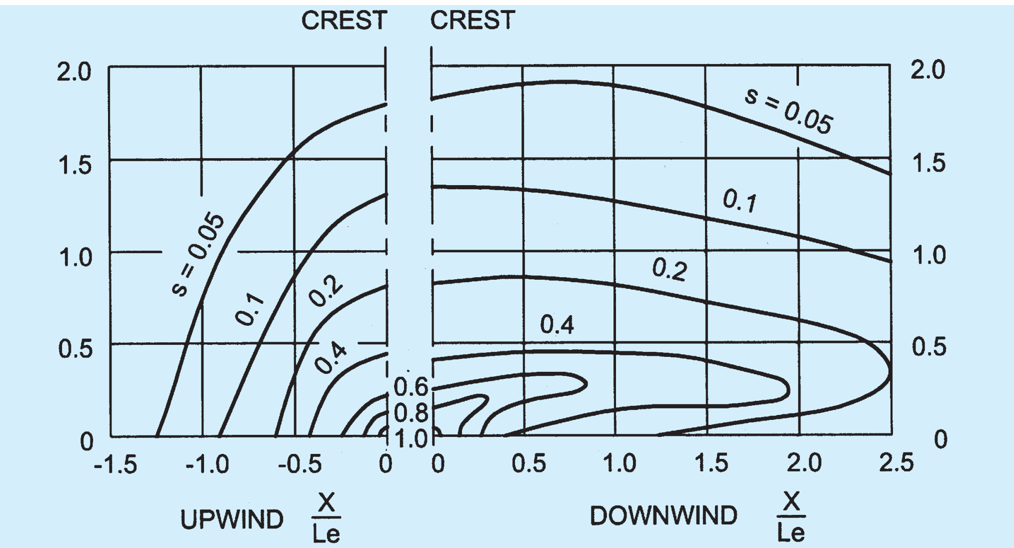


FIG. 22 FACTOR s FOR RIDGE AND HILL

ANNEX F

[*Clause* 4.5.3.2 (b)]

WIND FORCE ON CIRCULAR SECTIONS

F-1 The wind force on any object is given by:

$$F = C_f A_e \cdot p_d$$

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 upon 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 account of the velocity and viscosity of the flowing medium (in the case the wind) and the member diameter.

$$\text{Reynolds number, } R_e = \frac{DV_d}{\gamma}$$

where

D = Diameter of the member;

V_d = Design wind speed; and

γ = Kinematic viscosity of the air which is 1.46×10^5 m^2/s at 15°C 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 DV_d as the parameter instead of Reynolds numbers and this has been done in this Section.

The dependence of a circular section's force coefficient or 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. 23 and the force coefficient is typically 1.2. As the Reynolds number is increased, the wake gradually changes to that shown in Fig. 24, that is, the wake width d_w decreases and the separation point, S moves from the front to the back of the body.

As a result, the force coefficient shows a rapid drop at a critical value of Reynolds number, followed by a gradual rise and Reynolds number is increased still further.

The variation of C_f with parameter DV_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 $\varepsilon/D = 1 \times 10^{-5}$ in a steady air-stream, as found in a low-turbulence wind tunnel is shown for comparison.

It can be seen that the main effect of free-stream turbulence is to decrease the critical value of the parameter DV_d. For subcritical flows, turbulence can produce a considerable reduction in C_f below the steady air-stream values. For super-critical 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.

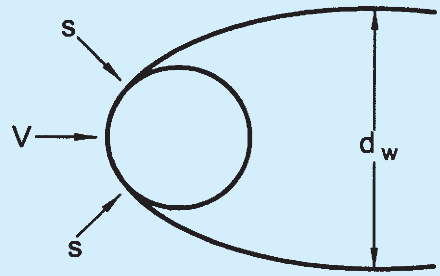


FIG. 23 WAKE IN SUBCRITICAL FLOW

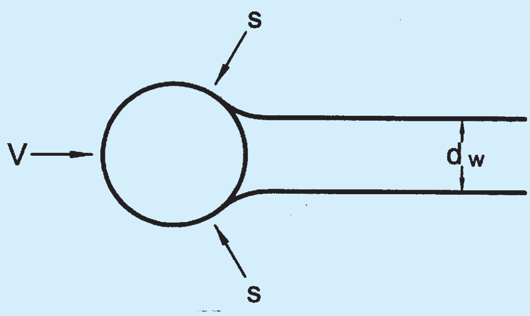


FIG. 24 WAKE IN SUPER-CRITICAL FLOW

ANNEX G

(*Clause* 5.0)

SYMBOLS

The symbols and notations given below apply to the provisions of this Code:

A_h Design horizontal seismic coefficient

A_k Design horizontal acceleration spectrum value for mode k of vibration

b_i ith Floor plan dimension of the building perpendicular to the direction of force

c Index for the closely-spaced

d Base dimension of the building, in metres, in the direction in which the seismic force is considered.

DL Response quantity due to dead load

e_{di} Static eccentricity to be used at floor i calculated as per **5.4.8.2**.

e_{si} Static eccentricity at floor i defined as the distance between centre of mass and centre of rigidity

EL_x Response quantity due to earthquake load for horizontal shaking along x-direction

EL_y Response quantity due to earthquake load for horizontal shaking along y-direction

EL_z Response quantity due to earthquake load for vertical shaking along z-direction

F_{roof} Design lateral forces at the roof due to all modes considered

F_i Design lateral forces at the floor i due to all modes considered

g Acceleration due to gravity

h Height of structure, in metres

h_i Height measured from the base of the building to floor i

I Importance factor

IL Response quantity due to imposed load

M_k Modal mass of mode k

n number of storeys

N SPT value for soil

P_k Modal participation factor of mode k

Q_i Lateral force at floor i

Q_{ik} Design lateral force at floor i in mode k

r Number of modes to be considered as per **5.4.8.4.2**

R Response reduction factor

S_k/g Average response acceleration coefficient for rock or soil sites as given by Fig. 2 and Table 3 based on appropriate natural periods and damping of the structure

T Undamped natural period of vibration of the structure (in second)

T_a Approximate fundamental period (in second)

T_i Fundamental natural period of vibration (in second)

T_k Undamped natural period of mode k of vibration (in second)

V_B Design seismic base shear

$\bar{V}_B$ Design base shear calculated using the approximate fundamental period T_a

V_i Peak storey shear force in storey i due to all modes considered

V_{ik} Shear force in storey i in mode k

V_{roof} Peak storey shear force at the roof due to all modes considered

W Seismic weight of the structure

W_i Seismic weight of floor i

Z Zone factor

ϕ_{ik} Mode shape coefficient at floor i in mode k

λ Peak response (for example, member forces, displacements, storey forces, storey shears or base reactions) due to all modes considered

λ_k Absolute value of maximum response in mode k

λ_c Absolute value of maximum response in mode c, where mode c is a closely-spaced mode

λ^* Peak response due to the closely-spaced modes only

P_{ij} Coefficient used in the Complete Quadratic Combination (CQC) method while combining responses of modes i and j

ω_i Circular frequency in rad/second in the ith mode

ANNEX H

(*Clause* 5.1.15)

COMPREHENSIVE INTENSITY SCALE (MSK 64)

The scale was discussed generally at the inter-governmental meeting convened by UNESCO in April 1964. Though not finally approved the scale is more comprehensive and describes the intensity of earthquake more precisely. The main definitions used are followings:

a) *Type of Structures (Buildings)*

Type A — Building in field-stone, rural structures, unburnt-brick houses, clay houses.

Type B — Ordinary brick buildings, buildings of large block and prefabricated type, half timbered structures, buildings in natural hewn stone.

Type C — Reinforced buildings, well built wooden structures.

b) *Definition of Quantity*:

Single, few	About 5 percent
Many	About 50 percent
Most	About 75 percent

c) *Classification of Damage to Buildings*

Grade 1	Slight damage	Fine cracks in plaster: fall of small pieces of plaster
Grade 2	Moderate damage	Small cracks in plaster: fall of fairly large pieces of plaster; pantiles slip off; cracks in chimneys parts of chimneys fall down
Grade 3	Heavy damage	Large and deep cracks in plaster: fall of chimneys
Grade 4	Destruction	Gaps in walls: parts of buildings may collapse; separate parts of the buildings lose their cohension; and inner walls collapse
Grade 5	Total damage	Total collapse of the buildings

d) *Intensity Scale*

1. *Not noticeable* — The intensity of the vibration is below the limits of sensibility; the tremor is detected and recorded by seismograph only.
2. *Scarcely noticeable (very slight)* — Vibration is felt only by individual people at rest in houses, especially on upper floors of buildings.
3. *Weak, partially observed only* — The earthquake is felt indoors by a few people, outdoors only in favourable circumstances. The vibration is like that due to the passing of a light truck. Attentive observers notice a slight swinging of hanging objects, somewhat more heavily on upper floors.
4. *Largely observed* — The earthquake is felt indoors by many people, outdoors by few. Here and there people awake, but no one is frightened. The vibration is like that due to the passing of a heavily loaded truck. Windows, doors, and dishes rattle. Floors and walls crack. Furniture begins to shake. Hanging objects swing slightly. Liquid in open vessels are slightly disturbed. In standing motor cars the shock is noticeable.
5. *Awakening*
   i) The earthquake is felt indoors by all outdoors by many. Many people awake. A few run outdoors uneasy. Building tremble throughout. Hanging objects swing considerably. Pictures knock against walls or swing out of place. Occasionally pendulum clocks stop. Unstable objects overturn or shift. Open doors and windows are thrust open and slam back again. Liquids spill in small amounts from well-filled open containers. The sensation of vibration is like that due to heavy objects falling inside the buildings.
   ii) Slight damages in buildings of Type A are possible.
   iii) Sometimes changes in flow of springs.
6. *Frightening*
   i) Felt by most indoors and outdoors. Many people in buildings are frightened and run outdoors. A few persons loose their balance. Domestic animals run out of their stalls in few instances, dishes and glassware may break and books fall down. Heavy furniture may possibly move and small steeple bells may ring.
   ii) Damage of Grade 1 is sustained in single buildings of Type B and in many of Type A. Damage in few buildings of Type A is of Grade 2.
   iii) In few cases, cracks up to widths of 1 cm possible in wet ground; in mountains occasional landslips; change in flow of springs and in level of well water are observed.
7. *Damage of buildings*
   i) Most people are frightened and run outdoors. Many find it difficult to stand. The vibration is noticed by persons driving motor cars. Large bells ring.
   ii) In many buildings of Type C damage of Grade 1 is caused; in many buildings of Type B damage is of Grade 2. Most buildings of Type A suffer damage of Grade 3, few of

Grade 4. In single instances, landslides of roadway on steep slopes; crack in roads; seams of pipelines damaged; cracks in stone walls.

iii) Waves are formed on water, and is made turbid by mud stirred up. Water levels in wells change, and the flow of springs changes. Some times dry springs have their flow resorted and existing springs stop flowing. In isolated instances parts of sand and gravelly banks slip off.

8. *Destruction of buildings*

i) Fright and panic; also persons driving motor cars are disturbed. Here and there branches of trees break off. Even heavy furniture moves and partly overturns. Hanging lamps are damaged in part.

ii) Most buildings of Type C suffer damage of Grade 2, and few of Grade 3. Most buildings of Type B suffer damage of Grade 3. Most buildings of Type A suffer damage of Grade 4. Occasional breaking of pipe seams. Memorials and monuments move and twist. Tombstones overturn. Stone walls collapse.

iii) Small landslips in hollows and on banked roads on steep slopes; cracks in ground up to widths of several centimeters. Water in lakes become turbid. New reservoirs come into existence. Dry wells refill and existing wells become dry. In many cases, change in flow and level of water is observed.

9. *General damage of buildings*

i) General panic; considerable damage to furniture. Animals run to and fro in confusion, and cry.

ii) Many buildings of Type C suffer damage of Grade 3, and few of Grade 4. Most buildings of Type B show a damage of Grade 4 and a few of Grade 5. Many buildings of Type A suffer damage of Grade 5. Monuments and columns fall. Considerable damage to reservoirs; underground pipes partly broken. In individual cases, railway lines are bent and roadway damaged.

iii) On flat land overflow of water, sand and mud is often observed. Ground cracks in widths of up to 10 cm on slopes and river banks more than 10 cm. Further more a large number of slight cracks in ground; falls of rock, many land slides and earth flows; large waves in wate. Dry wells renew their flow and existing wells dry up.

10. *General destruction of buildings*

i) Many buildings of Type C suffer damage of Grade 4, and few of Grade 5. Many buildings of Type B show a damage of Grade 5. Most of Type A have destruction of Grade 5. Critical damage of dykes and dams. Severe damage to bridges. Railways lines are bent slightly. Underground pipes are bent on broken. Road paving and asphalt show waves.

ii) In ground, cracks up to widths of several centimetres, sometimes up to 1 m. Parallel to water courses occur broad fissures. Loose ground slides from steep slopes. From river banks and steep coasts considerable landslides are possible. In coastal areas displacement of sand and mud; change of water level in wells; water from canals; lakes; rivers; etc, thrown on land. New lakes occur.

11. *Destruction*

i) Severe damage even to well built buildings, bridges, water damps and railway lines. Highways become useless. Underground pipes destroyed.

ii) Ground considerably distorted by broad cracks and fissures, as well as movement in horizontal and vertical directions. Numerous landslips and falls of rocks. The intensity of the earthquake requires to be investigated specifically.

12. *Landscape changes*

i) Practically all structures above and below ground are greatly damaged or destroyed.

ii) The surface of the ground is radically changed. Considerable ground cracks with extensive vertical and horizontal movements are observed. Falling of rock and slumping of river banks over wide areas, lakes are damaged, waterfalls appears and rivers are deflected. The intensity of the earthquake requires to be investigated specially.

ANNEX J

(*Clause* 5.3.4.2)

ZONE FACTORS FOR SOME IMPORTANT TOWNS

Town	*Zone*	*Zone Factor, Z*	*Town*	*Zone*	*Zone Factor, Z*
Agra	III	0.16	Goa	III	0.16
Ahmedabad	III	0.16	Gulbarga	II	0.10
Ajmer	II	0.10	Gaya	III	0.16
Allahabad	II	0.10	Gorakhpur	IV	0.24
Almora	IV	0.24	Hyderabad	II	0.10
Ambala	IV	0.24	Imphal	V	0.36
Amritsar	IV	0.24	Jabalpur	III	0.16
Asansol	III	0.16	Jaipur	II	0.10
Aurangabad	II	0.10	Jamshedpur	II	0.10
Bahraich	IV	0.24	Jhansi	II	0.10
Bangalore	II	0.10	Jodhpur	II	0.10
Barauni	IV	0.24	Jorhat	V	0.36
Bareilly	III	0.16	Kakrapara	III	0.16
Belgaum	III	0.16	Kalapakkam	III	0.16
Bhatinda	III	0.16	Kanchipuram	III	0.16
Bhilai	II	0.10	Kanpur	III	0.16
Bhopal	II	0.10	Karwar	III	0.16
Bhubaneshwar	III	0.16	Kohima	V	0.36
Bhuj	V	0.36	Kolkata	III	0.16
Bijapur	III	0.16	Kota	II	0.10
Bikaner	III	0.16	Kurnool	II	0.10
Bokaro	III	0.16	Lucknow	III	0.16
Bulandshahr	IV	0.24	Ludhiana	IV	0.24
Burdwan	III	0.16	Madurai	II	0.10
Calicut	III	0.16	Mandi	V	0.36
Chandigarh	IV	0.24	Mangalore	II	0.16
Chennai	III	0.16	Monghyr	IV	0.24
Chitradurga	II	0.10	Moradabad	IV	0.24
Coimbatore	III	0.16	Mumbai	III	0.16
Cuddalore	III	0.16	Mysore	II	0.10
Cuttack	III	0.16	Nagpur	II	0.10
Darbhanga	V	0.36	Nagarjunasagar	II	0.10
Darjeeling	IV	0.24	Nainital	IV	0.24
Dharwad	III	0.16	Nasik	III	0.16
Dehra Dun	IV	0.24	Nellore	III	0.16
Dharampuri	III	0.16	Osmanabad	III	0.16
Delhi	IV	0.24	Panjim	III	0.16
Durgapur	III	0.16	Patiala	III	0.16
Gangtok	IV	0.24	Patna	IV	0.24
Guwahati	V	0.36	Pilibhit	IV	0.24

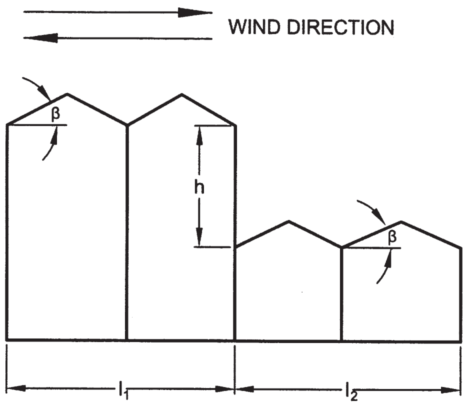
Town	Zone	Zone Factor, Z	Town	Zone	Zone Factor, Z
Pondicherry	II	0.10	Tarapur	III	0.16
Pune	III	0.16	Tezpur	V	0.36
Raipur	II	0.10	Thane	III	0.16
Rajkot	III	0.16	Thanjavur	II	0.10
Ranchi	II	0.10	Thiruvananthapuram	III	0.16
Roorkee	IV	0.24	Tiruchchirappalli	II	0.10
Rourkela	II	0.10	Tiruvennamalai	III	0.16
Sadiya	V	0.36	Udaipur	II	0.10
Salem	III	0.16	Vadodara	III	0.16
Shimla	IV	0.24	Varanasi	III	0.16
Sironj	II	0.10	Vellore	III	0.16
Solapur	III	0.16	Vijayawada	III	0.16
Srinagar	V	0.36	Vishakhapatnam	II	0.10
Surat	III	0.16			

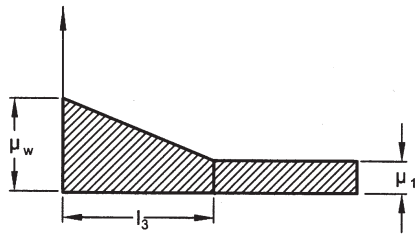
ANNEX K

(*Clauses* 6.4.2.4 *and* 6.4.3)

SHAPE COEFFICIENTS FOR MULTILEVEL ROOFS

A more comprehensive formula for the shape coefficient for multilevel roofs





$$\mu_w = 1 + \frac{1}{h}(m_1 l_1 + m_2 l_2)(l_2 - 2h)$$

$\mu_1 = 0.8$

$l_3 = 2h$

(*h* and *l* being in metres)

Restriction:

$$\mu_w \leq \frac{kh}{S_o}$$

where

S_o is in kilopascals (kilonewtons per square metre)

k is in newtons per cubic metre

$l_3 \leq 15$ m

Values of m_1 (m_2) for the higher (lower) roof 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 coefficients 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 L

(*Clause* 7.7)

VIBRATIONS IN BUILDINGS

L-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.

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.

L-2 SERVICEABILITY LIMIT STATE VERIFICATION OF STRUCTURE SUSCEPTIBLE TO VIBRATIONS

L-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.

L-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.

L-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 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. Where the structural response may significantly influence the force-time histories to be applied, these interactions have to be considered either in modelling 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.

L-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:

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 M

(*Clause* 9.2)

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, Vishakhapatnam, Vizianagram	East Godavari, Guntur, Prakasam, West Godavari	—
Assam	All 22 districts listed in Table 38 could have M.S.K. IX or more with flooding	No cyclone, but speed can be 50 m/s in districts of Table 38 causing local damage except Dhubri	—	—
Bihar	All 25 districts listed in Table 38	—	—	—
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 38	—	—	—

(1)	(2)	(3)	(4)	(5)
Kerala	Idduki, Kottayam, Palakkad, Pathanamthitta	—	Alappuzha, Ernakulum, Kannur, Kasargode, Kollam, Kozhikode, Malappuram, Thiruvanathapuram, Trissur	—
Maharashtra	—	—	—	Bombay, Rayagad, Ratnagiri, Sindhudurg, Thane
Orissa	—	Ganjam	Baleshwar, Cuttack, Puri	Dhenkanal
Punjab	All 12 districts listed in Table 38	—	—	—
Uttar Pradesh	All 50 districts listed in Table 38	—	—	—
West Bengal	Birbhum, Darjeeling, Jalpaiguri, Kooch Bihar, Malda, Murshidabad, West Dinajpur	—	Bardhaman, Calcutta, Hugli, Howra, Mednipur, Nadia, North and South 24 Parganas	Bankura
Union Territories	Delhi	—	Yanam (Py)	Diu
India	139 Districts	6 Districts	29 Districts	16 Districts

Table 38 Multi-Hazard Prone Districts

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

[3]**Haryana**

Ambala, Bhiwani, Faridabad, Gurgaon, Hissar, Jind, Kurukshetra, Rohtak

[4]**Punjab**

Amritsar, Bathinda, Faridkot, Firozpur, Gurdaspur, Hoshiarpur, Jalandhar, Kapurthala, Ludhiana, Patiala, Rup Nagar, Sangrur

[5]**Uttar Pradesh**

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

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 by the Authority as a guide in 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.

	IS No.	*Title*
(1)	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*)
(2)	8888 (Part 1) : 1993	Guide for requirements of low income housing: Part 1 Urban area (*first revision*)
(3)	807 : 1976	Code of practice for design, manufacture, erection and testing (structural portion) of cranes and hoists (*first revision*)
	3177 : 1999	Code of practice for electric overhead travelling cranes and gantry cranes other than steelwork cranes (*second revision*)
(4)	13920 : 1993	Code of practice for ductile detailing of reinforced concrete structures subjected to seismic forces
(5)	1893 (Part 4) : 2005	Criteria for earthquake resistant design of structures: Part 4 Industrial structures including stack-like structures
(6)	1888 : 1982	Method of load test on soils (*second revision*)
	6403 : 1981	Code of practice for determination of bearing capacity of shallow foundations (*first revision*)
(7)	1498 : 1970	Classification and identification of soils for general engineering purposes (*first revision*)
(8)	2131 : 1981	Method for standard penetration test for soils (*first revision*)
(9)	4326 : 1993	Code of practice for earthquake resistant design and construction of buildings (*second revision*)
(10)	3414 : 1968	Code of practice for design and installation of joints in buildings
(11)	1642 : 1989	Code of practice for fire safety of buildings (general): Details of construction (*first revision*)
(12)	15498 : 2004	Guidelines for improving cyclone resistance of low rise houses and other buildings/ structures
(13)	14458 (Part 1) : 1998	Guidelines for retaining wall for hill area: Part 1 Selection of type of wall
(14)	14458 (Part 2) : 1997	Guidelines for retaining wall for hill area: Part 2 Design of retaining/breast walls
(15)	14458 (Part 3) : 1998	Guidelines for retaining wall for hill area: Part 3 Construction of dry stone
(16)	14496 (Part 2) : 1998	Guidelines for preparation of landslide-hazard zonation maps in mountainous terrains: Part 2 Macro-zonation
(17)	14680 : 1999	Guidelines for landslide control

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 2 Soils and Foundations

BUREAU OF INDIAN STANDARDS

CONTENTS

National Building Code Sectional Committee, CED 46

FOREWORD

This Section 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. In the first revision design considerations in respect of shallow foundation were modified, provisions regarding pier foundation were added and provisions regarding draft foundation and pile foundation were revised and elaborated.

As a result of experience gained in implementation of 1983 version of the Code and feed back received as well as revision of standards and preparation of new standards in the field of soils and foundations, a need to revise this Section was felt. This revision has therefore been prepared to take into account these developments. The significant changes incorporated in this revision include:

a) Design considerations in respect of shallow foundations have been modified.
b) Method for determining depth of fixity, lateral deflection and maximum moment have been modified.
c) Reference has been made to ground improvement techniques.
d) References to Indian Standards made in the text have been updated.

For detailed information regarding structural analysis and soil mechanics aspects of individual foundations, reference should be made to standard textbooks and available literature.

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

IS No.	*Title*
1080 : 1985	Code of practice for design and construction of shallow foundations in soils (other than raft, ring and shell) (*second revision*)
1904 : 1986	Code of practice for design and construction of foundations in soils: General requirements (*third revision*)
2911 (Part 1/Sec 1) : 1979	Code of practice for design and construction of pile foundations: Part 1 Concrete piles, Section 1 Driven cast *in-situ* concrete piles (*first revision*)
2911 (Part 1/Sec 2) : 1979	Code of practice for design and construction of pile foundations: Part 1 Concrete piles, Section 2 Bored cast *in-situ* piles (*first revision*)
2911 (Part 1/Sec 3) : 1979	Code of practice for design and construction of pile foundations: Part 1 Concrete piles, Section 3 Driven precast concrete piles (*first revision*)
2911 (Part 1/Sec 4) : 1984	Code of practice for design and construction of pile foundations: Part 1 Concrete piles, Section 4 Bored precast concrete piles
2911 (Part 3) : 1980	Code of practice for design and construction of pile foundations: Part 3 Under-reamed piles (*first revision*)
2950 (Part 1) : 1981	Code of practice for design and construction of raft foundations: Part 1 Design (*second revision*)
9456 : 1980	Code of practice for design and construction of conical hyperbolic paraboidal types of shell foundations

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.

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 2 Soils and Foundations

1 SCOPE

This Section covers structural design (principles) of all building foundations such as raft, pile 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.

2 TERMINOLOGY

2.0 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* — Cohesionless aggregates of angular rounded or semi-rounded, fragments of more or less unaltered rocks or minerals, 50 percent or more of the particles having size greater than 4.75 mm and less than 80 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* — Cohesionless aggregate of rounded, sub-rounded, angular, sub-angular or flat fragments of more or less unaltered rock or minerals, 50 percent or more of particles greater than 0.075 mm or less than 4.75 mm in size.

2.1.9 *Sand, Coarse* — Sand which contains 50 percent or more of particles of size greater than 2 mm and less than 4.75 mm.

2.1.10 *Sand, Fine* — Sand which contains 50 percent of particles of size greater than 0.075 mm and less than 0.425 mm.

2.1.11 *Sand, Medium* — Sand which contains 50 percent of particles of size greater than 0.425 mm and less than 2.0 mm.

2.1.12 *Silt* — A fine grained soil with little or no plasticity. 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.

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.

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 *Total Settlement* — The total downward movement of the foundation unit under load.

2.2 Shallow Foundation

2.2.1 *Back Fill* — Materials used or re-used to fill an excavation.

2.2.2 *Bearing Capacity, Safe* — The maximum intensity of loading that the soil will safely carry with a factor of safety without risk of shear failure of soil irrespective of any settlement that may occur.

2.2.3 *Bearing Capacity, Ultimate* — The intensity of loading at the base of a foundation which would cause shear failure of the supporting soil.

2.2.4 *Bearing Pressure, Allowable (Gross or Net)* — The maximum allowable loading intensity on the ground in any given case (with full cognizance of surcharge) taking into account the maximum safe bearing capacity, the amount and kind of settlement expected and the capability of the structure to take up this settlement. It is, therefore, a combined function of both the site conditions and characteristics of the particular structure.

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.

2.2.5 *Factor of Safety (with Respect to Bearing Capacity)* — A factor by which the ultimate bearing capacity (net) must be reduced to arrive at the value of safe bearing capacity (net).

2.2.6 *Footing* — A spread 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.

2.2.7 *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.

2.2.8 *Make-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.

2.2.9 *Offset* — The projection of the lower step from the vertical face of the upper step.

2.2.10 *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.

2.2.11 *Shallow Foundation* — A foundation whose width is generally equal to or greater than its depth.

NOTE — These cover such types of foundations in which load transference is primarily through shear resistance of the bearing strata (the frictional resistance of soil above bearing strata is not taken into consideration) and are laid normally to depth of 3 m.

2.2.12 *Spread Foundation* — A foundation which transmits the load to the ground through one or more footings.

2.3 Pile Foundation

2.3.1 *Batter Pile (Raker Pile)* — The pile which is installed at an angle to the vertical.

2.3.2 *Bearing Pile* — A pile formed in the ground for transmitting the load of a structure to the soil by the resistance developed at its tip and/or along its surface. It may be formed either vertically or at an inclination (Batter Pile) and may be required to take uplift pressure.

If the pile supports the load primarily by resistance developed at the pile point or base, it is referred to as 'End Bearing Pile', if support is provided primarily by friction along its surface, it is referred to as 'Friction Pile'.

2.3.3 *Bored Cast* in-situ *Pile* — The pile formed within the ground by excavating or boring a hole within it, with or without the use of a temporary casing and subsequently filling it with plain or reinforced concrete. When the liner is left permanently it is termed as cased pile and when the casing is taken out it is termed as uncased pile.

In installing a bored pile the sides of the borehole (when it does not stand by itself) are required to be stabilized with the aid of a temporary casing, or with the aid of drilling mud of suitable consistency. For marine situations such piles are formed with permanent casing (liner).

2.3.4 *Bored Compaction Pile* — A bored cast *in-situ* pile with or without bulb(s) in which the compaction of the surrounding ground and freshly filled concrete in pile bore is simultaneously achieved by a suitable method. If the pile is with bulb(s), it is known as under-reamed bored compaction pile.

2.3.5 *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.

2.3.6 *Bored Precast Pile* — A pile constructed in reinforced concrete in a casting yard and subsequently lowered in the pre-bored holes and the space around grouted.

2.3.7 *Cut-off Level* — It is the level where the installed pile is cut-off to connect the pile caps or beams or any other structural components at that level.

2.3.8 *Driven Cast* in-situ *Pile* — A pile formed within the ground by driving a casing of permanent or temporary type and subsequently filling in the hole so formed with plain or reinforced concrete. For displacing the subsoil, the casing is installed with a plug or a shoe at the bottom end. When the casing is left permanently, it is termed as cased pile and when the casing is taken out, it termed as uncased pile.

2.3.9 *Driven Precast Pile* — A pile constructed in concrete (reinforced or prestressed) in a casting yard and subsequently driven in the ground when it has attained sufficient strength.

2.3.10 *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.

2.3.11 *Factor of Safety* — It is the ratio of the ultimate load capacity of a pile to the safe load of a pile.

2.3.12 *Multi-Under-Reamed Pile* — An under-reamed pile having more than one bulb. The piles having two bulbs may be called double under-reamed piles.

2.3.13 *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.

2.3.14 *Ultimate Load Capacity* — The maximum load which a pile can carry before failure of ground when the soils fails by shear or failure of pile materials.

2.3.15 *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.

3 SITE INVESTIGATION

3.1 General

In areas which have already been developed, information should be obtained regarding the existing local knowledge, records of trial pits, bore holes, 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 scope of further investigation for other structures.

3.1.1 If the existing information is not sufficient or is inconclusive, the site should be explored in detail as per good practice [6-2(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. Geological maps of the area give valuable information of the site conditions. The general topography will often give some indications of the soil conditions and their variations. In certain cases the earlier uses of the site may have a very important bearing on the proposed new structures.

3.2 Methods of Site Exploration

3.2.1 The common methods of site exploration are given below:

a) *Open trial pits* — The method consists of excavating trial pits and thereby exposing the subsoil surface thoroughly, enabling undisturbed samples to be taken from the sides and bottom of the trial pits. This is suitable for all types of formations, but should be used for small depths (up to 3 m). In the case of cuts which cannot stand below water table, proper bracing should be given.

b) *Auger boring* — The auger is either power of hand operated with periodic removal of the cuttings.

c) *Shell and auger boring* — Both manual and mechanized rig can be used for vertical borings. The tool normally consists of augers for soft to stiff clays, shells for very stiff and hard clays, and shells or sand pumps for sandy strata attached to sectional boring rods.

d) *Wash boring* — In wash boring, the soil is loosened and removed from the bore hole by a stream of water or drilling mud is worked up and down or rotated in the bore hole. The water or mud flow carries the soil up the annular space between the wash pipe and the casing, and it overflows at ground level, where the soil in suspension is allowed to settle in a pond or tank and the fluid is re-circulated as required. Samples of the settled out soil can be retained for identification purposes but this procedure is often unreliable. However, accurate identification can be obtained if frequent 'dry' sampling is resorted to using undisturbed sample tubes.

e) *Sounding/Probing including standard penetration test, dynamic and static cone penetration test*

f) *Geophysical method*

g) *Percussion boring and rotary boring*

h) *Pressure meter test*

3.2.2 *Number and Disposition of Test Locations*

The number and disposition of various tests shall depend upon type of structure/buildings and the soil strata variations in the area. General guidelines are, however, given below:

a) For a compact building site covering an area of about 0.4 hectare, one bore hole or trial pit in each corner and one in the centre should be adequate.

b) For smaller and less important buildings, even one bore hole or trial pit in the centre will suffice.

c) For very large areas covering industrial and residential colonies, the geological nature of the terrain will help in deciding the number of bore holes or trial pits. For plant and other main structures, number of bore holes and/or trial pits should be decided considering importance of structure and type as well as uniformity of strata. In general, dynamic or static cone penetration tests may be performed at every 100 m by dividing the area in a grid pattern and the number of bore holes or trial pits may be decided by examining the variation in the penetration curves. The cone penetration tests may not be possible at sites having generally bouldery strata. In such cases, geophysical methods should be resorted to.

3.2.3 *Depth of Exploration*

The depth of exploration required depends on the type of proposed structure, its total weight, the size, shape and disposition of the loaded areas, soil profile, and the physical properties of the soil that constitutes each individual stratum. Normally, it should be one and a half times the width of the footing below foundation level. In certain cases, it may be necessary to take at least one bore hole or cone test or both to twice the width of the foundation. If a number of loaded areas are in close proximity the effect of each is additive. In such cases, the whole of the area may be considered as loaded and exploration should be carried out up to one and a half times the lower dimension. In weak soils, the exploration should be continued to a depth at which the loads can be carried by the stratum in question without undesirable settlement and shear failure. In any case, the depth to which seasonal variations affect the soil should be regarded as the minimum depth for the exploration of sites. But where industrial processes affect the soil characteristics this depth may be more. The presence of fast growing and water seeking trees also contributes to the weathering processes.

NOTE — Examples of fast growing and water seeking trees are Banyan (*Ficus bengalensis*), Pipal (*Ficus religiosa*) and Neem (*Azadirachta indica*).

3.2.3.1 An estimate of the variation with depth of the vertical normal stress in the soil arising from foundation loads may be made on the basis of elastic theory. The net loading intensity at any level below a foundation may be obtained approximately by assuming a spread of load of two vertical to one horizontal from all sides of the foundations, due allowance being made for the overlapping effects of load from closely spaced footings. As a general guidance, the depth of exploration at the start of the work may be decided as given in Table 1, which may be modified as exploration proceeds, if required. However, for plant and other main structures, the depth of exploration may be decided depending upon importance of structure, loading conditions and type as well as uniformity of strata.

3.3 Choice of Method for Site Exploration

The choice of the method depends on the following factors.

3.3.1 *Nature of Ground*

a) *Soils* — In clayey soils, borings are suitable for deep exploration and pits for shallow exploration. In case of soft sensitive clayey soils field vane shear test may be carried out with advantage.

   In sandy soils, special equipments may be required for taking representative samples below the water table. Standard penetration test, dynamic cone penetration test and static cone penetration test are used to assess engineering properties.

b) *Gravel-boulder deposits* — In the deposits where gravel-boulder proportion is large (>30 percent), the sub-soil strata should be explored by open trial pits of about 5 m × 5 m but in no case less than 2 m × 2 m. The depth of excavation may be up to 6 m. For determining strata characteristics, *in-situ* tests should be preferred. For shear characteristics and allowable soil pressure dynamic cone penetration tests, load tests on cast *in-situ* footing and *in-situ* shear tests that is, boulder-boulder test or concrete-boulder test are more appropriate. For detailed information on these tests reference may be made to good practice [6-2(2)]. Depending on the structure, if required, the strata may be explored by drilling bore hole using suitable method.

c) *Rocks* — Drillings are suitable in hard rocks and pits in soft rocks. Core borings are suitable for the identification of types of rock, but they cannot supply data on joints and fissures which can be examined only in pits and large diameter borings.

3.3.2 *Topography*

In hilly country, the choice between vertical openings (for example, borings and trial pits) and horizontal openings (for example, headings) may depend on the geological structure, since steeply inclined strata are most effectively explored by headings and horizontal strata by trial pits or borings. Swamps and areas overlain by water are best explored by borings which may require use of a floating craft.

3.3.3 *Cost*

For deep exploration, borings are usual, as deep shafts are costly. For shallow exploration in soil, the choice between pits and borings will depend on the nature of the ground and the information required for shallow exploration in rock; the cost of bringing a core drill to the site will be justified only if several holes are required; otherwise, trial pits will be more economical.

Table 1 Depth of Exploration

(*Clause* 3.2.3.1)

Sl No.	Type of Foundation	Depth of Exploration *D*
(1)	(2)	(3)
i)	Isolated spread footing or raft	One and a half times the width (B) (*see* Fig. 1)
ii)	Adjacent footings with clear spacing less than twice the width	One and a half times the length (L) of the footing (*see* Fig. 1)
iii)	Adjacent rows of footings	*See* Fig. 1
iv)	Pile and well foundations	To a depth of one and a half times the width of structure from the bearing (toe of pile or bottom of well)
v)	a) Road cuts	Equal to the bottom width of the cut
	b) Fill	Two metres below ground level or equal to the height of the fill whichever is greater

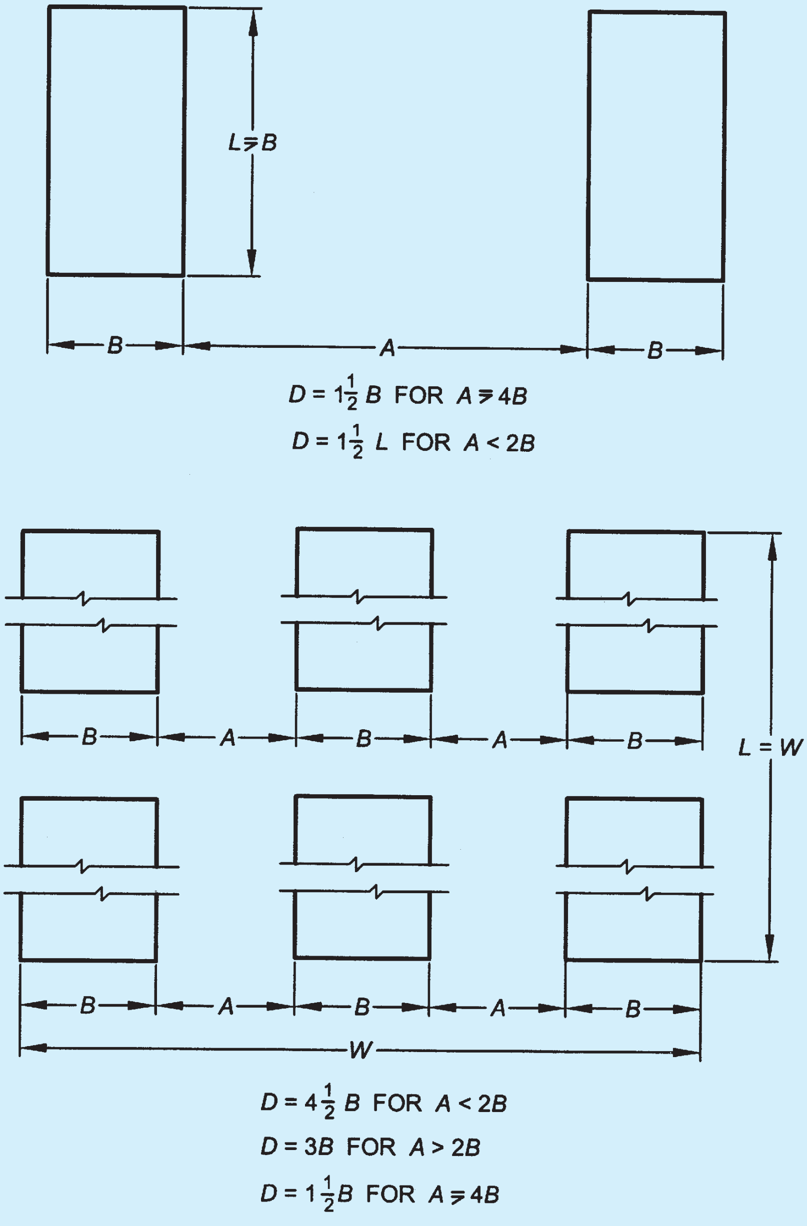


FIG. 1 DEPTH OF EXPLORATION

3.4 Sampling

3.4.1 *Methods of Sampling*

a) *Disturbed samples* — These are taken by methods which modify or destroy the natural structure of the material though with suitable precautions the natural moisture content can be preserved.

b) *Undisturbed samples* — These are taken by methods which preserve the structure and properties of the material. Such samples are easily obtained from most rocks, but undisturbed samples of soil can be obtained only by special methods. Thin walled tube samples may be used for undisturbed samples in soils of medium strength and tests for the same may be carried out in accordance with good practice [6-2(1)].

NOTE — In case of loose sandy soils and soft soils, specially below water table it may not be possible to take undisturbed sample, in which case other suitable methods may be adopted for exploration.

c) *Representative samples* — These samples have all their constituent parts preserved, but may or may not be structurally disturbed.

3.4.1.1 The methods usually employed are:

Nature of Ground	*Type of Sample*	*Method of Sampling*
(1)	(2)	(3)
Soil	Disturbed	Chunk samples Auger samples (for example, in clay) Shell samples (for example, in sand)
	Undisturbed	Chunk samples Tube samples
Rock	Disturbed	Wash samples from percussion of rotary drilling
	Undisturbed	Core barrel sampling

3.4.2 *Soil Samples*

a) *Disturbed soil samples* — The mass of sample generally required for testing purposes is given in Table 2.

b) *Undisturbed soil samples* — The minimum diameter of the sample shall be 38 mm with the minimum length/diameter ratio of 2.

3.4.3 *Rock Sample*

a) *Disturbed samples* — The sludge from percussion borings, or from rotary borings which have failed to yield a core, may be taken as a disturbed sample.

b) *Undisturbed samples*

   1) *Block samples* — Such samples taken from the rock formation shall be dressed to a size convenient for packing to about 90 mm × 75 mm × 50 mm.

   2) *Core sample; see also* good practice [6-2(3)]

3.4.4 *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.

3.4.5 *Examination and Testing of Samples*

3.4.5.1 The following tests shall be carried out in accordance with good practice [6-2(4)].

a) Particle size distribution,
b) Density,
c) Natural moisture content,
d) Consistency limits,
e) Consolidation characteristics,
f) Strength characteristics,
g) Sulphate, chloride and *p*H content of soil and ground water, and

Table 2 Mass of Soil Sample Required

[*Clause* 3.4.2(a)]

Sl No.	**Purpose of Sample**	**Type**	**Mass of Sample Required** kg
(1)	(2)	(3)	(4)
i)	Soil identification, natural moisture content tests, mechanical analysis, and index properties	Cohesive soil	1
	Chemical tests	Sands and gravels	3
ii)	Compaction tests	Cohesive soils and sands	12.5
		Gravely soils	25
iii)	Comprehensive examination of construction materials including stabilization	Cohesive soils and sands	25 to 50
		Gravely soils	50 to 100

h) Differential free swelling and swelling pressure.

4 CLASSIFICATION AND IDENTIFICATION OF SOILS

The classification and identification of soils for engineering purposes shall be in accordance with good practice [6-2(5)].

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 5 'Building Materials'.

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*

In the case of concrete placed against a soil containing harmful chemicals (sulphates, chlorides), among other protective measures, it shall be ensured to provide nominal cover required as prescribed in Part 6 'Structural Design, Section 5 Concrete for the Applicable Environment Exposure Condition'.

5.2.1.1 Preferably concrete of higher grade shall be used in situations subject to aggressive environment.

5.2.2 *Timber*

Where timber is exposed to soil, it shall be treated in accordance with good practice [6-2(6)].

6 TYPE OF FOUNDATIONS

6.1 Types of foundations covered in this Section are:

a) *Shallow Foundations*
   1) Pad or spread and strip foundations,
   2) Raft foundations, and
   3) Ring and shell foundations.

b) *Pile Foundations*
   1) Driven cast *in-situ* concrete piles,
   2) Bored cast *in-situ* concrete piles,
   3) Driven precast concrete piles,
   4) Bored precast concrete piles,
   5) Under-reamed concrete piles, and
   6) Timber piles.

c) *Other Foundations*
   Pier foundations.

7 SHALLOW FOUNDATIONS

7.0 Design Information

For the satisfactory design of foundations, the following information is necessary:

a) The type and condition of the soil or rock to which the foundation transfers the loads;
b) The 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) The allowable bearing pressure of the soils;
d) The changes in ground water level, drainage and flooding conditions and also the chemical conditions of the subsoil water, particularly with respect to its sulphate content;
e) The behaviour of the buildings, topography and environment/surroundings adjacent to the site, the type and depths of foundations and the bearing pressure assumed; and
f) Seismic zone of the region.

7.1 Design Considerations

7.1.1 *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 6 'Structural Design, Section 1 Loads, Forces and Effects'.

NOTES

1 For load, imposed, wind, seismic and other loads, *see* Part 6 'Structural Design, Section 1 Loads, Forces and Effects'.

2 For coarse grained soils, settlements shall be estimated corresponding to **7.1.1** (b) and for fine grained soils settlement shall be estimated corresponding to permanent loads only.

7.1.2 *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, or
b) The allowable bearing pressure that the soil can take without exceeding the permissible settlement (*see* **7.1.3**).

7.1.2.1 *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 allowable bearing pressure of shallow foundations based on shear and allowable settlement criteria shall be in accordance with good practice [6-2(7)].

7.1.2.2 *Field method for determining allowable bearing pressure*

Where appropriate, plate load tests can be performed and allowable pressure determined as per good practice [6-2(8)]. 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.

7.1.2.3 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.

7.1.2.4 *Effect of wind and seismic force*

Where the bearing pressure due to wind is less than 25 percent of that due to dead and live loads, it may be neglected in design. Where this exceeds 25 percent foundations may be so proportioned that the pressure due to combined dead, live and wind loads does not exceed the allowable bearing pressure by more than 25 percent.

When earthquake forces are considered for the computation of design loads, the permissible increase in allowable bearing pressure of pertaining soil shall be as given in Table 3, depending upon the type of foundation of the structure.

7.1.2.5 *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).

7.1.3 *Settlement*

The permissible values of total and differential settlement for a given type of structure may be taken as given in Table 4. Total settlements of foundation due to net imposed loads shall be estimated in accordance with good practice [6-2(10)]. 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;
   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.

Table 3 Percentage of Permissible Increase in Allowable Bearing Pressure or Resistance of Soils

(*Clauses* 7.1.2.4 *and* 8.2.7)

Sl No.	Foundation	Type of Soil Mainly Constituting the Foundation		
		Type I — Rock or Hard Soil: Well graded gravel and sand gravel mixtures with or without clay binder, and clayey sands poorly graded or sand clay mixtures (GB, CW, SB, SW, and SC)[1] having N[2] above 30, where N is the standard penetration value	Type II — Medium Soils: All soils with N between 10 and 30, and poorly graded sands or gravelly sands with little or no fines (SP[1]) with $N > 15$	Type III — Soft Soils: All soils other than SP[1] with $N < 10$
(1)	(2)	(3)	(4)	(5)
i)	Piles passing through any soil but resting on soil type I	50	50	50
ii)	Piles not covered under item (i)	—	25	25
iii)	Raft foundations	50	50	50
iv)	Combined isolated RCC footing with tie beams	50	25	25
v)	Isolated RCC footing without tie beams, or unreinforced strip foundations	50	25	—
vi)	Well foundations	50	25	25

NOTES

1 The allowable bearing pressure shall be determined in accordance with good practice [6-2(7)] and [6-2(8)].

2 If any increase in bearing pressure has already been permitted for forces other than seismic forces, the total increase in allowable bearing pressure when seismic force is also included shall not exceed the limits specified above.

3 Desirable minimum field values of N — If soils of smaller N-values are met, compacting may be adopted to achieve these values or deep pile foundations going to stronger strata should be used.

4 The values of N (corrected values) are at the founding level and the allowable bearing pressure shall be determined in accordance with good practice [6-2(7)] and [6-2(8)].

Seismic Zone	*Depth Below Ground Level (in m)*	*N Values*	*Remark*
(1)	(2)	(3)	(4)
III, IV and V	≤ 5	15	For values of depths between 5 m and 10 m, linear interpolation is recommended
	≥ 5	25	
II (for important structures only)	≤ 5	15	
	≥ 10	25	

5 The piles should be designed for lateral loads neglecting lateral resistance of soil layers liable to liquefy.

6 Good practice [6-2(5)] and [6-2(9)] may also be referred.

7 Isolated RCC footing without tie beams, or unreinforced strip foundation shall not be permitted in soft soils with $N < 10$.

[1] *See* good practice [6-2(5)].

[2] *See* good practice [6-2(9)].

Table 4 Permissible Differential Settlements and Tilt (Angular Distortion) for Shallow Foundations in Soils

(*Clause* 7.1.3)

Sl No.	Type of Structure	Isolated Foundations						Raft Foundations					
		Sand and Hard Clay			Plastic Clay			Sand and Hard Clay			Plastic Clay		
		Maximum settlement mm	Differential settlement mm	Angular distortion mm	Maximum settlement mm	Differential settlement mm	Angular distortion mm	Maximum settlement mm	Differential settlement mm	Angular distortion mm	Maximum settlement mm	Differential settlement mm	Angular distortion 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	100	.002 0*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	.002 5*L*	1/400	125	.003 3*L*	1/300
	b) For load bearing walls												
	1) *L*/H = 2*	60	.000 2*L*	1/5 000	60	.000 2*L*	1/5000	←		Not likely to be encountered			→
	2) *L*/H = 7*	60	.000 4*L*	1/2 500	60	.000 4*L*	1/2500						
iv)	For water towers and silos	50	.001 5*L*	1/666	75	.001 5*L*	1/666	100	.002 5*L*	1/400	125	.002 5*L*	1/400

NOTE — The values given in the table may be taken only as a guide and the permissible total settlement/different settlement and tilt (angular distortion) in each case should be decided as per requirements of the designer.

L denotes the length of deflected part of wall/raft or centre-to-centre distance between columns.

H denotes the height of wall from foundation footing.

* For intermediate ratios of *L*/*H*, the values can be interpolated.

7.1.4 *Depth of Foundations*

7.1.4.1 The depth to which foundations should be carried depends upon the following principal factors:

a) The securing of adequate allowable 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 zone in which trouble may be expected from frost.
d) The maximum depth of scour, wherever relevant, should also be considered and the foundation should be located sufficiently below this depth.
e) Other factors such as ground movements and heat transmitted from the building to the supporting ground may be important.

7.1.4.2 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.

7.1.4.3 Where there is excavation, ditch pond, water course, filled up ground or similar condition adjoining or adjacent to the subsoil on which the structure is to be erected and which is likely to impair the stability of structure, either the foundation of such structure shall be carried down to a depth beyond the detrimental influence of such conditions, or retaining walls or similar works shall be constructed for the purpose of shielding from their effects.

7.1.4.4 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.

7.1.4.5 Clay 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.

7.1.5 *Foundation at Different Levels*

7.1.5.1 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. 2).
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. 3).
c) In case of footing of 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. 4).

7.1.5.2 The requirement given in **7.1.5.1** shall not apply under the following conditions:

a) Where adequate provision is made for the lateral support (such as, with retaining walls) of the material supporting the higher footing.
b) When the factor of safety of the foundation soil against shearing is not less than four.

7.1.6 *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 8 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

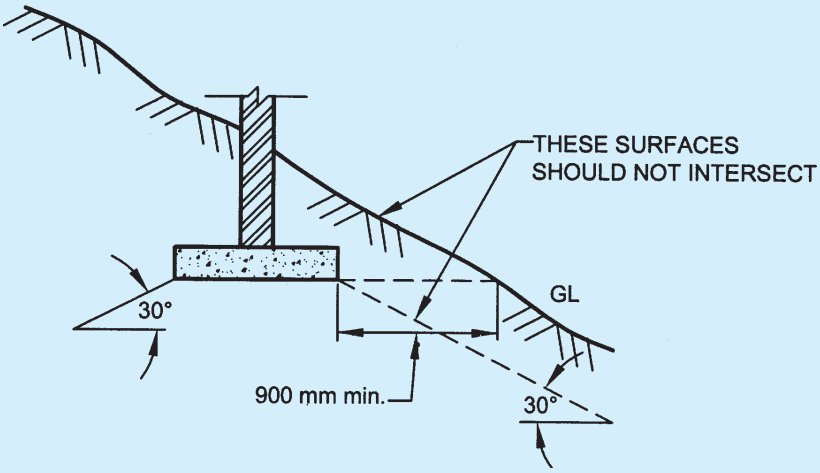


FIG. 2 FOOTING IN SLOPING GROUND

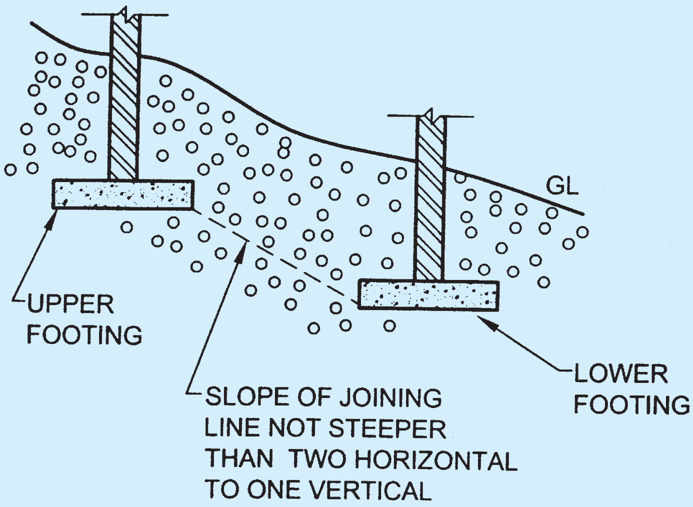


FIG. 3 FOOTING IN GRANULAR SOIL

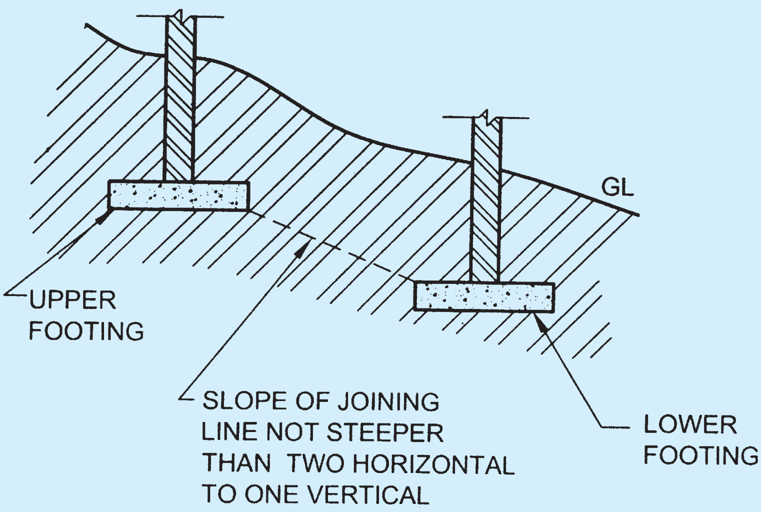


FIG. 4 FOOTING IN CLAYEY SOIL

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 sub-surface 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 must be taken to prevent such leakages.

7.1.7 *Effect of Mass Movements of Ground in Unstable Areas*

7.1.7.1 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.

7.1.7.2 *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.

7.1.7.3 *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.

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.

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.

7.1.8 *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.

7.1.9 *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.

7.1.10 *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.

7.1.11 *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. The minimum horizontal spacing between existing and new footings shall be equal to the width of the wider one. 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.

7.1.12 *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 re-designed 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 altered in accordance with Note 4 in Table 3.
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 corrected, the damaged portion repaired or the design capacity of the affected foundation recalculated to the satisfaction of the Authority.
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.

7.2 Pad or Spread and Strip Foundations

7.2.1 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.

7.2.2 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.

7.2.3 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.

7.2.4 *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.

7.2.5 *Dimensions of Foundation*

The dimensions of the foundation in plan should be such as to support loads as given in good practice [6-2(11)]. 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.

7.2.6 In the base of foundations for masonry foundation it is preferable to have the steps in multiples of thickness of masonry unit.

7.2.7 The plan dimensions of excavation for foundations should be wide enough to ensure safe and efficient working with good practice [6-2(12)].

7.2.8 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 ½ horizontal to 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.

7.2.9 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.

7.2.10 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) in compressible 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.

7.2.11 The cement concrete foundation (plain or reinforced) should be designed in accordance with good practice [6-2(13)] and masonry foundation in accordance with good practice [6-2(14)].

7.2.12 *Thickness of Footing*

The thickness of different types of footings, if not designed according to **7.1**, should be as given in Table 5.

7.2.13 *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.

7.2.14 In the foundations, the cover to the reinforcement shall be as prescribed in Part 6 'Structural Design, Section 5 Concrete for the Applicable Environment Exposure Condition'.

7.2.15 For detailed information regarding preparation of ground work, reference may be made to good practice [6-2(15)].

7.3 Raft Foundations

7.3.1 *Design Considerations*

Design provisions given in **7.1** shall generally apply.

7.3.1.1 The structural design of reinforced concrete rafts shall conform to Part 6 'Structural Design, Section 5 Concrete'.

7.3.1.2 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 6 'Structural Design, Section 5 Concrete' for the applicable environment exposure condition.

7.3.1.3 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.

7.3.1.4 Foundations subject to heavy vibratory loads should preferably be isolated.

7.3.1.5 The minimum depth of foundation shall generally be not less than 1 m.

7.3.1.5 *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.

7.3.1.7 *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.

7.3.1.8 *Properties of supporting soil*

Distribution of contact pressure underneath a raft is affected by the physical characteristics of the soil supporting it. Consideration must be given to the increased contact pressure developed along the edges of foundation on cohesive soils and the opposite effect

Table 5 Thickness of Footings

(*Clause* 7.2.12)

Sl No.	Type of Footings	Thickness of Footings, *Min*	Remarks
(1)	(2)	(3)	(4)
i)	Masonry	a) 250 mm b) Twice the maximum projection from the face of the wall	Select the greater of the two values
ii)	Plain concrete		
	For normal structures	a) 200 mm b) Twice the maximum offset in a stepped footing c) 300 mm	— For footings resting on top of the pile For footings resting on soil
	For lightly loaded structures	a) 150 mm b) 200 mm	Resting on soil Resting on pile
iii)	Reinforced concrete	a) 150 mm b) 300 mm	Resting on soil Resting on pile

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 must be based on the worst conditions.

7.3.1.9 *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.

7.3.1.10 *Rigidity of the superstructure*

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.

7.3.1.11 *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.

7.3.2 *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 loading 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 their area, hydrological information indicating ground water conditions and its seasonal variations, etc;
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.

7.3.3 *Choice of Raft Type*

7.3.3.1 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. 5A).

7.3.3.2 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. 5B).

7.3.3.3 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. 5C and 5D).

7.3.3.4 For very heavy structures, provision of cellular raft or rigid frames consisting of slabs and basement walls may be considered.

7.3.4 *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.

7.3.4.1 *Rigid foundation (conventional method)*

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.

This method may be used when either of the following conditions is satisfied:

a) The structure behaves as rigid (due to the

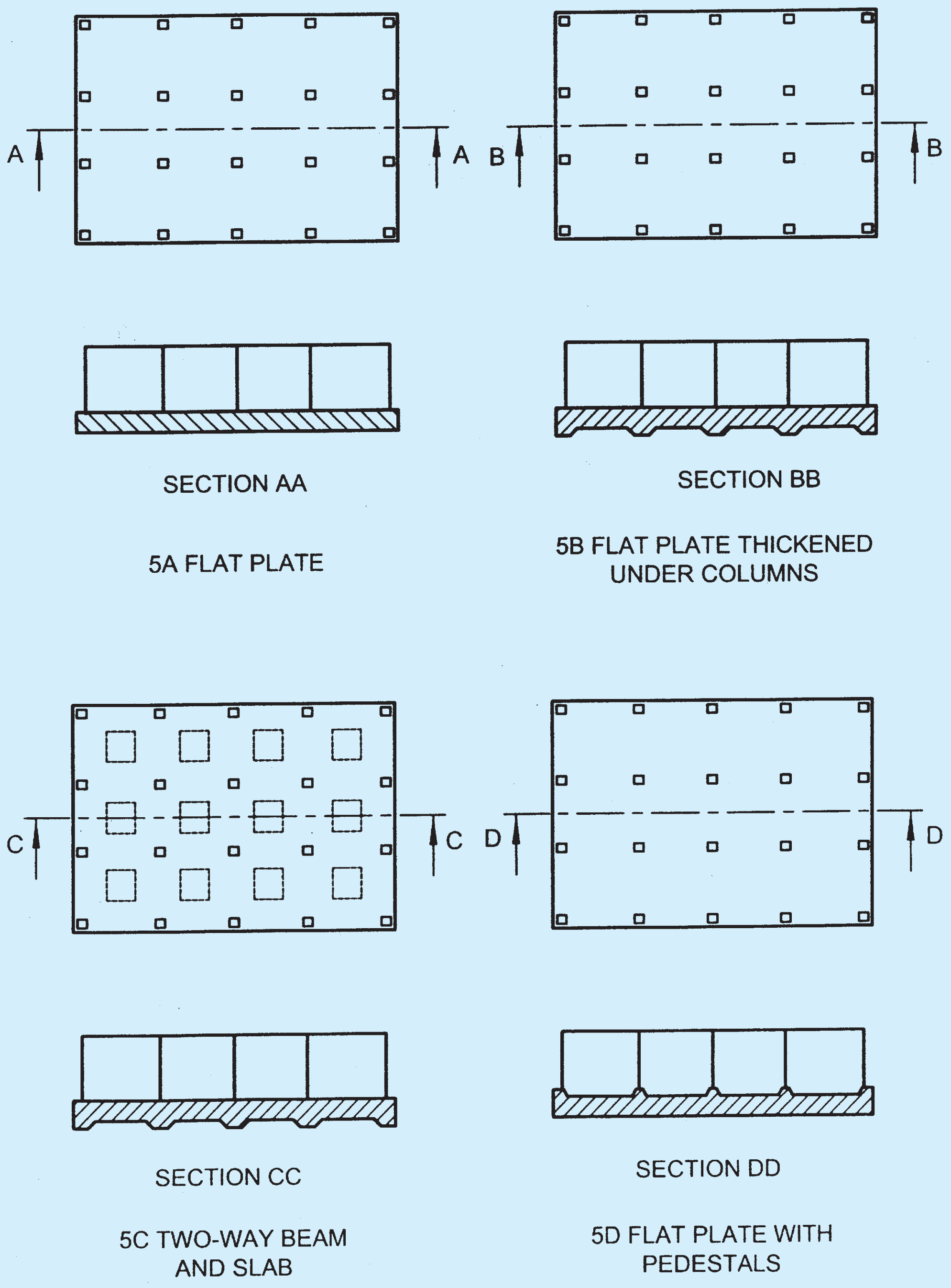


FIG. 5 COMMON TYPES OF RAFT FOUNDATION

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 $1.75/\gamma$ (*see* Annex C).

The raft is analysed 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.

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 midspans and each strip may be analysed 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.

7.3.4.2 *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:
   1) The structure (combined action of superstructure and raft) may be considered as flexible (relative stiffness factor $K > 0.5$, *see* Annex C).
   2) Variation in adjacent column load does not exceed 20 percent of the higher value.
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.

7.4 Ring Foundations

For provisions regarding ring foundations good practice [6-2(16)] shall be referred to.

8 DRIVEN/BORED CAST *IN-SITU* CONCRETE PILES

8.0 General

Piles find application in foundations to transfer load 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 yet to be fully understood, although application of pile foundations is in practice over many decades. Broadly, piles transfer axial loads either substantially by friction along their shafts and/or substantially by the end bearing. Construction of a pile foundation requires a careful choice of piling system, depending upon the subsoil conditions, the load characteristics of a structure and the limitation of total settlement, differential settlements and any other special requirement of a project.

8.1 Material

8.1.1 *Concrete*

The minimum grade of concrete to be used shall not be less than that arrived at in accordance with Part 6 ‘Structural Design, Section 5 Concrete’.

8.1.1.1 *For bored and driven cast-in-situ concrete piles including under-reamed piles*

The minimum cement content shall be 400 kg/m^3 in all conditions. For piles up to 6 m, minimum cement content of 350 kg/m^3 without provision for under water concreting may be used under favourable non-aggressive subsoil condition and where concrete of higher strength is not needed structurally or due to aggressive site conditions. For concreting in aggressive surroundings due to presence of sulphates, etc the provisions given in Part 6 ‘Structural Design, Section 5 Concrete’ shall be followed.

8.1.2 *Steel Reinforcement*

Steel reinforcement shall conform to any one of the types of steel specified in Part 6 ‘Structural Design, Section 5 Concrete’.

8.2 Design Considerations

Pile foundation shall be designed in such a way that the load from the structure it supports can be transmitted to the soil without causing any soil failure and without causing such settlement, differential or total under permanent/transient loading as may result in structural damage and/or functional distress. 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.

NOTE — When working near existing structures, care shall be taken to avoid any damage to structures.

8.2.1 *Soil Resistance*

The bearing capacity of a pile is dependent 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. A single pile is normally designed to carry load along its axis. The transverse load bearing capacity of a single pile depends on the soil reaction developed and the structural capacity of the shaft under bending. In case the horizontal loads are of higher magnitude, it is essential to investigate the phenomena using principles of horizontal subsoil reaction adopting appropriate values for horizontal modulus of the soil. Alternatively, piles may be installed in rake. The feasibility of constructing bored piles in rake under a given subsoil condition should, however, be examined critically.

8.2.1.1 The ultimate bearing capacity of a pile may be estimated approximately by means of a static formula on the basis of soil test results or by test loading. The settlement of pile obtained at safe load/ working load from load test results on a single pile shall not be directly used in forecasting the settlement of a structure unless experience from similar foundations on its settlement behaviour is available. The average settlement may be assessed on the basis of subsoil data and loading details of the structure as a whole using the principle of soil mechanics.

8.2.1.2 *Static formula*

By using static formula, the estimated value of the ultimate bearing capacity of a typical pile is obtained, the accuracy being dependent on the reliability of the formula and the reliability of the soil properties for various strata available. The soil properties to be adopted in such a formula may be assigned from results of laboratory tests and field tests as per good practice [6-2(1)]. Two separate static formulae commonly applicable for cohesive and non-cohesive soils are indicated in Annex G, to serve only as a guide. Other alternative formulae may be applicable, depending on the subsoil characteristics and method of installation of piles.

8.2.1.3 *Dynamic formula*

For driven piles in non-cohesive soils, such as gravels, coarse sand and other similar deposits, an approximate value of the bearing capacity may be determined by a dynamic pile formula as per good practice [6-2(17)]. Dynamic formulae are not directly applicable to cohesive soil deposits, such as saturated slits and clays, as the resistance to impact of the toe of the casing will be exaggerated by their low permeability, while the frictional resistance on the sides is reduced by lubrication. If as a result of test loadings on a given area a suitable coefficient can be applied to a dynamic formula, the results may then be considered as reasonable.

8.2.1.4 *Load test results*

The ultimate load capacity of a single pile is determined with reasonable accuracy from test loading as per good practice [6-2(18)]. The load test on a pile shall not be carried out earlier than four weeks from the time of casting the pile.

8.2.1.5 *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 good practice [6-2(19)].

8.2.2 *Negative Skin Friction or Dragdown Force*

When a soil stratum, through which a pile shaft has penetrated into an underlying hard stratum, compresses as a result of either its being unconsolidated or its being under a newly placed fill or as a result of re-moulding 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 downwards relative to the pile shaft. Recognition of the existence of such a phenomenon shall be made and a suitable reduction shall be made to the allowable load, where appropriate.

8.2.3 *Structural Capacity*

The piles shall have the necessary structural strength to transmit the loads imposed on them ultimately to the soil.

8.2.3.1 *Axial capacity*

Where a pile is fully embedded in the soil (having an undrained shear strength not less than 10 kN/m^2) its axial carrying capacity is not limited by its strength as a long column. Where piles are installed through very weak soils (having an undrained shear strength less than 10 kN/m^2), special consideration shall be given to determine whether the shaft would behave as a long column or not; if necessary suitable reductions shall be made in its structural strength considering 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 conditions 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 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 three times the diameter of the shaft. In weak soil (undrained shear strength less than 10 kN/m^2) such as soft clay and soft slit, 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 less. A stratum of liquid mud should be treated as if it was water. 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.

8.2.3.2 *Uplift capacity*

Total uplift capacity of pile will be the sum of the frictional resistance and weight of the pile (buoyant or total as relevant). The uplift capacity from the static formula (Annex G) can be approximately estimated by ignoring end bearing but adding weight of pile (buoyant or total as relevant). The safe uplift capacity can be obtained by applying a factor of safety 3. However, more reliance should be given to that obtained from test loading as per good practice [6-2(18)].

8.2.3.3 *Lateral load capacity*

A pile may be subjected to transverse forces 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 carrying 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 the application of a lateral load. While considering lateral load on piles, the 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 method for the determination of the depth of fixity of piles for driven cast *in-situ* and depth of fixity, lateral deflection and maximum moment for driven precast, bored cast-*in-situ* and bored precast piles required for design is given in Annex H. Other accepted methods, such as the method of Reese and Matlock or finite element analysis using linear/non-linear springs to represent the resistance of soil. A pile in a group of three or more piles connected by a rigid cap shall be designed considering as 'fixed head condition'. In caps of single piles interconnected by ground beams in two directions and for two piles by ground beams in a line transverse to the common axis of the piles is also to be considered as 'fixed head condition'. In all other conditions the pile shall be designed by treating it 'free head condition'.

NOTE — Because of limited information on horizontal modulus of soil, and requirements in the theoretical analysis, it is suggested that the adequacy of a design should be checked by an actual field load test.

8.2.3.4 *Raker piles*

Raker piles are normally provided where vertical piles cannot resist the required applied horizontal forces. 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 graphically or by analytical methods. Where necessary, due consideration should be given to 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 loose fill or consolidating deposit 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.

8.2.4 *Spacing of Piles*

The centre to centre spacing of a pile is considered from two aspects as follows:

a) Practical aspects of installing the piles; and
b) The nature of the load transfer to the soil and possible reduction in bearing capacity of a group of piles thereby.

8.2.4.1 In the case of piles founded on a very hard stratum and deriving their capacity mainly from end bearing, the spacing will be governed by the competency of the end bearing strata. The minimum spacing in such cases shall be 2.5 times the diameter of the shaft. In case of piles resting on rock, a spacing of two times the diameter may be adopted.

8.2.4.2 Piles deriving their bearing capacity mainly from friction shall be sufficiently apart to ensure that the zones of soil 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 three times the diameter of the shaft.

8.2.4.3 In the case of loose sand or filling, closer spacing than in dense sand may be possible, in driven piles since displacement during the piling may be absorbed by vertical and horizontal compaction of the strata. The minimum spacing in such strata may be two times the diameter of the shaft.

NOTE — In the case of piles of non-circular cross-section, the diameter of the circumscribing circle shall be adopted.

8.2.5 *Pile Grouping*

In order to determine the bearing capacity of a group of piles, a number of efficiency equations are in use. However, it is very 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.

8.2.5.1 The bearing capacity of a pile group may be either of the following:

a) Equal to the bearing capacity of individual piles multiplied by the number of piles in group; or
b) It may be less.

The former holds true in the case of friction piles, cast or driven into progressively stiffer materials or in end-bearing piles. In friction piles in soft and clayey soils, it is normally smaller. For driven piles in loose sandy soils, the group value may be higher due to the effect of compaction. In such a case, a load test should be made on a pile from the group after all the piles have been installed. The group capacity may then be decided by taking into account the interference effects. This would be done by multiplying the total capacity of a pile group with the group efficiency factor.

8.2.5.2 In the case of piles deriving their support mainly from friction and connected by a rigid pile cap, the group may be visualized to transmit load to the soil, as if from a column of soil, enclosed by the piles. The ultimate capacity of the group may be computed following this concept, taking into account the frictional capacity along the perimeter of the column of soil as above and the end bearing of the said column using the accepted principles of soil mechanics.

8.2.5.3 When the cap of the pile group is cast directly on a reasonably firm stratum which supports the piles, it may contribute to the bearing 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 **8.2.5.2**.

8.2.5.4 When a moment is applied on the pile group either from the superstructure or as a consequence of unavoidable inaccuracies of installation, the adequacy of the pile group in resisting the applied moment should be checked. In the case of a single pile subjected to moments due to lateral forces or eccentric loading, beams may be provided to restrain the pile caps effectively from lateral or rotational movement.

8.2.5.5 In the case of a structure supported on a single pile/group of piles, resulting in large variation in the number of piles from column to column, it is likely, depending on the type of subsoil supporting the piles, to result in a high order of differential settlement. Such high order of differential settlement may be either catered for in the structural design or it may be suitably reduced by judicious choice of variations in the actual pile loadings. For example, a single pile cap may be loaded to a level higher than that of a pile in a group in order to achieve reduced differential settlement between the adjacent pile caps supported on different number of piles.

8.2.6 *Factor of Safety*

8.2.6.1 The factor of safety should be judiciously chosen after considering the following:

a) The reliability of the value of the ultimate bearing capacity of a pile,
b) The type of superstructure and the type of loading, and
c) Allowable total/differential settlement of the structure.

8.2.6.2 When the ultimate bearing capacity is compound from either static formula or dynamic formula, the factor of safety would depend on the reliability of the formulae, depending on a particular site and locality and the reliability of the subsoil parameters employed in such computation. The minimum factor of safety on static formula shall be 2.5. The final solution of a factor of safety shall take into consideration the load settlement characteristics of the structure as a whole on a given site.

8.2.6.3 The factor of safety for assessing the safe load on piles from load test data should be increased in unfavourable conditions where:

a) settlement is to be limited or unequal settlement avoided as in the case of accurately aligned machinery or a superstructure with fragile finishings;
b) large impact or vibrating loads are expected;
c) the properties of the soil may be expected to deteriorate with time; and
d) the live load on a structure carried by friction piles is a considerable portion of the total and approximates to the dead load in its duration.

8.2.7 *Transient Loading*

The maximum permissible increase over the safe load of a pile as arising out of wind loading is 25 percent. In the case of loads and moments arising out of earthquake effects, the increase of safeload shall be as given in Table 3.

8.2.8 *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 of up to 10 percent of the pile capacity may be allowed on each pile. The total overloading on the group should not be more than 10 percent of the capacity of the group nor more than 40 percent of the allowable load on a single pile.

8.2.9 *Reinforcement*

8.2.9.1 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 (any type or grade) within the pile shaft shall be 0.4 percent of the sectional area calculated on the basis of the outside area of the casing of the shaft.

8.2.9.2 The curtailment of reinforcement along the depth of the pile, in general, depends on the type of loading and subsoil strata. In the case of piles subjected to compressive load only, the designed quantity of reinforcement may be curtailed at an appropriate level as per the design requirements. For piles subjected to uplift load, lateral load and moments, separately or with compressive loads, it may be necessary to provide reinforcement for the full depth of pile. In soft clays or loose sands, or where there is likelihood of danger to green concrete due to driving of adjacent piles, the reinforcement should be provided up 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 **8.2.9.1** should be provided in the full length of the pile.

Piles shall always be reinforced with a minimum amount of reinforcement as dowels, keeping the minimum bond length into the pile shaft and with adequate projection into the pile cap.

NOTE — In some cases the cage may lift at bottom or at the top during withdrawal of casing. This can be minimized by making the reinforcement 'U' shaped at the bottom and up to well secured joints. Also the lifting 5 percent of the length should be considered not to affect the quality of pile.

8.2.9.3 Clear cover to all main reinforcements 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 are so chosen as to impart adequate rigidity to the reinforcing cage during its handling and installation. The minimum diameter of the links or spirals shall be 6 mm and the spacing of the links or spirals shall be not less than 150 mm.

8.2.10 *Design of Pile Cap*

8.2.10.1 The pile caps may be designed considering that the reaction from any pile is concentrated at the centre of the pile. The critical section for shear in diagonal tension is taken at a distance equal to half the effective depth of cap from the face of column/pedestal or wall. For bending moment and shear for bond, the critical section is taken at the face of column/pedestal or wall for cap supporting a concrete column, pedestal or wall; half way between the centre line and the edge of the wall for caps under masonry walls and half-way between the face of the column or pedestal and the edge of the gusseted base for caps under gusseted bases. In computing the external shear or the critical section, the entire action of any pile of diameter *D* whose centre is located *D*/2 or more outside the section shall be assumed as producing no shear on the section. For intermediate positions of the pile centre, the portion of the pile reaction to be assumed as producing shear on the section shall be based on straight-line interpolation between full value at *D*/2 outside the section and zero value at *D*/2 inside the section. Further, design may be carried out as specified in Part 6 'Structural Design, Section 5 Concrete'.

8.2.10.2 The pile cap shall be deep enough to allow for necessary anchorage of the column and pile reinforcement and the minimum thickness shall be 300 mm.

8.2.10.3 The pile cap should normally be rigid enough, so that the imposed load could be distributed on the piles in a group equitably.

8.2.10.4 In the case of a large cap, where differential settlement may be imposed between piles under the same cap, due consideration should be given to the consequential moment.

8.2.10.5 The clear overhang of the pile cap beyond the outer most pile in the group shall normally be 100 mm to 150 mm, depending upon the pile size.

8.2.10.6 The cap is generally cast over a 75 mm thick levelling course of concrete. The clear cover for the main reinforcement in the cap slab shall not be less than 60 mm.

8.2.10.7 The pile should project 50 mm into the cap concrete.

8.2.11 *Grade Beams*

8.2.11.1 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.

For the design of beams, a maximum bending moment of $\frac{wl^2}{50}$, where w is uniformly distributed load per metre run (worked out by considering a maximum height of two storeys in structures with load bearing walls and one storey in framed structures) and l is the effective span in metres, will be taken if the beams are supported during construction till the masonry above it gains strength. The value of bending moment shall be increased to $\frac{wl^2}{30}$, if the beams are not supported. For considering composite action, the minimum height of wall shall be 0.6 times the beam span. 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.

8.2.11.2 The minimum overall depth of grade beams shall be 150 mm. The reinforcement at the bottom should be kept continuous and an equal amount may be provided at top to a distance of a quarter span both ways from pile centres. The longitudinal reinforcement both at top and bottom should not be less than three bars of 10 mm diameter mild steel (or equivalent deformed steel) and stirrups of 6 mm diameter bars should be spaced at a minimum of 300 mm spacing.

8.2.11.3 In expansive soils, the grade beams shall be kept a minimum of 80 mm clear off the ground. In other soils, beams may rest on ground over a levelling concrete course of about 80 mm (*see* Fig. 6).

8.2.11.4 In the case of exterior beams over piles in expensive soils, an edge projection of 75 mm thickness and extending 80 mm into ground (*see* Fig. 6) shall be provided on the outer side of the beam.

8.3 For detailed information on driven/bored cast *in-situ* concrete piles regarding control of piling, installation, defective pile and recording of data, reference may be made to good practice [6-2(17)].

9 DRIVEN PRECAST CONCRETE PILES

9.1 Provisions of **8** except **8.2.9** shall generally apply.

9.2 Design of Piles

9.2.1 The 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, driving stresses without damage.

9.2.2 Any shape having radial symmetry will be satisfactory for precast piles. The most common cross-sections used are square and octagonal or circular.

9.2.3 Where exceptionally long lengths of piles are required, hollow sections may advantageously be used. If the final conditions require a larger cross-sectional area, the hollow sections may be filled with concrete after driving in position.

9.2.4 Excessive whippiness in handling precast pile may generally be avoided by limiting the length of pile to a maximum of 50 times the least width.

9.2.5 *Lifting and Handling Stresses*

Stresses induced by bending in the cross-section of a precast pile during lifting and handling may be estimated just as for any reinforced concrete section in accordance with relevant provisions of good practice [6-2(13)]. The calculations with regard to moments depending on the method of support during handling will be as given below. Excessive whippiness in handling precast pile may generally be avoided by limiting the length of pile to a maximum of 50 times the least width.

Number of Points of Pick Up	*Location of Point of Support from in Terms of Length of Pile for Minimum Moments*	*Bending Moment to be Allowed for Design*
One	0.293 L	$\frac{WL}{23.3}$
Two	0.207 L	$\frac{WL}{46.6}$
Three	0.145 L, the middle point will be at the centre	$\frac{WL}{95}$

where

W = Mass of pile in kg, and

L = Length in metres.

During hoisting the pile will be suspended at one point near the head and the bending moment will be the least when it is pulled in a distance of 0.293 L, and the value of bending moment will be:

$$\frac{WL}{23.3}$$

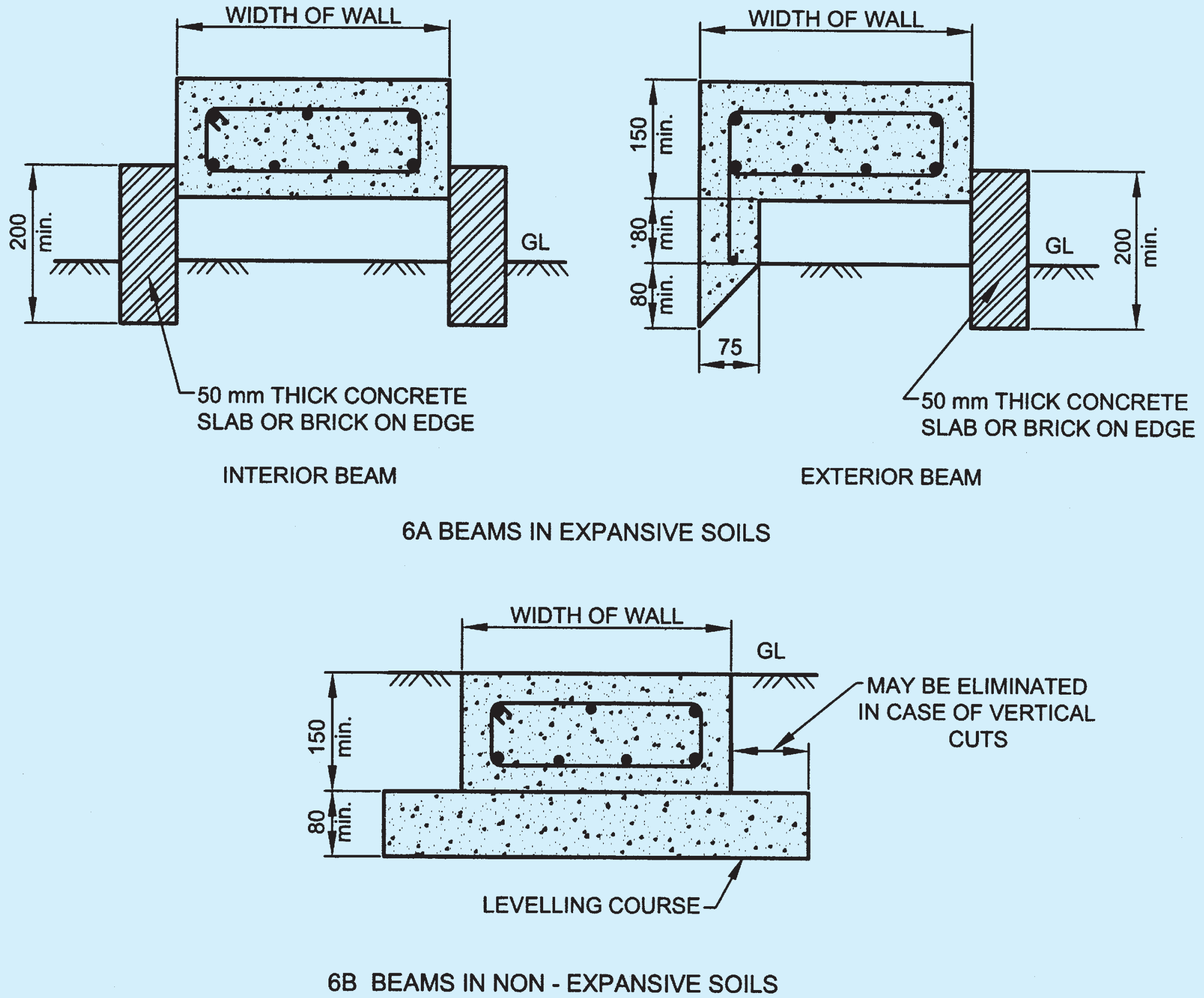
9.3 Reinforcement

9.3.1 The longitudinal reinforcement shall be provided in precast reinforced concrete piles for the entire length. All the main longitudinal bars shall be of the same length with lap welded at joints 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 the main longitudinal reinforcement shall not be less than the following percentages of the cross-sectional area of the piles:

a) For piles with length less than 30 times the least width — 1.25 percent
b) For piles with length 30 to 40 times the least width — 1.5 percent, and
c) For piles with length greater than 40 times the least width — 2 percent.

9.3.2 The lateral reinforcement is of particular importance in resisting the driving stresses induced in the piles and should be in the form of hoops or links and of diameter not less than 6 mm. The volume of lateral reinforcement shall not be less than the following:

a) At each end of the pile for a distance of about 3 times the least width — not less than 0.6 percent of the gross volume of that part of the pile; and



All dimensions in millimetres.

FIG. 6 TYPICAL SECTIONS OF GRADE BEAMS

b) In the body of the pile — not less than 0.2 percent of the gross volume of the pile.

The spacing shall be such as to permit free flow of concrete around it. The transition between the close spacing of lateral reinforcement near the ends and the maximum spacing shall be gradually over a length of 3 times the least width of the pile.

9.3.3 The cover of concrete over all the reinforcement, including ties, should not be less than 40 mm. But where the piles are exposed to sea-water or water having other corrosive content, the cover should be nowhere less than 50 mm. Cover should be measured clear from the main or longitudinal reinforcement.

NOTE — Where concrete of the pile is liable to be exposed to the attack of sulphates and chlorides present in the ground water, the piles may be coated with a suitable material.

9.3.4 Piles should be provided with flat or pointed co-axial shoes if they are driven into or through ground, such as rock, coarse gravel, clay with cobbles and other soils likely to damage the concrete at the tip of the pile. The shoe can be of steel or cast iron. In uniform clay or sand, the shoe may be omitted.

Where jetting is necessary for concrete piles, a jet tube may be cast into the pile, the tube, being connected to the pile shoe which is provided with jet holes. Generally, a central jet is inadvisable, as it is liable to become choked. At least two jet holes will be necessary on opposite sides of the shoe, four holes giving best results. Alternatively, two or more jet pipes may be attached to the sides of the pile.

9.4 For detailed information regarding casting and curing, storing and handling, control of pile driving and recording of data, reference may be made to good practice [6-2(20)].

10 BORED PRECAST CONCRETE PILES

10.1 Provisions of **9** except **9.3** shall generally apply.

10.2 For grouting the space around the pile, the precast pile should be provided with a central duct/hole or suitable jet holes to pump the grouting material. The bottom end of the pile shall have proper arrangements for flushing/cleaning for grouting.

10.3 Reinforcement

The longitudinal reinforcement shall be provided in for the entire length preferably of high yield strength to withstand the handling stresses to the extent to meet requirement as given in **9.2.5**. All the main longitudinal bars shall be of the same length. The area of the main longitudinal reinforcement of any type and grade shall not be less than 0.4 percent of the cross-section area of the piles or as required to cater for handling stresses whichever is greater. The lateral reinforcement shall be links or spirals preferably of not less than 6 mm diameter bars. The cover of concrete over all the reinforcement including bending wire should not be less than 40 mm, but where the piles are exposed to the sea-water or water having other corrosive contents the cover should be no where less than 50 mm. 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.

10.4 For detailed information regarding casting and curing, storing and handling, control of pile installation and recording of data, reference may be made to good practice [6-2(21)].

11 UNDER-REAMED PILES

11.0 General

Under-reamed piles are bored cast *in-situ* and bored compaction concrete types having one or more bulbs formed by enlarging the borehole for the pile stem. 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 favourable. 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 underreamed 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 can 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.

11.1 Materials

11.1.1 Provisions of **8.1** shall generally apply.

11.2 Design Considerations

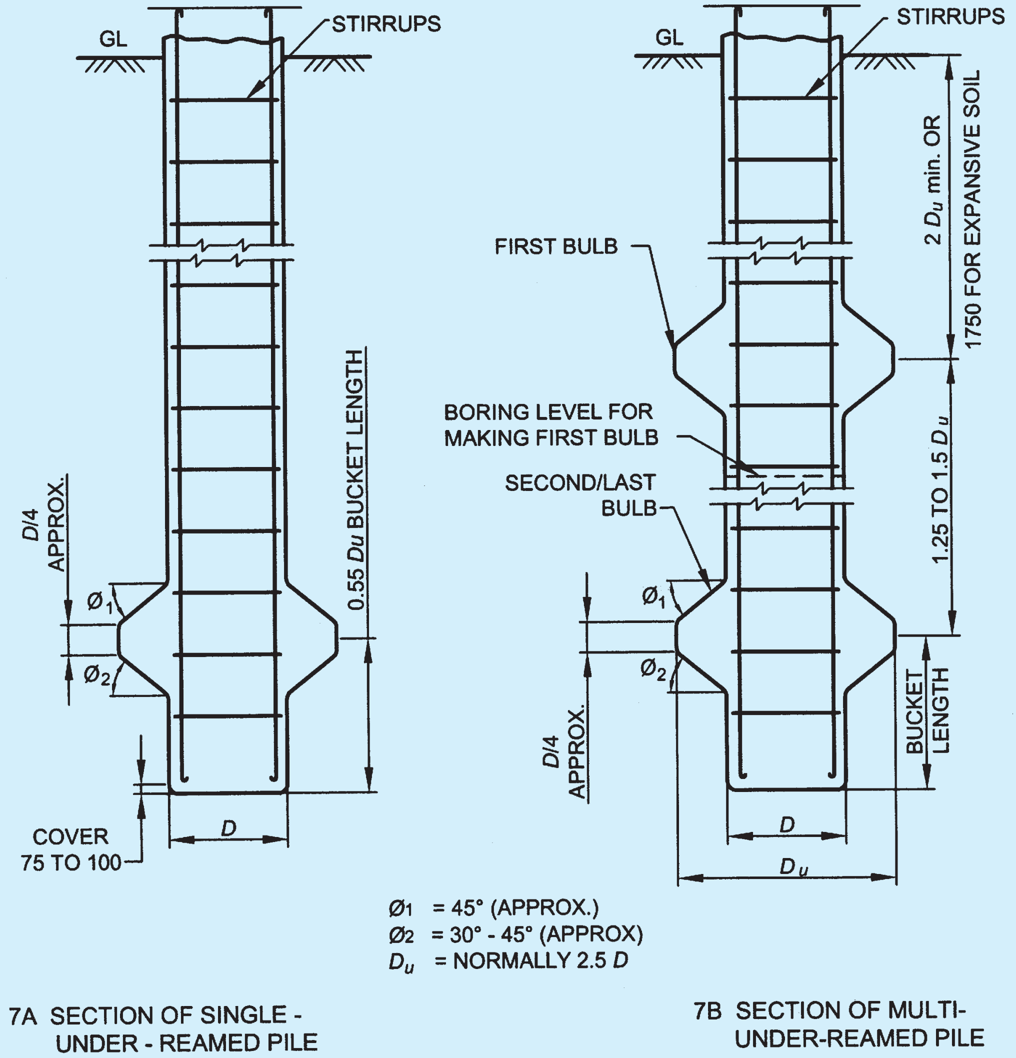
11.2.1 *General*

Under-reamed pile foundation shall be designed in such a way that the load from the structure they support can be transmitted to the soil without causing failure of soil or failure of pile material and without causing settlement (differential or total) under permanent transient loading as may result in structural damage and/or functional distress (*see* Fig. 7).

11.2.1.1 In deep deposits of expansive soils the minimum length of piles, irrespective of any other considerations, shall be 3.5 m below ground level. If the expansive soil deposits are of shallow depth and overlying on non-expansive soil strata of good bearing or rock, piles of smaller length can also be provided. In recently filled up grounds or other strata or poor bearing the piles should pass through them and rest in good bearing strata.

11.2.1.2 The minimum stem diameter of under-reamed pile can be 200 mm up to 5 m depth in dry conditions, that is strata with low water table. The minimum stem diameter for piles up to 5 m depth in strata with high water table within pile depth, shall be 300 mm for normal under-reamed pile and 250 mm for compaction under-reamed pile. For piles of more than 5 m depth, the minimum diameter in two cases shall be 375 mm and 300 mm respectively. The minimum diameter of stem for strata consisting of harmful constituents, such as sulphates, should also be 375 mm.

11.2.1.3 The diameter of under-reamed bulbs may vary from 2 to 3 times the stem diameter, depending,



All dimensions in millimetres.

FIG. 7 TYPICAL DETAILS OF BORED CAST *IN-SITU* UNDER-REAMED PILE FOUNDATION

upon the feasibility of construction and design requirements. In bored cast *in-situ* under-reamed piles and under-reamed compaction piles, the bulb diameter shall be normally 2.5 and 2 times the stem diameter respectively.

11.2.1.4 For piles of up to 300 mm diameter, the spacing of the bulbs should not exceed 1.5 times the diameter of the bulb. For piles of diameter greater than 300 mm, spacing can be reduced to 1.25 times the bulb diameter.

11.2.1.5 The topmost bulb should be at a minimum depth of two times the bulb diameter. In expansive soils it should also be not less than 2.75 m below ground level. The minimum clearance below the underside of pile cap embedded in the ground and the bulb should be a minimum of 1.5 times the bulb diameter.

11.2.1.6 Under-reamed piles with more than one bulb are not advisable without ensuring their feasibility in strata needing stabilization of bore holes by drilling mud. The number of bulbs in the case of bored compaction piles should also not exceed one in such strata.

11.2.1.7 Under-reamed batter piles without lining in dry conditions, that is, strata with low water table can be constructed with batter not exceeding 15°.

11.2.2 *Safe Load*

Safe load on a pile can be determined:

a) by calculating the ultimate load from soil properties and applying a suitable factor of safety as given in Annex J;
b) by load test on pile as good practice [6-2(18)]; and
c) from safe load tables.

11.2.2.1 *Load test and non-destructive testing*

Provisions of **8.2.1.4** and **8.2.1.5** shall generally apply.

11.2.2.2 In the absence of detailed sub-soil 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 in accordance with good practice [6-2(22)].

11.2.3 *Negative Skin Friction or Dragdown Force*

Provisions of **8.2.2** shall generally apply subject to the condition that the under-reamed bulb is provided below the strata susceptible to negative skinfriction.

11.2.4 *Structural Capacity*

Provisions of **8.2.3** 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 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 H or as per other accepted methods.

11.2.5 *Spacing*

11.2.5.1 Generally the centre to centre spacing for bored cast *in-situ* under-reamed piles in a group should be two times the bulb diameter (2 D_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, an aveage value of bulb diameter should be taken for spacing.

11.2.6 *Group Efficiency*

For bored cast *in-situ* under-reamed piles at a usual spacing of 2 D_u, the group efficiency will be equal to the safe load of an individual pile multiplied by the number of piles in the group. For piles at a spacing of 1.5 D_u, the safe load assigned per pile in a group should be reduced by 10 percent.

In under-reamed compaction piles, at the usual spacing of 1.5 D_u, the group capacity will be equal to the safe load on an individual pile multiplied by the number of piles in the group.

11.2.7 *Transient and Overloading*

Provisions of **8.2.7** and **8.2.8** shall generally apply.

11.2.8 *Reinforcement*

11.2.8.1 The minimum area of longitudinal reinforcement (any type or grade) within the pile shaft shall be 0.4 percent of the sectional area calculate on the basis of outside area of the shaft or casing if used. Reinforcement is to be provided in full length and further a minimum of 3 bars of 10 mm diameter mild steel or three 8 mm diameter high strength steel bars shall be provided. Transverse reinforcement shall not be less than 6 mm diameter at a spacing of not more than the stem diameter or 300 mm, whichever is less.

In under-reamed compaction piles, a minimum number of four 12 mm diameter bars shall be provided. For piles of lengths exceeding 5 m and of 375 mm diameter, a minimum number of six 12 mm diameter bars shall be provided. For piles exceeding 400 mm diameter, a minimum number of six 12 mm diameter bars shall be provided. The circular stirrups for piles of lengths exceeding 5 m and diameter exceeding 375 mm shall be minimum 8 mm diameter bars.

For piles in earthquake prone areas, a minimum number of six bars of 10 mm diameter shall be provided. Also transverse reinforcement in the form of stirrups or helical should be at 150 mm centre-to-centre in top few metre depth.

11.2.8.2 The minimum clear cover over the longitudinal reinforcement shall be 40 mm. In aggressive environment of sulphates etc, it may be increased to 75 mm.

11.2.9 The design of pile cap and grade beams shall conform to the requirements specified in **8.2.10** and **8.2.11** respectively.

11.2.10 For detailed information on under-reamed piles regarding control of pile, installation, reference may be made to good practice [6-2(22)].

12 TIMBER PILES

12.1 Materials

12.1.1 *Timber*

The timber shall have the following characteristics:

a) Only structural timber shall be used for piles (*see* Part 6 'Structural Design, Section 3 Timber and Bamboo, 3A Timber');

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.

12.2 Design Considerations

12.2.1 *General — See* **8.0**.

12.2.2 *Soil Resistance — See* **8.2.1**.

12.2.3 *Structural Capacity*

The pile shall have the necessary structural strength to transmit the load imposed on it to the soil. Load tests shall be conducted on a single pile or preferably on a group of piles. For compaction piles, test should be done on a group of piles with their caps resting on the ground as good practice [6-2(18)]. If such test data is not available, the load carried by the pile shall be determined by the Engineering News formula (*see* Note).

NOTE — For timber piles, the load carried shall be determined by the Engineering News formula given below. Care shall be taken that while counting the number of blows, the head of the timber pile is not broomed or brushed and in case of interrupted driving counting shall be done after 300 mm of driving.

For piles driven with drop hammer,

$$P = \frac{160\,WH}{S + 25}$$

For piles driven with single-acting steam hammer,

$$P = \frac{160\,WH}{S + 2.5}$$

where

P = Safe load on pile in kN,
W = Weight of monkey in kN,
H = Free fall of monkey in m, and
S = Penetration of pile in mm to be taken as the average of the last three blows.

12.2.4 For detailed information on timber piles regarding spacing, classification, control of pile driving, storing and handling, reference may be made to good practice [6-2(23)].

13 OTHER FOUNDATIONS

13.1 Pier Foundations

13.1.1 *Design Considerations*

13.1.1.1 *General*

The design of concrete piers shall conform to the requirements for columns specified in Part 6 'Structural Design, Section 5 Concrete'. 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** and **9**).

13.1.1.2 *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:

$$f_c' = f_c\left(1.3 - \frac{H}{20D}\right)$$

where

f'_c = Reduced allowable stress,
f_c = Allowable stress,
H = Height of pier, and
D = Least lateral dimension.

NOTE — The above provision shall not apply for piers where the least lateral dimension is 1.8 m or greater.

13.1.1.3 *Reinforced concrete piers*

When the height of the pier exceeds 18 times its least dimension, the maximum load shall not exceed:

$$P' = P\left(1.5 - \frac{H}{36D}\right)$$

where

P' = Permissible load;
P = Permissible load when calculated as axially loaded short column,
H = Height of the pier measured from top of bell, if any, to the level of cut-off of pier; and
D = Least lateral dimension.

13.2 Design of foundation units not already covered by this section, such as well foundations, machine foundations, shell foundations, etc, may be designed and constructed in accordance with good practice [6-2(24)].

14 GROUND IMPROVEMENT

In poor and weak subsoils, the design of conventional shallow 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, developed over the recent years 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. Selection of ground improvement techniques may be done in accordance with good practice [6-2(25)].

Use of suitable geo-synthetics/geo-textiles may be made in an approved manner for ground improvement, where applicable; *see also* good practice [6-2(26)].

ANNEX A

(*Clause* 7.3.1.11)

DETERMINATION OF MODULUS OF ELASTICITY (E_s) AND POISSON'S RATIO (μ)

A-1 DETERMINATION OF MODULUS OF ELASTICITY (E_s)

A-1.1 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 (**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 good practice [6-2(8)].

$$E_s = qB\frac{(1-\mu^2)}{s}\, l_w$$

where

- q = Intensity of contact pressure,
- B = Least lateral dimension of test plate,
- s = Settlement,
- μ = Poisson's ratio, and
- l_w = Influence ratio
  = 0.82 for a square plate.

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:

$$E_s = E_p \frac{B_f}{B_p}\left[\frac{B_f + B_p)}{2\,B_f}\right]^2$$

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 good practice [6-2(1)]. 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 good practice [6-2(4)] on samples collected with least disturbances.

A-3.2 In the first phase of the tri-axial 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.

ANNEX B

(*Clause* 7.3.1.11)

DETERMINATION OF MODULUS OF SUBGRADE REACTION

B-1 GENERAL

B-1.1 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 6 for cohesionless soils and in Table 7 for cohesive soils. Unless more specific determination of k is done (*see* **B-2** and **B-3**) these values 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 extra-polation 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 good practice [6-2(27)]. 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* 6-2(4)].

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:

$$k = 0.65 \times \left(\frac{E_s B^4}{EI} \right)^{1/12} \frac{E_s}{(1-\mu^2)} \frac{1}{B}$$

Table 6 Modulus of Subgrade Reaction (k) for Cohesionless Soils

(*Clause* B-1.1)

Soil Characteristic		[1] Modulus of Subgrade Reaction (k) kN/m^3	
Relative Density	Standard Penetration Test Value (N) (Blows per 300 mm)	For Dry or Moist State	For Submerged State
(1)	(2)	(3)	(4)
Loose	< 10	15 000	9 000
Medium	10 to 30	15 000 to 47 000	9 000 to 29 000
Dense	30 and over	47 000 to 180 000	29 000 to 108 000

[1] The above values apply to a square plate 300 mm × 300 mm or beams 300 mm wide.

Table 7 Modulus of Subgrade Reaction (k) for Cohesive Soils

(*Clause* B-1.1)

Soil Characteristic		[1] Modulus of Subgrade Reaction (k) kN/m^3
Consistency	Unconfined Compressive Strength, kN/m^2	
(1)	(2)	(3)
Stiff	100 to 200	27 000
Very Stiff	200 to 400	27 000 to 54 000
Hard	400 and over	54 000 to 108 000

[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.

where

E_s = Modulus of elasticity of soil (*see* Annex A),
E = Young's modulus of foundation material,
μ = Poisson's ratio of soil,
I = Moment of inertia of the foundation, and
B = Width of the footing.

B-4 CALCULATIONS

When the structure is rigid (*see* Annex C), the average modulus of subgrade reaction may also be determined as follows:

$$k_s = \frac{\text{Average contact pressure}}{\text{Average settlement of the raft}}$$

ANNEX C

(*Clauses* 7.3.4.1, 7.3.4.2 *and* B-4)

RIGIDITY OF SUPERSTRUCTURE AND FOUNDATION

C-1 DETERMINATION OF THE RIGIDITY OF THE STRUCTURE

C-1.1 The flexural rigidity *EI* of the structure of any section may be estimated according to the relation given below (*see also* Fig. 8)

$$EI = \frac{E_1 I_i b^2}{2\,H^2} + \sum E_2 I_b \left[1 + \frac{(I'_u + I'_1) b^2}{(I'_b + I'_u + I'_f) I^2} \right]$$

where

E_1 = Modulus of elasticity of the infilling material (wall material) in kN/m^2,
I_i = Moment of inertia of the infilling in m^4,
b = Length or breadth of the structure in the direction of bending in m,
H = Total height of the infilling in m,
E_2 = Modulus of elasticity of the frame material in kN/m^2,
I_b = Moment of inertia of the beam in m^4,

$I'_u = \dfrac{I_u}{h_u}$

$I'_1 = \dfrac{I_1}{h_1}$

$I'_b = \dfrac{I_b}{I}$

I = Spacing of the columns in m,
h_u = Length of the upper column in m,
h_i = Length of the lower column in m,

$I'_f = \dfrac{I_f}{L}$

I_u = Moment of inertia of the upper column in m^4,
I_i = Moment of inertia of the lower column in m^4, and
I_f = Moment of inertia of the foundation beam or raft in m^4 .

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_1 becomes zero, whereas for the topmost beam I'_u becomes zero.

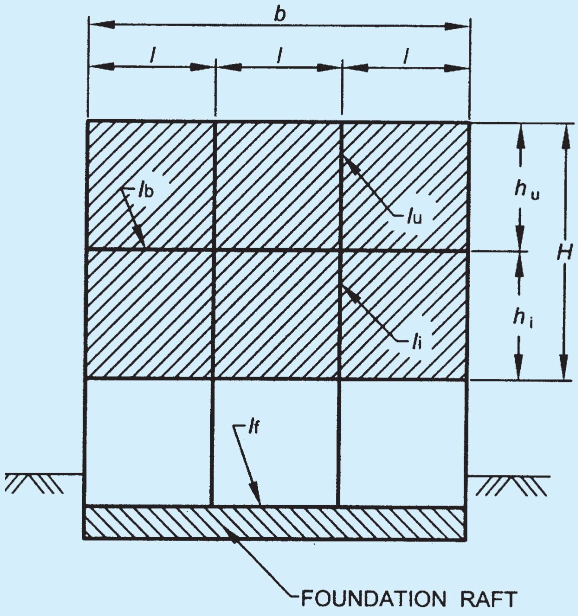


FIG. 8 DETERMINATION OF RIGIDITY OF STRUCTURE

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, $K = \dfrac{EI}{E_s b^3 a}$

b) For rectangular rafts, $K = \dfrac{E}{12\,E_s}\left(\dfrac{d}{b}\right)^3$

c) For circular rafts, $K = \frac{E}{12\,E_s}\left(\frac{d}{2R}\right)^3$

where

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,

b = Length of the section in the bending axis in m,

a = Length perpendicular to the section under investigation in m,

d = Thickness of the raft or beam in m, and

R = Radius of the raft in m.

C-2.1.1 For $K > 0.5$, the foundation may be considered as rigid.

C-3 DETERMINATION OF CRITICAL COLUMN SPACING

C-3.1 Evaluation of the characteristics λ is made as follows:

$$\lambda = \left(\frac{kB}{4\,E_c I}\right)^{1/4}$$

where

k = Modulus of subgrade reaction in kN/m^3 for footing of width B in m (*see* Annex B),

B = Width of raft B in m,

E_c = Modulus of elasticity of concrete in kN/m^2, and

I = Moment of inertia of raft in m^4.

ANNEX D

(*Clause* 7.3.4.1)

CALCULATION OF PRESSURE DISTRIBUTION BY CONVENTIONAL METHOD

D-1 DETERMINATION OF PRESSURE DISTRIBUTION

D-1.1 The pressure distribution (q) under the raft shall be determined by the following formula:

$$q = \frac{Q}{A} \pm \frac{Qe'_y}{I'_x}\,y \pm \frac{Qe'_x}{I'_y}\,x$$

where

Q = Total vertical load on the raft,

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, and

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.

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

I_x, I_y = Moment of inertia of the area of the raft respectively about the x and y axes through the centroid,

I_{xy} = $\int xy\,dA$ for the whole area about x and y axes through the centroid, and

e_x, e_y = Eccentricities in the x and y directions of the load from the centroid.

For a rectangular raft, the equation simplifies to:

$$q = \frac{Q}{A}\left(1 \pm \frac{12\,e_y\,y}{b^2} \pm \frac{12\,e_x\,x}{a^2}\right)$$

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 nagative 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.

ANNEX E

(*Clause* 7.3.4.2)

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. 9 A):

$$p_i = \frac{5\,P_i}{\bar{l}} + \frac{48\,M_i}{(\bar{l})^2}$$

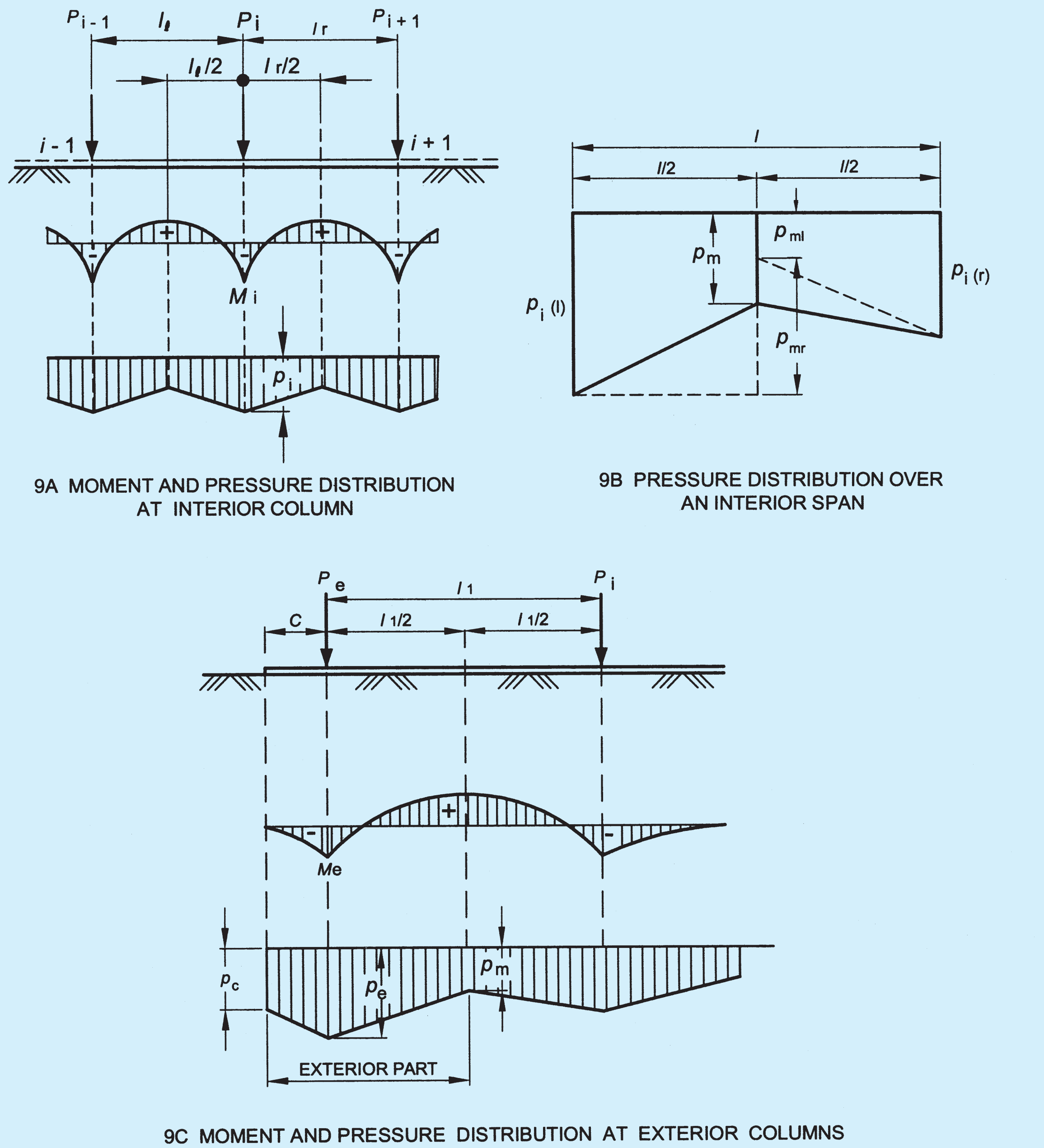


FIG. 9 MOMENT AND PRESSURE DISTRIBUTION AT COLUMNS

where

$\bar{l}$ = Average length of adjacent span (m),

P_i = Column load at point i, and

M_i = Moment under an interior columns loaded at 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. 9A and 9B).

$$p_{ml} = 2\,P_i\left(\frac{l_r}{l_l\,\bar{l}}\right) - p_i\left(\frac{\bar{l}}{l_l}\right)$$

$$p_{mr} = 2\,P_i\left(\frac{l_r}{l_r\,\bar{l}}\right) - p_i\left(\frac{\bar{l}}{l_r}\right)$$

$$p_m = \frac{p_{mr} + p_{mi}}{2}$$

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. 9C):

$$p_e = \frac{4\,p_e + \frac{6\,M_e}{C} - p_m . l_l}{C + l_l}$$

$$p_e = -\frac{3\,M_e}{C^2} - \frac{p_2}{2}$$

E-1.5 If **E-2.3** (b) governs the moment under the exterior columns, the contact pressures are determined as (*see* Fig. 9C):

$$p_e = p_c = \frac{4\,p_e - p_m l_r}{4\,C + l_l}$$

E-2 BENDING MOMENT DIAGRAM

E-2.1 The bending moment under an interior column located at i (*see* Fig. 9A) 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. 9A):

$$M_m = M_o + M_i$$

where

M_o = Moment of simply supported beam

$$= \frac{(\bar{l})^2}{48}\left[p_i(l) + 4\,p_m + p_i(r)\right]$$

M_i = Average of negative moments M_i at each end of the bay.

E-2.3 The bending moment M_e under exterior columns can be determined as the least of (*see* Fig. 9C):

a) $M_{el} = -\dfrac{P_e}{4\,\lambda}(0.13\,\lambda l_l + 1.06\,\lambda C - 0.50)$

b) $M_{e2} = -\dfrac{(4\,P_e - p_m l_l)}{(4\,C + l_l)}\,\dfrac{C^2}{2}$

ANNEX F

(*Clause* 7.3.4.2)

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:

$$M_r = -\frac{P}{4}\left[Z_4\left(\tfrac{r}{L}\right) - (1-\mu)\frac{Z_3'\left(\tfrac{r}{L}\right)}{\left(\tfrac{r}{L}\right)}\right]$$

$$M_t = -\frac{P}{4}\left[\mu Z_4\left(\tfrac{r}{L}\right) + (1-\mu)\frac{Z_3'\left(\tfrac{r}{L}\right)}{\left(\tfrac{r}{L}\right)}\right]$$

$$w = \frac{PL^2}{4\,D}\cdot Z_3\left(\tfrac{r}{L}\right)$$

where

P = Column load,

r = Distance of the point under investigation from column load along radius, and

L = Radius of effective stiffness

$$\left(\frac{D}{k}\right)^{1/4}$$

where

k = Modulus of subgrade reaction for footing of width B,

D = Flexural rigidity of the foundation,

$$P = \frac{Et^2}{12\,(1-\mu^2)},$$

t = Raft thickness,

E = Modulus of elasticity of the foundation material,

μ = Possion's ratio of the foundation material, and

$$\left.\begin{array}{l} Z_3\left(\frac{r}{L}\right), \\ Z_3'\left(\frac{r}{L}\right), \\ Z_4\left(\frac{r}{L}\right), \end{array}\right] = \text{Functions of shear, moment and deflection (\textit{see} Fig. 10)}$$

F-1.2 The radial and tangential moments can be converted to rectangular co-ordinates:

$$M_x = M_r \cos^2\phi + M_t \sin^2\phi$$
$$M_y = M_r \sin^2\phi + M_t \cos^2\phi$$

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:

$$Q = -\frac{P}{4\,L}\quad Z_4'\left(\frac{r}{L}\right)$$

$Z_4'\left(\frac{r}{L}\right)$ = function for shear (*see* Fig. 10).

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.

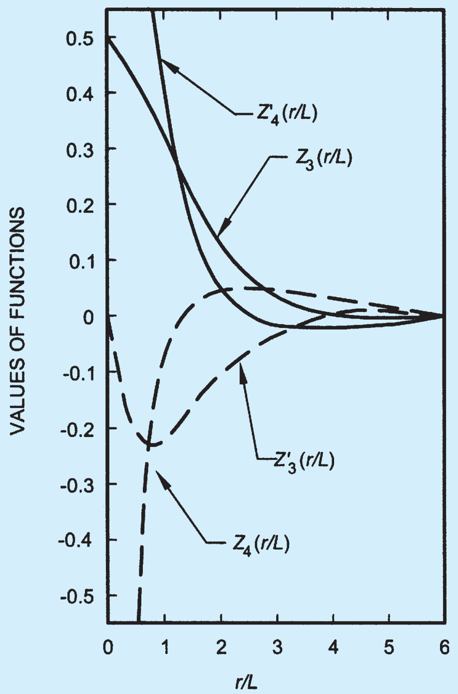


FIG. 10 FUNCTIONS FOR SHEAR MOMENT AND DEFLECTION

ANNEX G

(*Clauses* 8.2.1.2 *and* 8.2.3.2)

LOAD CARRYING CAPACITY — STATIC FORMULA

G-1 PILES IN GRANULAR SOILS

G-1.1 The ultimate bearing capacity (Q_u) of piles in granular soils is given by the following formula:

$$Q_u = A_p(\tfrac{1}{2}\,D.\gamma.\,N_\gamma + P_D.N_q) + \sum_{i=1}^{n} K.P_{Di}.\tan\delta.A_{si}$$

where

A_p = Cross-sectional area of pile toe in m^2;

D = Stem diameter in m;

γ = Effective unit weight of soil at pile toe in kN/m^3;

P_D = Effective overburden pressure at pile toe in kN/m^2;

N_y, N_q = Bearing capacity factors and depending upon the angle of internal friction ø, at toe;

$\sum_{i=1}^{n}$ = Summation for n layers in which pile is installed;

K = Coefficient of earth pressure;

P_{Di} = Effective overburden pressure in kN/m^2 for the ith layer, where i varies from 1 to n;

δ = Angle of wall friction between pile and soil, in degrees (may be taken equal to ϕ); and

A_{si} = Surface area of pile stem in m² in the *i*th layer, where *i* varies from 1 to *n*.

NOTES

1 For N_γ factors refer to good practice [6-2(7)].

2 N_q factor will depend, apart from nature of soil on the type of pile and the method of its construction and the values are given in Fig. 11 and Fig. 12 for bored and driven piles respectively.

3 The earth pressure coefficient *K* depends on the nature of soil strata, type of pile and the method of its construction. For driven piles in loose to medium sands, *K* values of 1 to 3 should be used. For bored piles, *k* values can be taken between 1 and 2.

4 The angle of wall friction may be taken equal to the angle of shear resistance of soil.

5 In working out pile capacities using static formula, for piles larger than 15 to 20 pile diameters, the maximum effective overburden at the pile tip should correspond to pile length equal to 15 to 20 times of the diameters.

G-2 PILES IN COHESIVE SOILS

G-2.1 The ultimate bearing capacity of piles (Q_u) in cohesive soil is given by the following formula:

$$Q_u = A_p N_c \,.\, C_p + \alpha \bar{C} \,.\, A_s$$

where

A_p = Cross-sectional area of pile toe in m²,

N_c = Bearing capacity factor usually taken as 9,

C_p = Average cohesion at pile tip in kN/m²,

α = Reduction factor,

$\bar{C}$ = Average cohesion throughout the length of pile in kN/m², and

A_s = Surface area of pile shaft in m².

NOTES

1 The following values of α may be taken, depending upon the consistency of the soils:

Consistency	*N Value*	*Value of α*	
		Bored piles	Driven cast *in-situ* piles
(1)	(2)	(3)	(4)
Soft to very soft	< 4	0.7	1
Medium	4 to 8	0.5	0.7
Stiff	8 to 15	0.4	0.4
Stiff to hard	>15	0.3	0.3

2 a) Static formula may be used as a guide only for bearing capacity estimates. Better reliance may be put on load test on piles.

b) For working out safe load, a minimum factor of safety 2.5 should be used on the ultimate bearing capacity estimated by static formulae.

3 In case of soft to very soft soils which are not sensitive, the value of α can be taken up to 1.

G-3 When full static penetration data is available for the entire depth, the following correlations may be used as a guide for the determination of shaft resistance of a pile:

Type of Soil	*Local Side Friction* f_s
Clays and peats where $q_c < 1$	$\frac{q_c}{3} < f_s < \frac{q_c}{1}$
Clays	$\frac{q_c}{2.5} < f_s < \frac{2\,q_c}{2.5}$
Silty clays and silty sands	$\frac{q_c}{10} < f_s < \frac{q_c}{2.5}$
Sands	$\frac{q_c}{10} < f_s < \frac{2\,q_c}{10}$
Coarse sands and gravels	$f_s < \frac{q_c}{15}$

where

q_c = Static point resistance in N/mm², and

f_s = Local side friction in N/mm².

For non-homogeneous soils, the ultimate point bearing capacity may be calculated using the following relationships:

$$q_u = \frac{\frac{q_{c0} + q_{c1}}{2} + q_{c2}}{2}$$

where

q_u = Ultimate point bearing capacity,

q_{c0} = Average static cone resistance over a depth of 2 *d* below the base level of the pile,

q_{c1} = Minimum static cone resistance over the same 2 *d* below the pile tip,

q_{c2} = Average of the minimum cone resistance values in the diagram over a height of 8 *d* above the base level of the pile, and

d = Diameter of the pile base or the equivalent diameter for a non-circular cross-section.

G-3.1 The correlation between standard penetration test value *N* and static point resistance q_c (in N/mm²) given below may be used for working out the shaft resistance and skin friction of piles:

Soil type	q_c/N
Clays, silts, sandy silts and slightly cohesive silt-sand mixtures	0.2
Clean fine to medium sands and slightly silty sands	0.3-0.4
Coarse sands and sands with little gravel	0.5-0.6
Sandy gravels and gravel	0.8-0.10

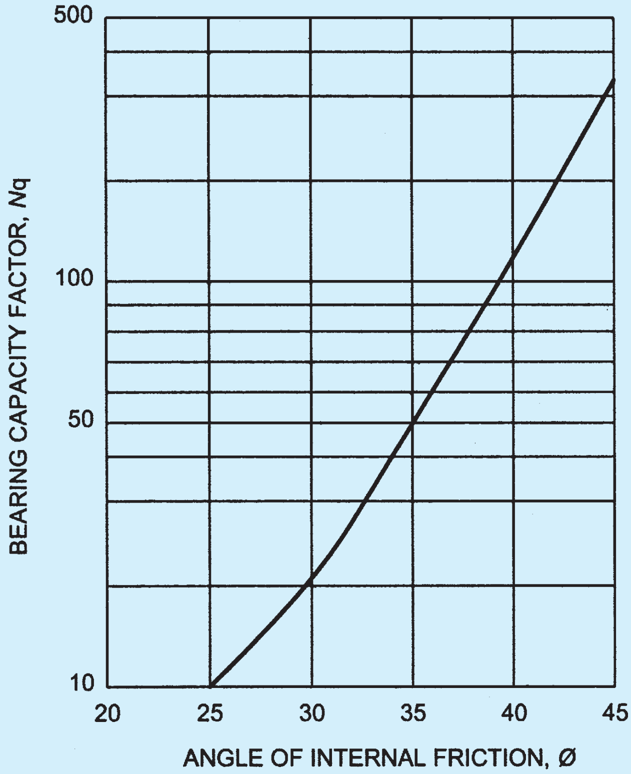


FIG. 11 BEARING CAPACITY FACTOR, N_q FOR BORED PILES

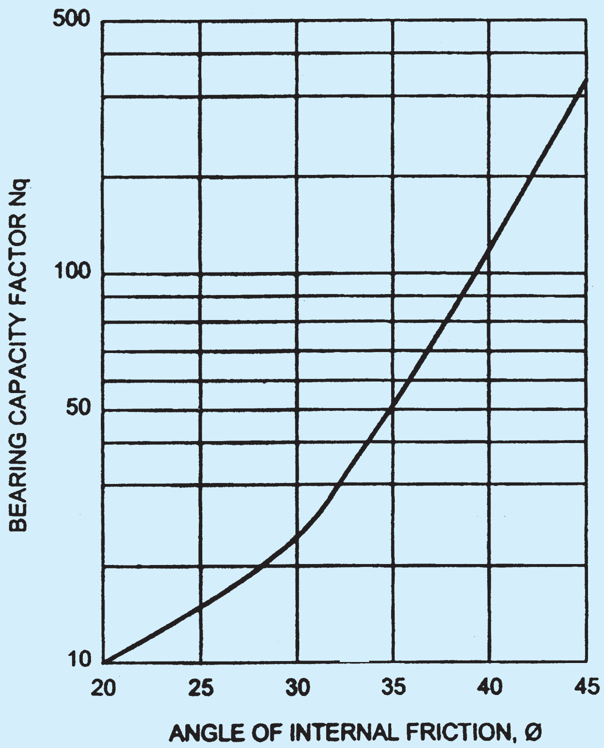


FIG. 12 BEARING CAPACITY FACTOR, N_q FOR DRIVEN PILES

ANNEX H

(*Clauses* 8.2.3.3 *and* 11.2.4)

DETERMINATION OF DEPTH OF FIXITY, LATERAL DEFLECTION AND MAXIMUM MOMENT OF LATERALLY LOADED PILES

H-1 DETERMINATION OF LATERAL DEFLECTION AT THE PILE HEAD AND DEPTH OF FIXITY

H-1.1 The long flexible pile, fully or partially embedded, is treated as a cantilever fixed at some depth below the ground level (*see* Fig. 13).

H-1.2 Determine the depth of fixity and hence the equivalent length of the cantilever using the plots given in Fig. 13.

where

$$T = 5\sqrt{\frac{EI}{K_1}} \quad \text{and} \quad R = 4\sqrt{\frac{EI}{K_2}}$$

(K_1 and K_2 are constants given in Tables 8 and 9, E is the Young's modulus of the pile material in kN/m^2 and I is the moment of inertia of the pile cross-section in m^4).

NOTE — Figure 12 is valid for long flexible piles where the embedded length L_e is $\geq 4R$ or $4T$.

Table 8 Values of Constant K_1 (kN/m^3)

(*Clause* H-1.2)

Type of Soil	Value	
	Dry	Submerged
(1)	(2)	(3)
Loose sand	2 600	1 460
Medium sand	7 750	5 250
Dense sand	20 750	12 450
Very loose sand under repeated loading or normally loading clays	—	400

Table 9 Values of Constant K_1 (kN/m^3)

(*Clause* H-1.2)

Unconfined Compressive Strength kN/m^2	Value
(1)	(2)
20 to 40	7.75×10^4
100 to 200	48.80×10^4
200 to 400	97.75×10^4
More than 400	195.50×10^4

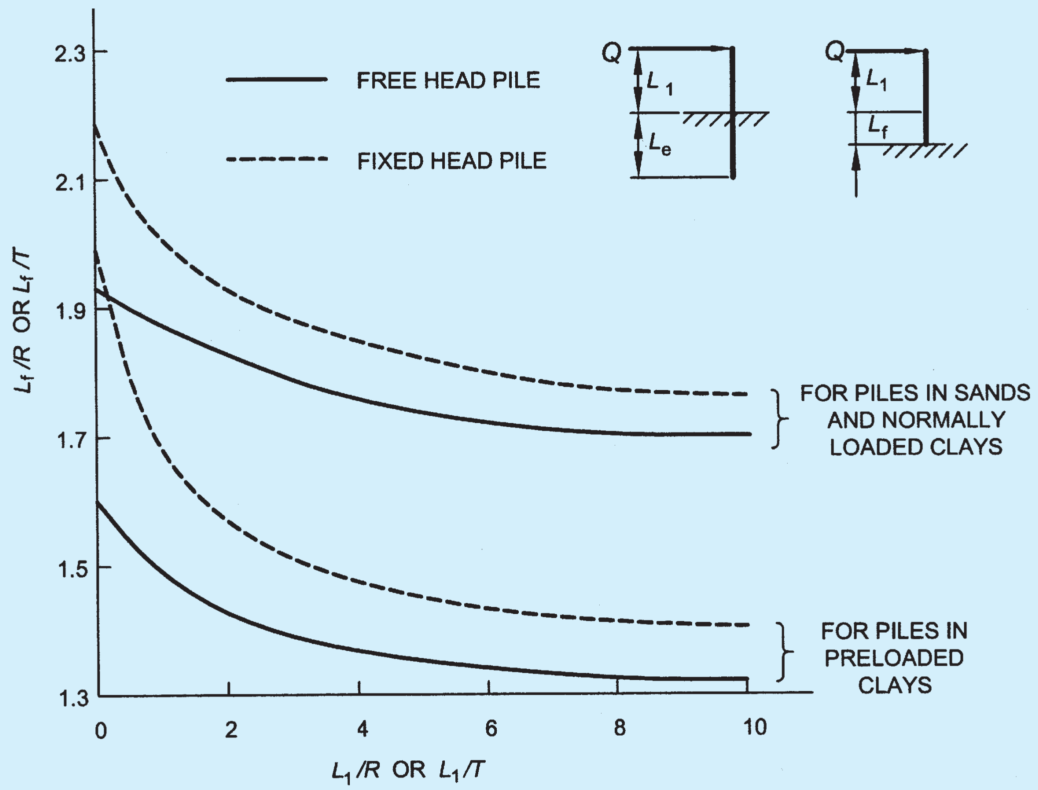


FIG. 13 DETERMINATION OF DEPTH FIXITY

H-1.3 Knowing the length of the equivalent cantilever the pile head deflection (Y) shall be computed using the following equations:

$$Y(\text{in } m) = \frac{Q\,(L_1 + L_f)^3}{3\,EI} \quad \text{...... for free head pile}$$

$$= \frac{Q\,(L_1 + L_f)^3}{12\,EI} \quad \text{...... for fixed head pile}$$

where Q is the lateral load in kN.

H-2 DETERMINATION OF MAXIMUM MOMENT IN THE PILE

H-2.1 The fixed end moment (M_f) of the equivalent cantilever is higher than the actual maximum moment (M) of the pile. The actual maximum moment is obtained by multiplying the fixed end moment of the equivalent cantilever by a reduction factor, m given in Fig. 14. The fixed end moment of the equivalent cantilever is given by:

$$M_f = Q\,(L_1 + L_f) \quad \text{...... for free head pile}$$

$$= \frac{Q\,(L_1 + L_f)^3}{12\,EI} \quad \text{...... for fixed head pile}$$

The actual maximum moment (M) = m (M_F).

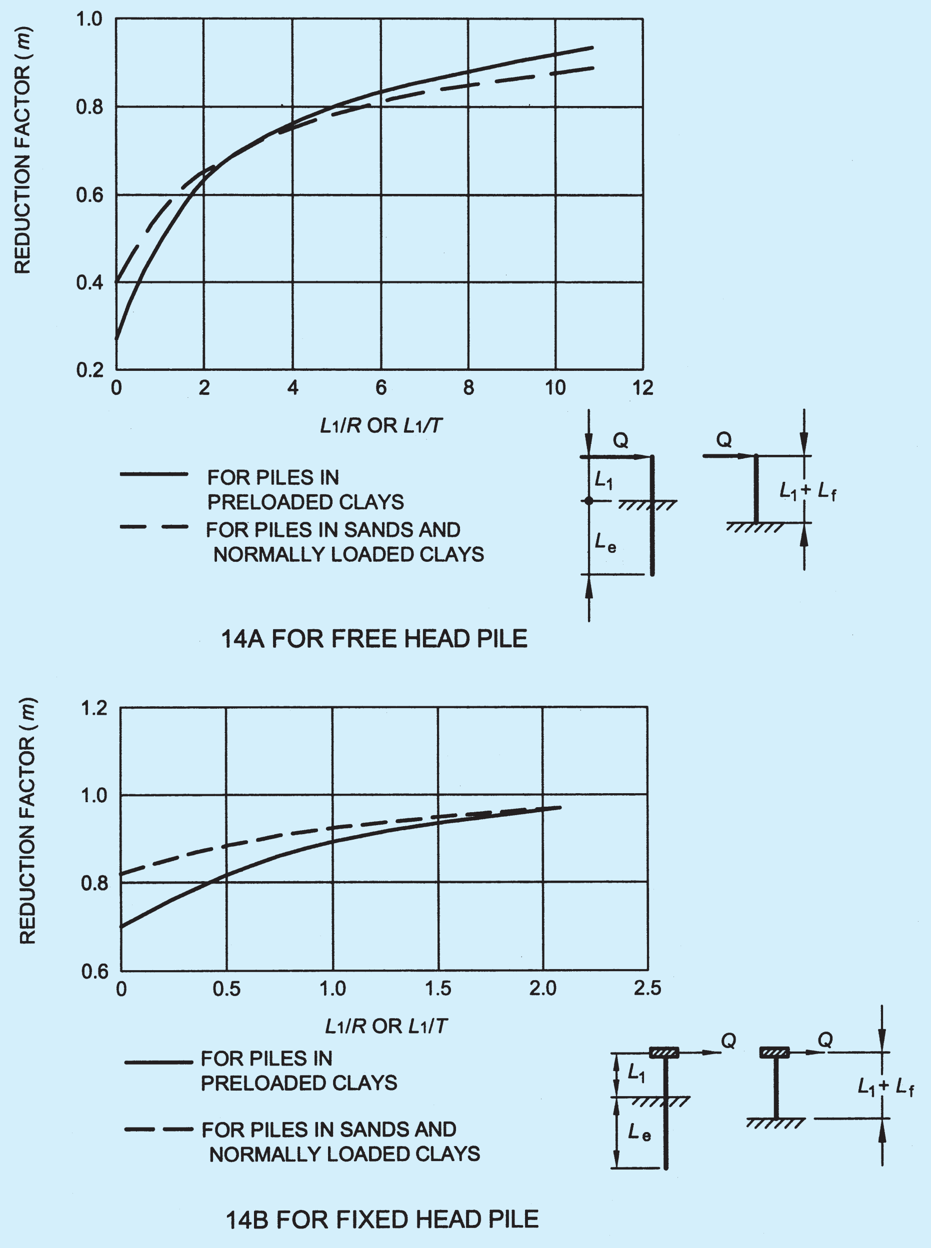


FIG. 14 DETERMINATION OF REDUCTION FACTORS FOR COMPUTATION OF MAXIMUM MOMENT IN PILE

ANNEX J

(*Clause* 11.2.2)

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 clayey 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

Q_u = Ultimate bearing capacity of pile in kN;

A_p = Cross-sectional area of the pile stem at the toe level in m^2;

N_c = Bearing capacity factor, usually taken as 9;

C_p = Cohesion of the soil around toe in kN/m^2;

A_a = $(\pi/4)(D_u^2 - D^2)$, where D_u and D are the under-reamed and stem diameter, respectively in m;

C_a = Average cohesion of the soil along the pile stem in kN/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;

C'_a = Average cohesion of the soil around the under-reamed bulbs; and

α = Reduction factor (usually taken 0.5 for clays).

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*

$$Q_u = A_p(\tfrac{1}{2} D \gamma N_\gamma + \gamma d_f N_q) + A_a(\tfrac{1}{2} D_u n \gamma N_\gamma)$$

$$+ \gamma N_q \sum_{r=1}^{r=n} d_r + \tfrac{1}{2}\pi D \gamma K \tan\delta\,(d_1^2 + d_1^2 + d_n^2)$$

where

A_p = $\pi D^2/4$, where D is stem diameter in m;

A_a = $\pi/4\,(D_u^2 - D^2)$ where D_u is the under-reamed bulb diameter in m;

n = Number of under-reamed bulbs;

γ = Average unit weight of soil (submerged unit weight in strata below water table) in kN/m^3;

N_γ, N_q = Bearing capacity factors, depending upon the angle of internal friction;

d_r = Depth of the centre of different under-reamed bulbs below ground level in m;

d_f = Total depth of pile below ground level in m;

K = Earth pressure coefficient (usually taken as 1.75 for sandy soils);

δ = Angle of wall friction (may be taken as equal to the angle of internal friction Ø);

d_1 = Depth of the centre of the first under-reamed bulb in m; and

d_n = Depth of the centre of the last under-reamed bulb in m.

NOTES

1 For uplift bearing on pile tip, A_p will not occur.

2 N_γ will be as specified in good practice [6-2(7)] and N_q will be taken from Fig. 11.

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_γ, 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.

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 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 by the Authority as a guide in conformance with the requirements of the referred clauses in the Code.

	IS No.	*Title*
(1)	1892 : 1979	Code of practice for subsurface investigation for foundation (*first revision*)
	2131 : 1981	Method of standard penetration test for soils (*first revision*)
	2132 : 1986	Code of practice for thin walled tube sampling of soils (*second revision*)
	4434 : 1978	Code of practice for *in-situ* vane shear test for soils (*first revision*)
	4968	Method for sub-surface 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*)
	8763 : 1978	Guide for undisturbed sampling of sands and sandy soils
	9214 : 1979	Method for determination of modulus of subgrade reaction (*k*-value) of soils in the field
(2)	10042 : 1981	Code of practice for site-investigations for foundation in gravel boulder deposits
(3)	13365 (Part 1) : 1998	Guidelines for quantitative classification systems of rock mass: Part 1 RMR for predicting of engineering properties
(4)	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 limits (*second revision*)
	(Part 10) : 1991	Determination of unconfined compressive strength (*second 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*)
	(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
(5)	1498 : 1970	Classification and identification of soils for general engineering purposes (*first revision*)
(6)	401 : 2001	Code of practice for preservation of timber (*fourth revision*)
(7)	6403 : 1981	Code of practice for determination of bearing capacity of shallow foundations (*first revision*)
(8)	1888 : 1982	Method of load tests on soils (*second revision*)
(9)	2131 : 1981	Method for standard penetration test for soils (*first revision*)

IS No.	*Title*
(10) 8009 (Part 1) : 1976	Code of practice for calculation of settlement of foundations: Part 1 Shallow foundations subjected to symmetrical static vertical loads
(11) 1904 : 1986	Code of practice for design and construction of foundations in soils: General requirements (*third revision*)
(12) 3764 : 1992	Code of safety for excavation work (*first revision*)
(13) 456 : 2000	Code of practice for plain and reinforced concrete (*fourth revision*)
(14) 1905 : 1987	Code of practice for structural use of unreinforced masonry (*third revision*)
(15) 1080 : 1985	Code of practice for design and construction of shallow foundations in soils (other than raft, ring and shell) (*second revision*)
(16) 11089 : 1984	Code of practice for design and construction of ring foundations
(17) 2911	Code of practice for design and construction of pile foundations:
(Part 1/Sec 1): 1979	Concrete piles, Section 1 Driven cast *in-situ* concrete piles (*first revision*)
(Part 1/Sec 2) : 1979	Concrete piles, Section 2 Bored cast *in-situ* piles (*first revision*)
(18) (Part 4) : 1985	Load test on piles (*first revision*)
(19) 14893 : 2001	Guidelines for non-destructive integrity testing of piles
(20) 2911	Code of practice for design and construction of pile foundations:
(Part 1/Sec 3) : 1979	Concrete piles, Section 3 Driven precast concrete piles (*first revision*)
(21) (Part 1/Sec 4) : 1979	Concrete piles, Section 4 Bored precast concrete piles
(22) (Part 3) : 1980	Under-reamed pile foundation (*first revision*)
(23) (Part 2) : 1980	Timber piles (*first revision*)
(24) 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 jetter) (*first revision*)
13301 : 1992	Guidelines for vibration isolation for machine foundations
9556 : 1980	Code of practice for design and construction of diaphragm walls
(25) 13094 : 1992	Guidelines for selection of ground improvement techniques for foundation in weak soils
(26) 13162 (Part 2) : 1991	Geotextiles — Methods of test: Part 2 Determination of resistance to exposure of ultra-violet light and water (Xenon arc type apparatus)
13321 (Part 1) : 1992	Glossary of terms for geo-synthetics: Part 1 Terms used in materials and properties
13325 : 1992	Method of test for the determination of tensile properties of extruded polymer geogrids using the wide strip
13326 (Part 1) : 1992	Method of test for the evaluation of interface friction between geosynthetics and soil: Part 1 Modified direct shear technique
14293 : 1995	Geotextiles — Method of test for trapezoid tearing strength
14294 : 1995	Geotextiles — Method for determination of apparent opening size by dry sieving technique
14324 : 1995	Geotextiles — Methods of test for determination of water permeability-permittivity

IS No.	*Title*
14706 : 1999	Geotextiles — Sampling and preparation of test specimens
14714 : 1999	Geotextiles — Determination of abrasion resistance
14715 : 2000	Woven jute geotextiles — Specification
14716 : 1999	Geotextiles — Determination of mass per unit area
14739 : 1999	Geotextiles — Methods for determination of creep
14986 : 2001	Guidelines for application of jute geo-grid for rain water erosion control in road and railway embankments and hill slopes
15060 : 2001	Geotextiles — Tensile test for joints/seams by wide width method
(27) 9214 : 1979	Method of determination of subgrade reaction (*K*-value) of soils in the field

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 3 Timber and Bamboo: 3A Timber

BUREAU OF INDIAN STANDARDS

CONTENTS

National Building Code Sectional Committee, CED 46

FOREWORD

This Section deals with the structural design aspect of timber structures. In this section, the various Species of Indian timber, classified into three groups depending on the structural properties influencing the design, most are included.

In the previous version of the Code, timber was covered under Section 3 of Part 6 under the title 'Wood', which did not cover bamboo. Now this Section 3 has been enlarged as Section 3 Timber and Bamboo, which has been sub-divided into sub-section 3A Timber and sub-section 3B Bamboo. This sub-section pertains to 3A Timber.

This Section was first published in 1970 which was subsequently revised in 1983. In the first revision provisions of this Section were updated and design of nailed laminated timber beams were included and information on bolted construction joints was added. As a result of experience gained in implementation of 1983 version of this Code and feedback received as well as formulation of new standards in the field and revision of some of the existing standards, a need to revise this Section 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) A number of terminologies related to timber for structural purpose have been added.
b) Strength data of additional species of timber have been included.
c) Requirements for structural timber and preferred cut sizes thereof have been modified.
d) Requirements for glued laminated construction and finger joints have been introduced.
e) Requirements for laminated veneer lumber have been introduced.
f) Brief details have been included for structural sandwiches, glued laminated beams, lamella roofing, nail and screw holding power of timber, structural use of plywood and trussed rafter; these are proposed to be further elaborated in future revisions of this Section.
g) Guidelines for protection against termite attack in buildings have been added.
h) Reference to all the concerned Indian Standards have been updated.

In the present day context of dwindling forest resources, all efforts are being made to effect judicious use of timber. In this context, the Indian Standards now permit use of plantation timbers including certain fast growing species and suitable guidelines in terms of their seasoning, sawing, treatment, etc have been made available. In the same way, use of finger jointing and glued laminated timber is important and standardization on the same is desirable and is under due consideration. However, in the absence of detailed Indian Standard Specifications and Codes of practice in these areas at present, general details on the same have been incorporated in the revision of this part.

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

IS No.	*Title*
399 : 1963	Classification of commercial timbers and their zonal distribution (*revised*)
883 : 1994	Code of practice for design of structural timber in building (*fourth revision*)
1150 : 2000	Trade names and abbreviated symbols for timber species (*third revision*)
2366 : 1983	Code of practice for nail-jointed timber construction (*first revision*)
4891 : 1988	Specification for preferred cut sizes of structural timber (*first revision*)
4983 : 1968	Code of practice for design and construction of nailed laminated timber beams
11096 : 1984	Code of practice for design and construction of bolt-jointed timber construction
14616 : 1999	Specification for laminated veneer lumber

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.

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 3 Timber and Bamboo: 3A Timber

1 SCOPE

1.1 This Section relates to the use of structural timber in structures or elements of structures connected together 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. Nor is the classification of timber into strength groups to 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

2.0 For the purpose of this Section, the following definitions and those in accordance with accepted standard [6-3A(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 *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.

2.1.4 *Duration of Load* — Period during which a member or a complete structure is stressed as a consequence of the loads applied.

2.1.5 *Edge Distance* — The distance measured perpendicular to grain from the centre of the connector to the edge of the member.

2.1.6 *End Distance* — The distance measured parallel to grain of the member from the centre of the connector to the closest end of timber.

2.1.7 *Finger Joint* — Joint produced by connecting timber members end-to-end by cutting profiles (tapered projections) in the form of V-shaped grooves to the ends of timber planks or scantlings to be joined, glueing the interfaces and then mating the two ends together under pressure.

2.1.8 *Fundamental or Ultimate Stress* — The stress which is determined on small clear specimen of timber, in accordance with good practice [6-3A(2)]; and does not take into account the effect of naturally occurring characteristics and other factors.

2.1.9 *Inside Location* — Position in buildings in which timber remains continuously dry or protected from weather.

2.1.10 *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.

2.1.11 *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.

2.1.12 *Location* — A term generally referred to as exact place where a timber is used in building.

2.1.13 *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.

2.1.14 *Permissible Stress* — Stress obtained by applying factor of safety to the ultimate stress.

2.1.15 *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.

2.1.16 *Spaced Column* — Two column sections adequately connected together by glue, bolts, screws or otherwise.

2.1.17 *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.

2.1.18 *Structure, Temporary* — Structures which are erected for a short period, such as hutments at project sites, for rehabilitation, temporary defence constructions, exhibition structures, etc.

2.1.19 *Structural Element* — The component timber members and joints which make up a resulting structural assembly.

2.1.20 *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.

2.1.21 *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.

2.1.22 *Termite* — An insect of the order *Isoptera* which may burrow in the wood or wood products of a building for food or shelter.

2.1.23 *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. Syn. 'Integrown 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.

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

3.1 For the purpose of this Section, the following letter symbols shall have the meaning indicated against each:

- a = Projected area of bolt in main member ($t' \times d_3$), 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 = Length of the notch measured along the beam span from the inner edge of the support to the farthest edge of the notch, mm
E = Modulus of elasticity in bending, N/mm^2
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^{Θ} = 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
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
K_3, K_4, K_5, and K_6 = Form factors
K_7 = Modification factor for bearing stress
K_8 = Constant equal to $0.584\sqrt{\dfrac{E}{f_{cp}}}$
K_9 = Constant equal to $\dfrac{\pi}{2}\sqrt{\dfrac{U\ E}{5q\ f_{cp}}}$
K_{10} = Constant equal to $0.584\sqrt{\dfrac{2.5E}{f_{cp}}}$
L = Span of a beam or truss, mm
M = Maximum bending moment in beam, N/mm^2
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 = Statical 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
γ = A 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.

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 and tension (f_b).

The characteristics of these groups are as given below:

Group A — E above 12.6 × 10^3 N/mm^2 and f_b above 18.0 N/mm^2.

Group B — E above 9.8 × 10^3 N/mm^2 and up to 12.6 × 10^3 N/mm^2 and f_b above 12.0 N/mm^2 and up to 18.0 N/mm^2.

Group C — E above 5.6×10^3 N/mm^2 and up to 9.8×10^3 N/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 [6-3A(3)].

4.2 The general characteristics like durability and treatability 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, Dehra Dun.

4.3 The permissible lateral strength (in double shear) of mild steel wire 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.

4.5 Sawn Timber

4.5.1 *Sizes*

Preferred cut sizes of timber for use in structural components shall be as given in Tables 5 to 7.

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
c) Grade II

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 good practice [6-3A(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.

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 load-bearing temporary structures or semi-structural components at construction site

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.

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 *and* 6.5.8.3.1 (b)]

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 in N/mm^2 for Grade I											Preservative Characters		
Botanical Name	Trade Name				Bending and Tension Along Grain, Extreme Fibre Stress			Shear all Locations		Compression Parallel to Grain			Compression Perpendicular to Grain			†Durability Class	‡Treat-ability Grade	§Refrac-teriness to All Seasoning
			kg/m^3	N/mm^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)
GROUP A																		
Acacia catecthu	Khair (KHA)	U.P.	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
Acacia chundra	Red kutch	M.P.	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
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
Bruguiera spp.	Bruguiera (BSV) (Mangrove)	Andmans	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	—	—
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
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	—	—	—
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
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
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
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
Mimusops littoralis	Bullet-wood (BUL)	S. 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
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
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
Sageraea elliptica	Chooi (COC)	Andmans	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
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
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
GROUP B																		
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

Table 1 — *Continued*

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
Anogeissus latifolia	Dhaura, Axle wood (AXL) (Bakli)	U.P.	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
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
Acacia nilotica	Babul (BAB)	U.P.	797	—	—	12.9	10.3	1.4	2.1	8.9	7.9	6.4	5.2	4.0	3.3	I	b	B
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	—	—	—
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
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
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
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	—	—	—
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
Amoora spp.	Amari (AMA)	W. 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
Bucklandia populnea (Syn Exbucklandia populnea)	Pipli (PIP)	W. 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
Cassia fistula	Amaltas (AMT)	U.P.	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
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	—	—	—
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
Cassia sienea	Kasod	M.P.	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	—	—	—
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
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
Chloroxylon swielenia	Satin wood (CFI)	M.P.	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
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
Diploknema butyracea (Syn Bassia butyrance)	Hill mahua (HMA)	S. 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	—	—	—
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
Dipterocarpus grandiflorus	Gurjan (GUR)	N. 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
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
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
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
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
Diospyros pyrrhocarpus	Ebony (EBO)	N. 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
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
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
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	—	—	—

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
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	—
Eugenia jambolana	Jaman (JAM)	U.P.	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			
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
Grewia veslita	Dhaman (DHA)	W. 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
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
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
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	—	—
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
Lagerstromia parviflora	Lendi (LEN)	U.P.	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
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
Machilus macrantha	Machilus (MAC)	W. 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
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
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	—	—	—
Pterocarpus dolbergioides	Padauk (PAD)	N. 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
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	—
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
Fraxlnus macrantha	Ash (ASH)	U.P.	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
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
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	—	—
Quercus lamellosa	Oak	W. 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
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
Quercus incana	Oak	Punjab	1008	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
Quercus lineate	Oak	W. 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
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
*Shorea robusta**	Sal (SAL)	M.P.	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
Soymida fabrifuga	Rohini (ROH)	Chennai	1116	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
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	—	—	—
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
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
Terminalia bellirica	Bahera (BAH)	U.P.	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
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

Table 1 — *Continued*

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
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	—	—	—
Terminalia manii	Black-chuglam (BCH)	S. 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
Tectona grandis	Teak (TEA)	U.P.	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
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
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
Terminalia bilata	White-chuglam (WCH)	S. 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
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
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
Zanthoxylum budranga	Mullilam (MUL)	W. 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
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	—	—	—
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
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	—	—	—
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	—	—	—
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	—
GROUP C																		
Tbizia procera	White siris	U.P.	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
Artocarpus lakocha	Lakooch (LAK)	U.P.	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
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
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
*Adina cordifolia**	Haldu (HAL)	U.P.	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
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	—
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
Acacia leucophloea	Hiwar (HIW)	M.P.	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
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	—	—	—
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	—	—	—
Accer spp.	Maple (MAP)	Punjab, U.P.	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
Aegla marmalos (Syn. Intsia bijuga)	Bael (BEL)	U.P.	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
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	—	—	—
Ailanthus grandis	Gokul (GOK)	W. 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
Anogeissus pendula	Kardhai (KAH)	U.P.	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
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	—	—	—

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
Albizia lucida	—	Arunachal, A.P.	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	—	—	—
Azadirachta indica	Neem (NEE)	U.P.	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	—	—	—
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
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
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
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
Burserra serrata (Syn. Protium serratum)	Muntenga (MUR)	A.P.	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	—
Careya arbersa	Kumbi (KUM)	U.P.	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
Cedrus deodara	Deodar (DEO)	H.P.	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
Cupressus torulosa	Cypress (CYP)	U.P.	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
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
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
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
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
Chlorophora excelsa	—																	
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	—	—	—
Dalbergia latifolia	Rosewood (ROS)	M.P.	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
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
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
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
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
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
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
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	—
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	—	—	—
Garuga pinnata	Garuga (GAU)	U.P.	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
gGmeline arborea	Gamari (GAM)	U.P.	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
Gardonia latifolia	Gardenia (GAI)	M.P.	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	—	—	—
Hardwickis binata	Anjan (ANJ)	M.P.	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	—
Heloptelea integrifolia	Kanju (KAN)	U.P.	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

Table 1 — *Continued*

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
Heterrophragma rexburghii	Palang (PAL)	M.P.	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	—	—	—
Juglans spp.	Walnut (WAL)	U.P.	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
Lagerstrosmia speciosa (Syn. L. flesregihal)	Jarul (JAAR)	N. 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
Lannea grandis (Syn. L.coromandelica)	Jhingan (JHI)	U.P.	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
Leucanena leucocephala	Subabul (SUB)	U.P.	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	—	—	
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
Madhuca longifolia varlatifolia (Syn. Bassia latifolia)	Mahua (MAU)	M.P.	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
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
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
Mallotus philippinensis	Raini (RAI)	U.P.	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
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	—	—	—
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
Mitragyna pervifolia (Syn. Stephagyne pervifolia)	Kaim (KAI)	U.P.	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
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
Miliusa velutnia	Domsal (DOM)	U.P.	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	—	—
Morus alba	Mulberry (MUL)	U.P.	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
Morus serrata	Mulberry (MUL)	H.P.	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
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
Ougeinia eejeinensis (Syn. O. delbergioides)	Sandan (SAD)	M.P.	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
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
Pinus roxburghii (Syn. P. longifolia)	Chir (CHR)	U.P.	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
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
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
Parretiopsis jacquementiena	Rohu Parrotia	H.P.	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
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
Pistacia integerrima	Kikar singhi	J&K	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	—	—	—
Podocarpus nerrifolius	Thitmin (THT)	S. 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	—	—
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
Polyalthia coreoides		M.P.	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	—	—	—

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
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	—	—	—
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
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
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	—
Schleichera oleosa (Syn. S. trijuga)	Kusum (KUS)	Bihar	1032	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
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
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
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
Stereospermum suaveolans	Padri (PAD)	U.P.	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
Tactona grandis	Teak (TEA)	M.P.	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
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
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
Terminalia procera	White bombwae (WBO)	N.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
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	—	—	—
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
Toena ciliata	Toon (TOO)	U.P.	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
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
Aeculas indica	Horse chestnut (HCH)	U.P.	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
Borassus flabelsfer	Tad (Palmyra) (TAD)	A.P.	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	—	—	—
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
Eucalyptus camaldulenis	Eucalyptus	U.P.	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
Eucalyptus pilularia	Eucalyptus	T.N.	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
Eucalyptus propingus	Eucalyptus	T.N.	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
Eucalyptus saligna	Eucalyptus	U.P.	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

* 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, U.P. 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.

Table 1 — *Concluded*

† 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.

‡ 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;

Data based on strength properties at three years of age of tree.

§ Classifications based on seasoning behaviour of timber and refractoriness *w.r.t.* cracking, spliting 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.

Table 2 Permissible Lateral Strengths (in Double Shear) of Nails 3.55 mm Dia, 80 mm Long
(*Clause* 4.3)

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)
1.	*Albies pirdrow*[1)]	Fir	8	2	12
2.	*Acacia nilotica*	Babul	15	11	34
3.	*Acrocarpus fraxinifolius*	Mundani	18	9.5	19.5
4.	*Adina cordifolia*	Haldu	23.5	10	22
5.	*Albizia lebbeck*	Kokko	20	7	24
6.	*Albizia odoratissima*	Kala Siris	14	5	22
7.	*Anogeissus latifolia*	Axlewood	20	10	29
8.	*Aphanamixis polystachya*	Pitraj	19	9	19
9.	*Calophyllum spp.*[1)]	Poon	16	9	21
10.	*Canarium euphyllum*	White dhup	9	8	10.5
11.	*Castanopsis spp.*	Indian chestnut	18	10.5	23.5
12.	*Cedrus deodara*[1)]	Deodar	9	4	15
13.	*Chukrasia tabularis*	Chikrassy	24	8	27
14.	*Cinnamomum spp.*[1)]	Cinnomon	12	9	13
15.	*Cupressus torulosa*	Cypress	6	5	18
16.	*Dipterocarpus macrocarpus*	Hollong	17	7	20
17.	*Dipterocarpus spp.*	Gurjan	19	9	19
18.	*Dillenia pertagyna*	Dillenia	16.5	12	16
19.	*Diospyros melanoxylon*	Ebony	26.5	10	30.5
20.	*Eucalyptus eugenioides*	Eucalyptus	17	10	30
21.	*Grewia tilifolia*[1)]	Dhaman	13	5	24
22.	*Lagerstroemia spp.*	Jarul	24.5	21.5	22.5
23.	*Hopea parviflora*	Hopea	31.5	13	28.5
24.	*Lagerstroemia spp.*[1)]	Lendi	19	5	26
25.	*Mangifera indica*	Mango	11	9	16
26.	*Maniltoa polyandra*	Ping	26	23.5	32
27.	*Mesua ferrea*	Mesua	26	8	41
28.	*Michelia spp.*	Champ	13	9	20
29.	*Millingtonia spp.*[1)]	—	10.5	6	17
30.	*Morus alba*	Mulberry	13	10.5	22.5
31.	*Melia azedarach*	Persian lilac (bakain)	10.5	2.5	9
32.	*Ougeinia oojeinensis*	Sandan	17	11	18
33.	*Phoebe spp.*[1)]	Bonsum	12	6	13
34.	*Pinus roxburghii*[1)]	Chir	11	10	16
35.	*Pinus wallichiana*[1)]	Kail	7	3	9
36.	*Pterocarpus marsupium*	Bijasal	15	12	27
37.	*Pterocarpus dalbergiodes*	Pauduak	19	14	23
38.	*Planchonia andamanica*	Red bombwe	14	13	29
39.	*Quercus spp.*	Oak	11	11	27
40.	*Scheichera cleosa*	Kusum	23	16	40
41.	*Shorea roubusta*	Sal (M.P.)	23	15.5	19.5
42.	*Shorea robusta*	Sal	10	5	19
43.	*Stereospenmum*	Padriwood	16	8	19.5
44.	*Syzygium spp.*	Jamum	15	12	25
45.	*Tectona grandis*	Teak	14	8	13
46.	*Terminalia Bellirica*	Bahera	10	10	14
47.	*Terminalia biolata*	White chuglam	18	9	21
48.	*Terminalia procera*	Badam	18	10.5	20
49.	*Terminalia manii*[1)]	Black chuglam	23	10	33
50.	*Terminalia myriocarpa*	Hollock	13	10	19
51.	*Terminalia alata*	Sain	16	16	29
52.	*Toona spp.*	Toona	10	8	21
53.	*Xylia xylacarpa*	Irul	23	6	33
54.	*Toona ciliata*	Toon	16	9	21

NOTES

1 Nails of 3.55 mm diameter are most commonly used. The above values can also be used for 4 mm diameter 100 mm long nails.

2 The values in N are approximate converted values from kgf. For exact conversion the value is 1 kgf = 9.806 65 N.

[1)] Species requiring no preboring for nail penetration.

Table 3 Permissible Lateral Strengths (in Double Shear) of Nails 5.00 mm Dia, 125 mm and 150 mm Long

(*Clause* 4.3)

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)
1.	*Abies pindrow*[1)]	Fir	16.5	4.5	21
2.	*Acacia catechu*	Khair	42	25	71.5
3.	*Acacia nilotica*[1)]	Babul	27	13.5	53
4.	*Alibizia procera*	Safed siris	35	18	—
5.	*Alibizia odoratissima*[1)]	Kala siris	27.5	17.5	45
6.	*Alstonia scholaris*	Chatian	9.5	5.5	27
7.	*Anogeissus latifolia*	Axlewood	22.5	13	46.5
8.	*Cupressus torulosa*	Cypress	20	7	27
9.	*Cullenia rosayroana*	Karani	11	9.5	30
10.	*Dalbergia sissoo*	Sissoo	17	15	43
11.	*Diptrocarous spp.*	Gurjan	19.5	9.5	33
12.	*Hardwickia binata*	Anjan	32	19	59
13.	*Hopea perviflora*	Hopea	60.5	25	61.5
14	*Holoptelea integrifolia*	Kanju	18	12.5	37.5
15.	*Mangifera indica*[1)]	Mango	22.5	15	32
16.	*Mesua ferrea*	Mesua	24	15.5	57.5
17.	*Michelia champaca*[1)]	Champ	26	12.5	39
18.	*Pterocarpus marsupium*	Bijasal	20.5	15	43
19.	*Pinus roxburghii*[1)]	Chir	9	6	24
20.	*Shorea robusta (U.P.)*	Sal	19.5	17	37
21.	*Shorea robusta*	Sal	30.5	20	41
22.	*Schleichera cleosa*	Kusum	15	14	55
23.	*Stereospimum personatum*	Padriwood	22	8	34
24.	*Syzygium cumini*	Jamum	18	14.5	38.5
25.	*Terminalia myriocarpa*	Hollock	27.5	9	41
26.	*Tectona grandis*	Teak	28	13	30
27.	*Hopea utilis*	Karung kangoo	31	10	58
28.	*Phoebe spp*[1)]	Bonsum	20	7.5	30

NOTES

1 Nails of 5.00 mm diameter are most commonly used.

2 The values in N are approximate converted values from kgf. For exact conversion the value is 1 kgf = 9.806 65 N.

[1)] Species requires no preboring for nail penetration.

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:

Zone I — Average annual relative humidity less than 40 percent.
Zone II — Average annual relative humidity 40 to 50 percent.
Zone III — Average annual relative humidity 50 to 67 percent.
Zone IV — Average annual relative humidity more than 67 percent.

For detailed zonal classification, tolerances, etc reference may be made to good practice [6-3A(4)].

Table 5 Preferred Cut Sizes of Structural Timbers for Roof Trusses (Span from 3 m to 20 m)

(*Clause* 4.5.1)

Thickness mm	Width mm							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
20	40	50	60	80	*100*	—	—	—
25	40	50	60	80	100	120	140	160
30	40	50	60	80	100	120	140	160
35	—	—	60	80	100	120	140	160
40	—	—	60	80	100	120	140	160
50	—	—	60	80	100	120	140	160
60	—	—	—	80	100	120	140	160
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, 1.5, 2, 2.5 and 3 m.

Table 6 Preferred Cut Sizes of Structural Timber for Roof Purlins, Rafters, Floor Beams, Etc

(*Clause* 4.5.1)

Thickness mm	Width mm						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
50	80	100	120	140	—	—	—
60	80	100	120	140	160	—	—
80	—	100	120	140	160	—	—
100	—	—	—	140	160	180	200

NOTE — Preferred lengths of timber: 1.5, 2, 2.5 and 3 m.

Table 7 Preferred Cut Sizes of Structural Timbers for Partition Framing and Covering, and for Centering

(*Clause* 4.5.1)

Thickness mm	Width mm								
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
10	40	50	60	80	—	—	—	—	—
15	40	50	60	80	100	—	—	—	—
20	40	50	60	80	100	120	160	200	—
25	40	50	60	80	100	120	160	200	240
30	40	50	60	80	100	120	160	200	240
40	40	—	60	80	100	120	160	200	240
50	—	50	—	80	100	120	160	200	240
60	—	—	60	80	100	120	160	200	240
80	—	—	—	80	100	120	160	200	240

NOTE — Preferred lengths of timber: 0.5, 1, 1.5, 2, 2.5 and 3 m.

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:			

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	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)
75	10	10	19	19	29	30
100	13	13	25	25	38	39
150	19	19	38	38	57	57
200	22	25	44	50	66	75
250	25	29	50	57	66	87
300	27	38	54	75	81	114
350	29	41	57	81	87	123
400	32	44	63	87	96	132
450	33	47	66	93	99	141
500	35	50	69	100	105	150
550	36	52	72	103	108	156
600	38	53	75	106	114	159

v) Checks and shakes:

Width of the Face of the Timber Max	*Permissible Depth Max*	*Permissible Depth Max*	*Permissible Depth Max*
(1)	(2)	(3)	(4)
75	12	25	36
100	18	35	54
150	25	50	75
200	33	65	99
250	40	81	120
300	50	100	150
350	57	115	171
400	66	131	198
450	76	150	225
500	83	165	249
550	90	181	270
600	100	200	300

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.

5 PERMISSIBLE STRESSES

5.1 Fundamental stress values of different groups of timber are determined on small clear specimen according to good practice [6-3A(1)]. These values are then divided by the appropriate factors of safety to obtain the permissible stresses. In these values, are then applied, appropriate safety factors given in the relevant table of the accepted standard [6-3A(5)] to obtain the permissible stress.

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 [6-3A(6)], and

c) The loads should be continuous and permanent and not of impact type.

Table 9 Minimum Permissible Stress Limits (N/mm²) 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)] Horizontal	All locations	1.05	0.64	0.49
	Along grain	All locations	15	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/mm²)	All locations and grade	12.6	9.8	5.6

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.

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.

Table 10 Modifications Factor K_1 to Allow for Change in Slope of Grain

(*Clause* 5.4.1)

Slope	Modification Factor K_1	
	Strength of Beams, Joists and Ties	Strength of Posts or Columns
(1)	(2)	(3)
1 in 10	0.80	0.74
1 in 12	0.90	0.82
1 in 14	0.98	0.87
1 in 15 and flatter	1.00	1.00

NOTE — For intermediary slopes of grains, values of modification factor may be obtained by interpolation.

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 Modifications Factor K_2, for Change in Duration of Loading

(*Clause* 5.4.2)

Duration of Loading	Modification Factor K_2
(1)	(2)
Continuous (Normal)	1.0
Two months	1.15
Seven days	1.25
Wind and earthquake	1.33
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.

[*Explanation*: In any structural timber design for dead loads, snow loads and wind or earthquake forces, members may be designed on the basis of total of stresses due to dead, snow and wind loads using K_2 = 1.33, factor for the permissible stress (of 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 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 6 'Structural Design, Section 1 Loads, Forces and Effects', without exceeding the limits of relevant stresses specified in this Section.

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 a plane or a series of connected planes transversely through the members.

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 6 'Structural Design, Section 1 Loads, Forces and Effects'.

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:

$$f_{\mathrm{ab}} = \frac{M}{Z} \leq f_{\mathrm{b}}$$

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:

$$K_4 = 0.8 + 0.8y\left(\frac{D^2 + 89\,400 - 1}{D^2 + 55\,000}\right)$$

where

$$y = p_1^2\ (6 - 8\,p_1 + 3p_1^2)\ (1 - q_1) + q_1$$

c) *Solid Circular Cross-Sections* — For solid circular cross sections the form factor K_5 shall be taken as 1.18.

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.

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:

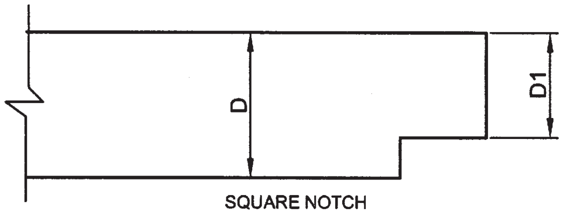
$$H = \frac{VQ}{Ib}$$

b) For rectangular beams:

$$H = \frac{3V}{2bD}$$

c) For notched beams, with tension notch at supports (*see* **6.5.7.3**):

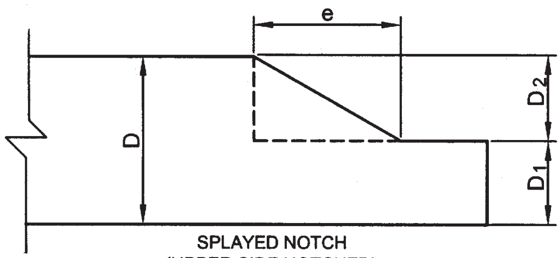
$$H = \frac{3VD}{2bD_1^2}$$



SQUARE NOTCH
(BOTTOM SIDE NOTCHED)

d) For notched at upper (compression) face, where $e > D$:

$$H = \frac{3V}{2bD_1}$$



SPLAYED NOTCH
(UPPER SIDE NOTCHED)

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:

$$V = \frac{10\,C(I - x)\,(x/D)^2}{9I[2 + (x/D)^2]}$$

and for uniformly distributed loads,

$$V = \frac{W}{2}\left(1 - \frac{2D}{I}\right)$$

After arriving at the value of V, its value will be substituted in the formula:

$$H = \frac{V\,Q}{I\,b}$$

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

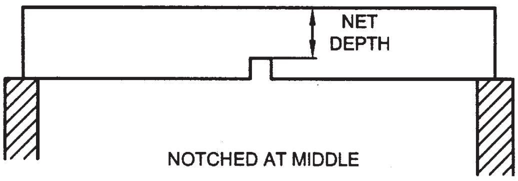
[*Clause* 6.5.7.3 (c)]

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)
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
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.



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. 1, 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:

$$f_c\theta = \frac{f_{cp} \times f_{cn}}{f_{cp}\sin^2\theta + f_{cn}\cos^2\theta}$$

6.5.9 *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.

6.5.9.1 Usual formula for deflection shall apply:

$$\delta = \frac{KWL^3}{EI}$$ (ignoring deflection due to shear strain)

K-values =	1/3	for cantilevers with load at free end,
	1/8	for cantilevers with uniformly distributed load,

Table 13 Modification Factor K_7 for Bearing Stresses

(*Clause* 6.5.8.3.1)

Length of Bearing, in mm	15	25	40	50	75	100	150 or more
Modification Factor, K_7	1.67	1.40	1.25	1.20	1.13	1.10	1.00

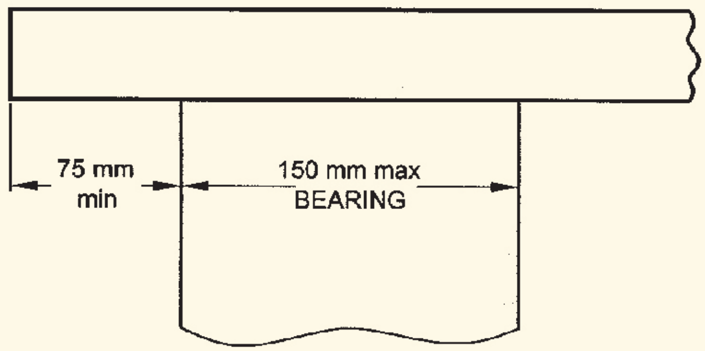


FIG. 1 POSITION OF END BEARINGS

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.

6.5.9.2 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.

6.5.9.3 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:

$$f_c = \frac{0.329\,E}{(S/d)^2}$$

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.

6.6.1.6 For determining *S/d* ratio of a tapered column, 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:

$$f_c = qf_{cp}$$

6.6.2.3 For intermediate columns, the permissible compressive stress shall be obtained using the following formula:

$$f_c = qf_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{k_9\sqrt{d_1^2 + d_1^2}}\right)^4\right]$$

6.6.2.4 For long columns, the permissible compressive stress shall be calculated by using the following formula:

$$f_c = \frac{0.329\,UE}{\left(\frac{S}{\sqrt{d_1^2 + d_2^2}}\right)^2}$$

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:

t mm	*U*	*q*
25	0.80	1.00
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.

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 formula shall be:

$$f_c = \frac{0.329\,E \times 2.5}{(S/d)^2}$$

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 Structural Members Subject to Bending and Axial Stresses

6.7.1 Structural members subjected both to bending and axial compression shall be designed to comply with the following formula:

$$\frac{f_{ac}}{f_c} + \frac{f_{ab}}{f_b} \text{ is not greater than 1.}$$

6.7.2 Structural members subjected both to bending and axial tension shall be designed to comply with the following formula:

$$\frac{f_{at}}{f_t} + \frac{f_{ab}}{f_b} \text{ is not greater than 1.}$$

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.

7.2 Dimensions of Members

7.2.1 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.

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 5 'Building Materials'. 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 spacings 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 *Lenthening Joints*

The requirement of spacing of nails in a lengthening joint shall be as follows (*see also* Fig. 2):

Sl No.	*Spacing of Nails*	*Type of Stress in the Joint*	*Requirement, Min*
(1)	(2)	(3)	(4)
i)	End distance	Tension	12 *n*
		Compression	10 *n*
ii)	In direction of grain	Tension	10 *n*
		Compression	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.

7.5.2 *Node Joints*

The requirement for spacing of nails in node joints shall be as specified in Fig. 3 where the members are at right angle and as in Fig. 4 where the members are inclined to one another at angles other than 90° and subjected to either pure compression or pure tension.

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.

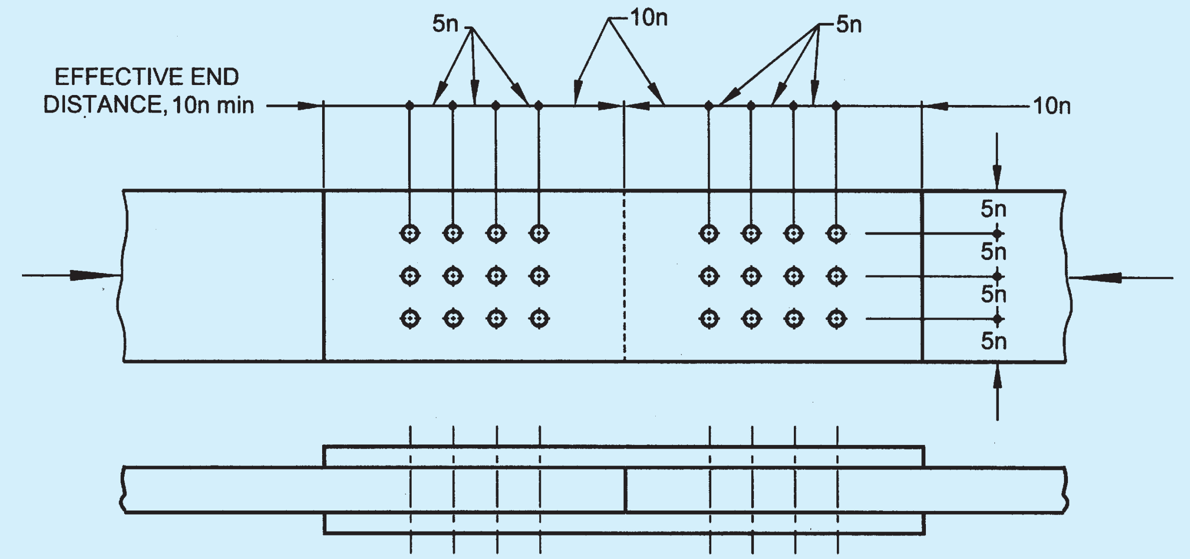
7.7.4 For a rigid joint, a minimum of 2 nails for nodal joints and 4 nails for lengthening joint shall be driven.

7.7.5 Two nails in a horizontal row are better than using the same number of nails in a vertical row.

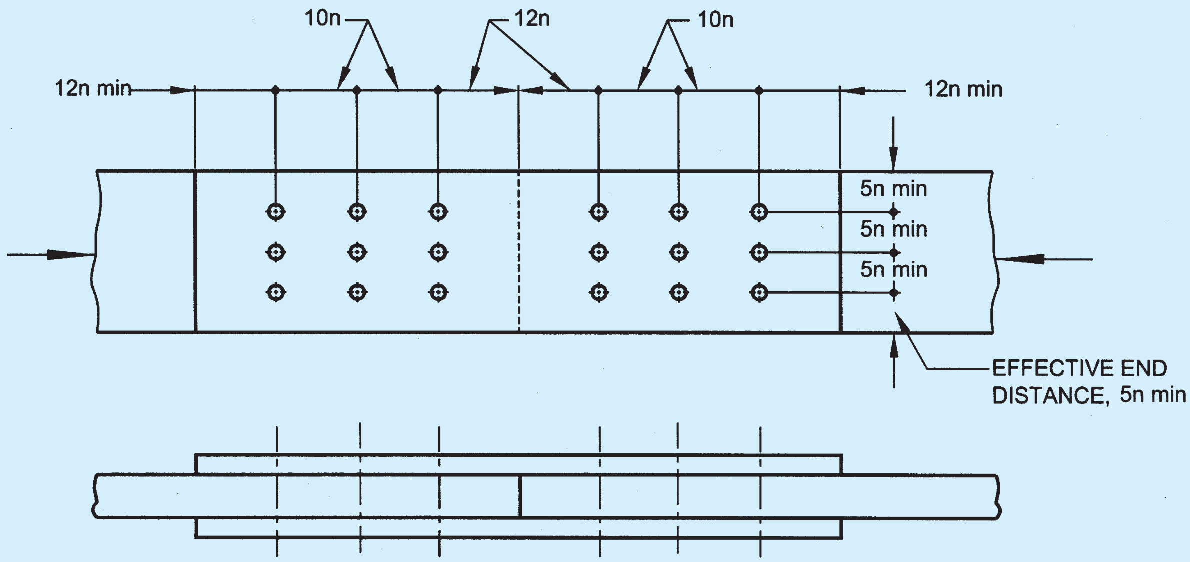
7.8 Special Consideration in Nail-Jointed Truss Construction

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.



2A MONOCHORD TYPE BUTT JOINT SUBJECT TO COMPRESSION



n = SHANK DIAMETER OF NAIL

2B MONOCHORD TYPE BUTT JOINT SUBJECT TO TENSION

FIG. 2 SPACING OF NAILS IN A LENGHTHENING JOINT — *Continued*

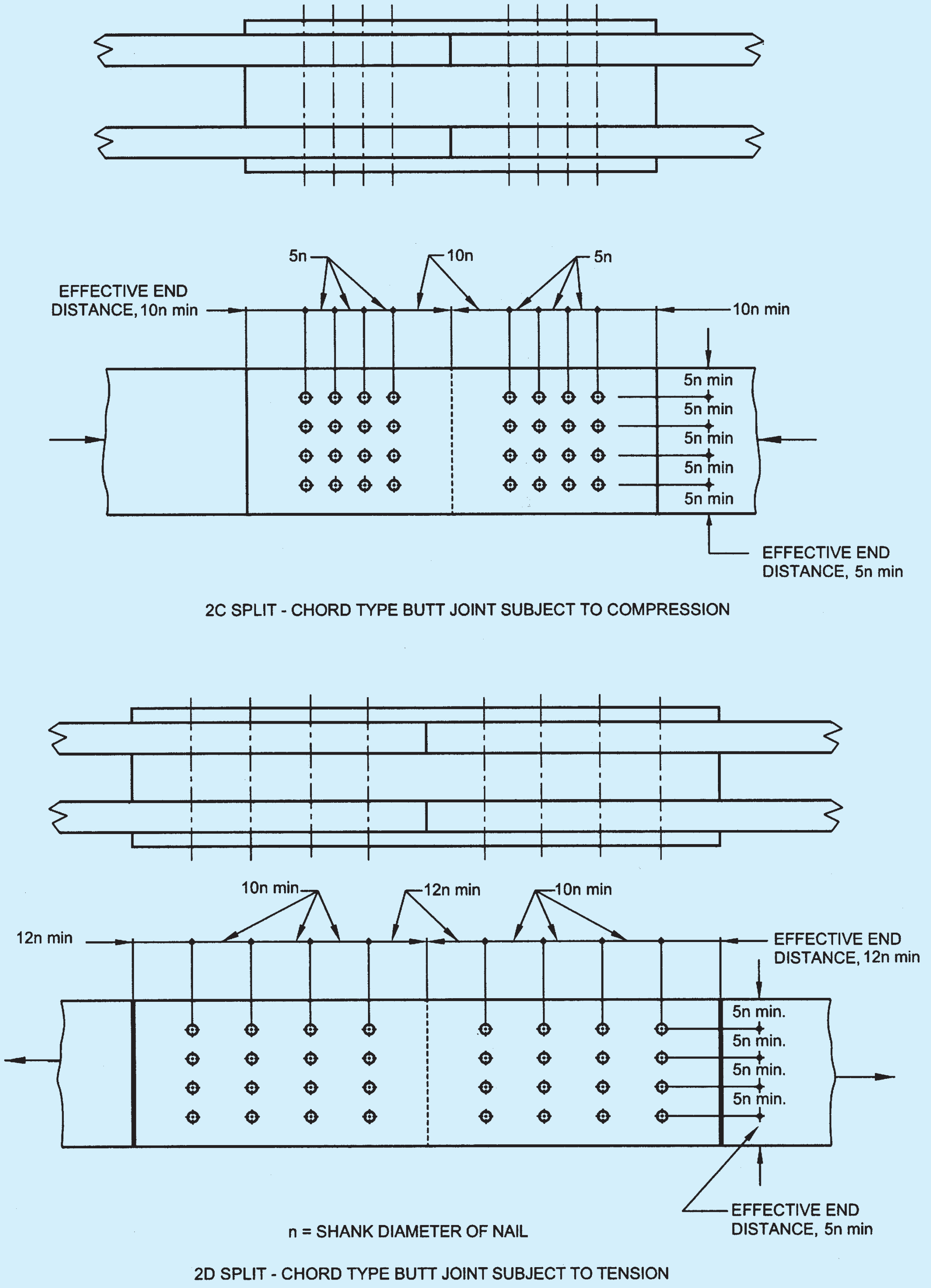


FIG. 2 SPACING OF NAILS IN A LENGHTHENING JOINT

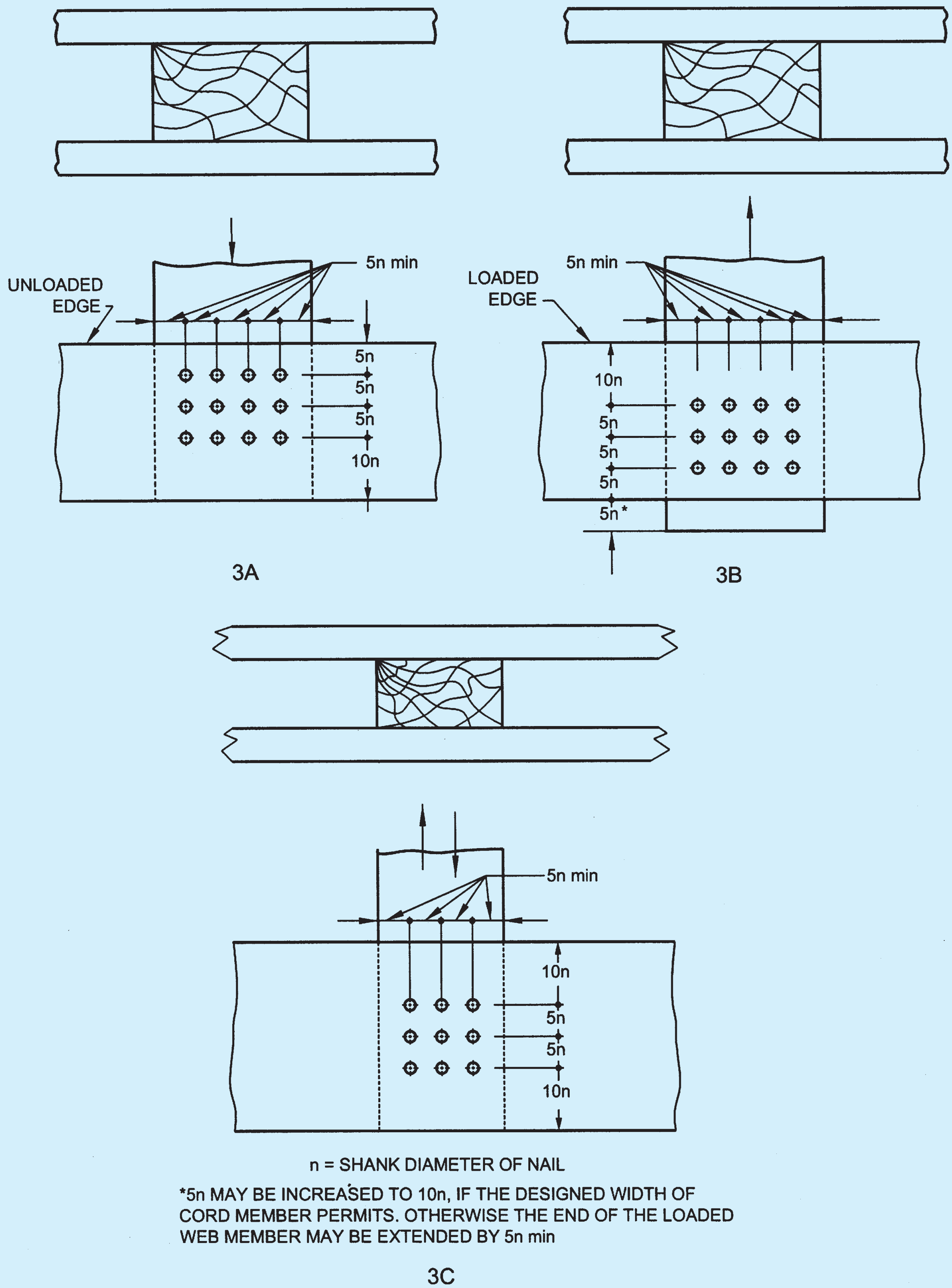


FIG. 3 SPACING OF NAILS WHERE MEMBERS ARE AT RIGHT ANGLES TO ONE ANOTHER

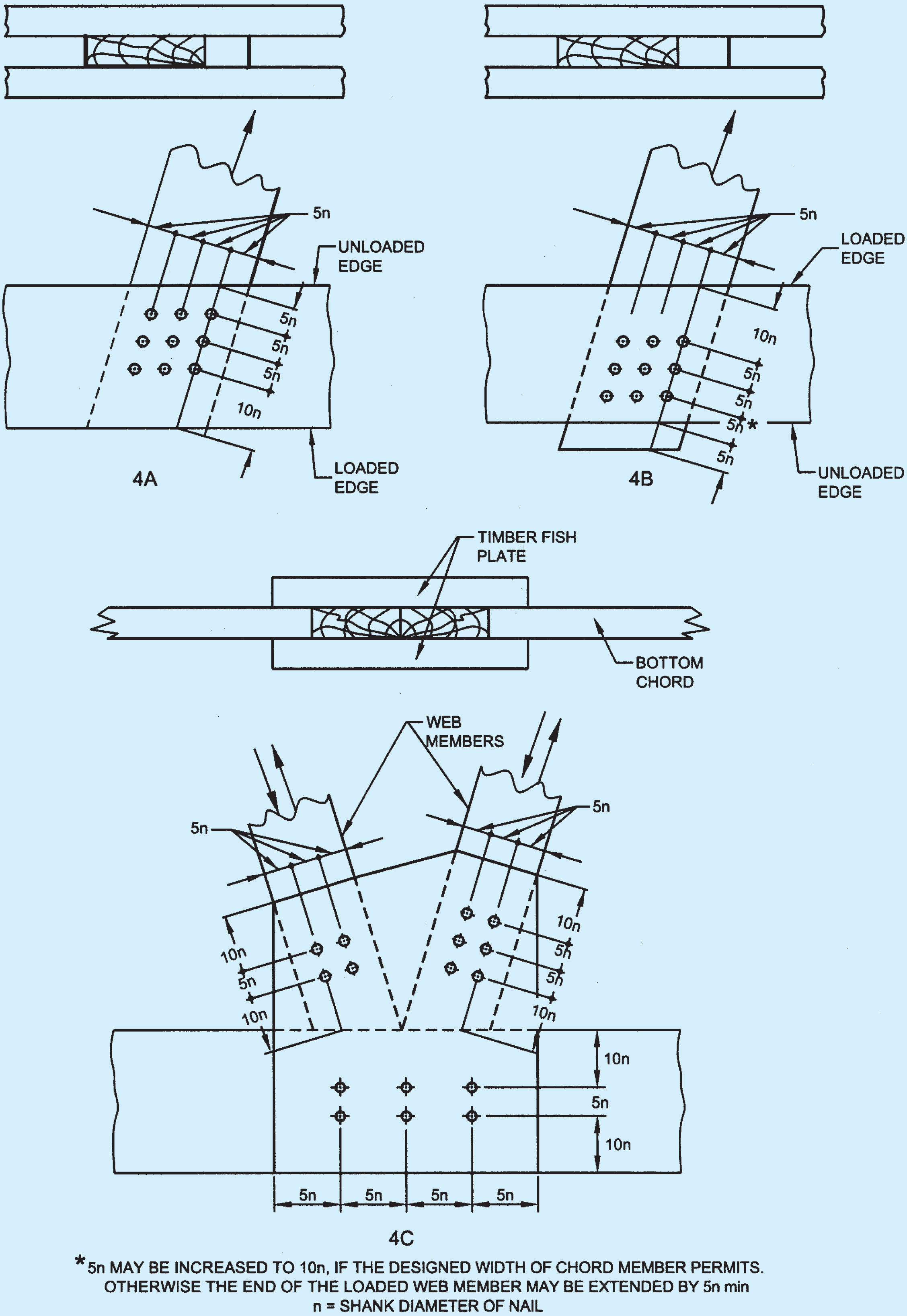


FIG. 4 SPACING OF NAILS AT NODE JOINTS WHERE MEMBERS ARE INCLINED TO ONE ANOTHER

NOTES

1 The allowable load or lateral strength values of nails shall be those as given in Table 13.

2 The strength data for joints given in the section 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 [6-3A(7)].

8 DESIGN OF NAIL LAMINATED TIMBER BEAMS

8.1 Method of Arrangement

8.1.1 The beam is 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. 5).

8.1.2 The advantage in laminations lies in dimensional stability, 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, 25 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 14. 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**.

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 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.

8.3.3 *Arrangement of Nails*

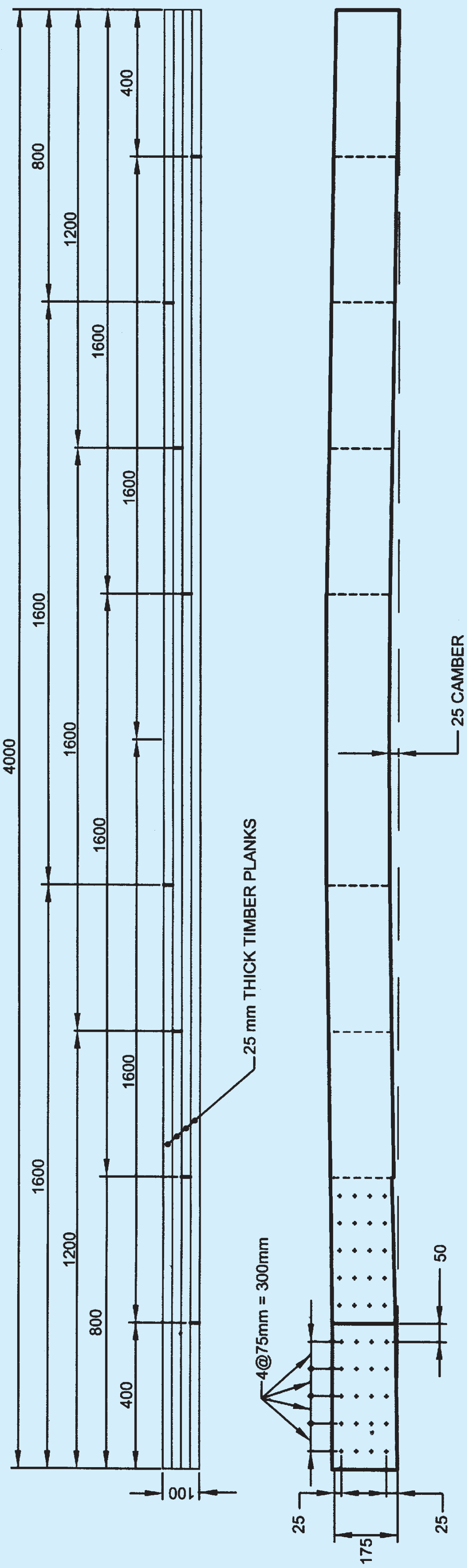
8.3.3.1 A minimum number of four nails in a vertical row at regular interval not exceeding 75 mm to take up horizontal shear as well as to keep the planks in position shall be used. Near the joints of the planks this distance may, however, be limited to 5 cm instead of 75 mm.

Table 14 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

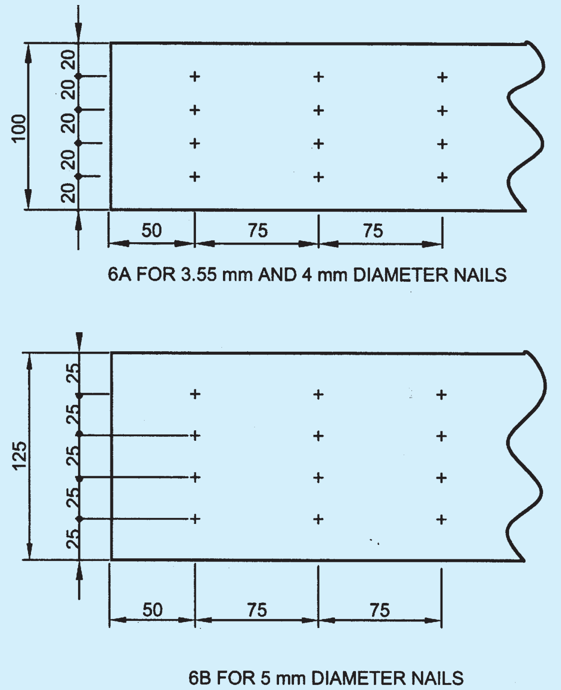
NOTE — A number of combinations of the different thickness of planks may be adopted as long as the minimum and maximum thickness of the planks are adhered to.



All dimensions in millimetres.

FIG. 5 PLAN AND ELEVATION OF A TYPICAL NAILED LAMINATED TIMBER BEAM

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. 6.



All dimensions in millimetres.

FIG. 6 STANDARD LENGTHWISE SPACING IN NAILED LAMINATED BEAM

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

8.4.1 The fabrication of nailed laminated timber beams shall be done in accordance with good practice [6-3A(8)].

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 rivetted 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 15.

Table 15 Percentage of Safe Working Compressive Stress of Timber for Bolted Joints in Double Shear

(*Clause* 9.2.2)

t'/d_3 Ratio	Stress Percentage	
	Parallel to Grain λ_1	Perpendicular to Grain λ_2
(1)	(2)	(3)
1.0	100	100
1.5	100	96
2.0	100	88
2.5	100	80
3.0	100	72
3.5	100	66
4.0	96	60
4.5	90	56
5.0	80	52
5.5	72	49
6.0	65	46
6.5	58	43
7.0	52	40
7.5	46	39
8.0	40	38
8.5	36	36
9.0	34	34
9.5	32	33
10.0	30	31
10.5	—	31
11.0	—	30
11.5	—	30
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 16.

Table 16 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

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.
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.
f) Washers shall be used between the head of bolt and wood surface as also between the nut and wood.

9.3 Arrangement of Bolts

9.3.1 The following spacings in bolted joints shall be followed (*see* Fig. 7):

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 to 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 soft woods in tension, 5 d_3 for hardwoods in tension and 4 d_3 for all species in compression.
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, (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 inter-section 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*

$$P = f_{cp}\ a\ \lambda_1$$

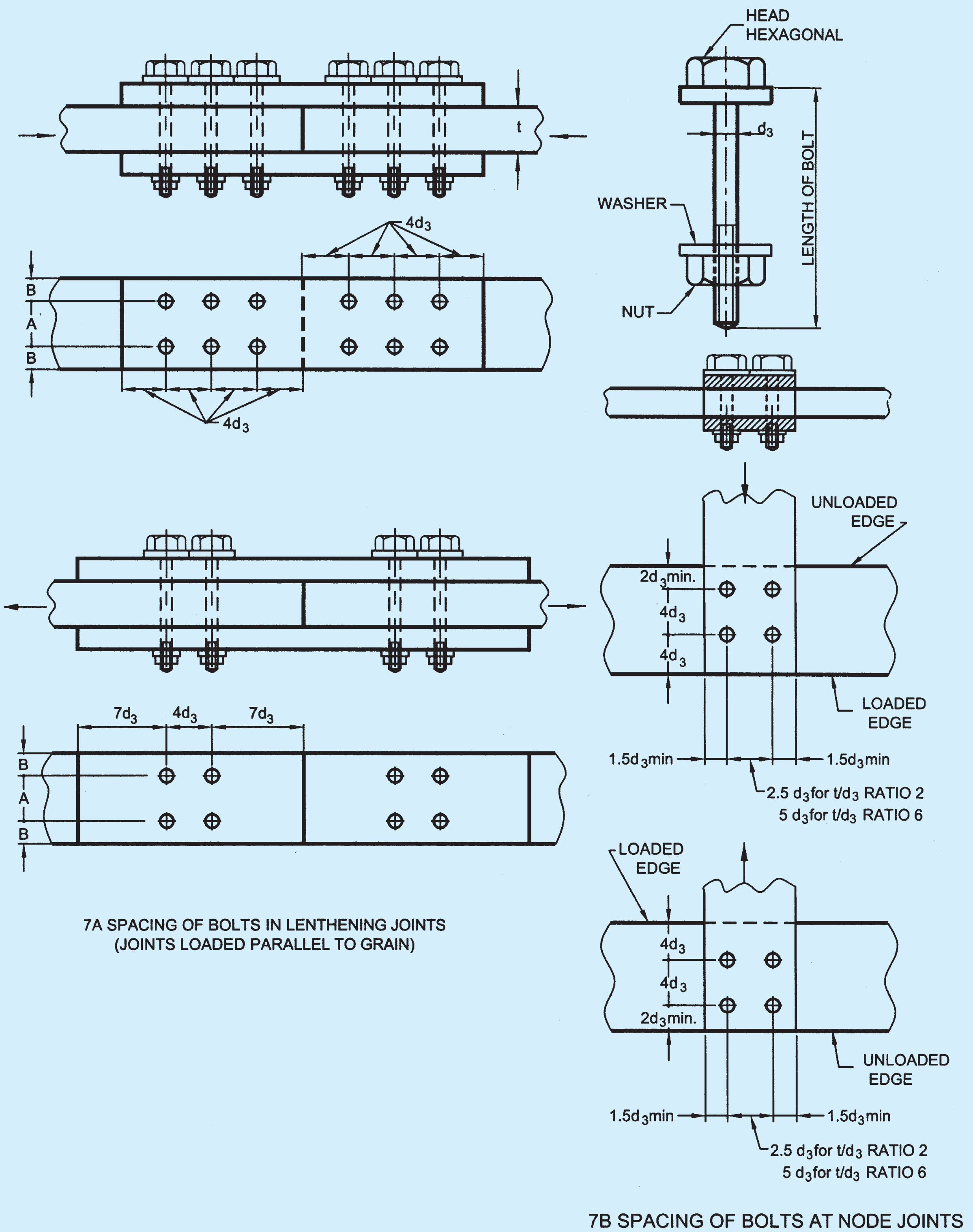


FIG. 7 TYPICAL SPACING OF BOLTS IN STRUCTURAL JOINTS

b) *For Loads Perpendicular to Grain*

$$R = f_{cn}\ a \lambda_2 d_f ,$$

c) *For Loads at an Angle to Grain*

$$F = \frac{PR}{P \sin^2 \theta + R \cos^2 \theta}$$

9.5 Fabrication

The fabrication of bolt jointed construction shall be in accordance with good practice [6-3A(9)].

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. Improvized 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.

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*

Figure 8 illustrates the primary stresses in a split ring connector joint under tension. The shaded areas represent the part of wood in shear, compression and tension. Related formulae for the same are indicated in Fig. 8.

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 good practice [6-3(10)].

NOTE — A pilot study on determination of strength of ring connector joint in a compression test for a specific design problem yielded the data as given below:

Strength of Improvized Split Ring Connectors in Mesua (*Mesua ferra*) (Pilot Study)

No. and Diameter of Ring used in a Joint		*No. and Size of Bolt used in a Joint*		*Side Member*		*Central Member*		*Load Direction w.r.t. Grains of Wood*	*End Distance*	*Inter-mediate Distance*	*Load per Pair of Connector*
No.	Size mm	No.	Size mm	Thick-ness mm	Width mm	Thick-ness mm	Width mm		mm	mm	kgf
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
2	63	1	12 × 125	31	92	38	92	Parallel	63	—	3 930
2	63	1	12 × 125	31	92	38	92	Parallel	75	—	4 185
2	63	1	12 × 150	38	92	63	92	Parallel	75	—	4 010
2	63	1	12 × 150	38	117	63	117	Parallel	75	—	4 450
2	63	1	12 × 125	31	138	38	92	Perpendicular	69	—	2 520
2	63	1	12 × 125	38	138	38	92	Perpendicular	69	—	3 515
2	100	1	19 × 175	38	138	66	138	Parallel	100	—	7 075
2	100	1	19 × 175	38	138	66	138	Parallel	125	—	7 370
2	100	1	19 × 175	41	138	75	138	Parallel	100	—	7 220
2	100	1	19 × 175	41	138	75	138	Parallel	125	—	7 645
4	100	2	19 × 200	38	138	66	138	Parallel	100	150	5 655
4	100	2	19 × 200	41	138	75	138	Parallel	100	150	5 925
4	100	2	19 × 200	41	138	75	138	Parallel	125	200	7 135

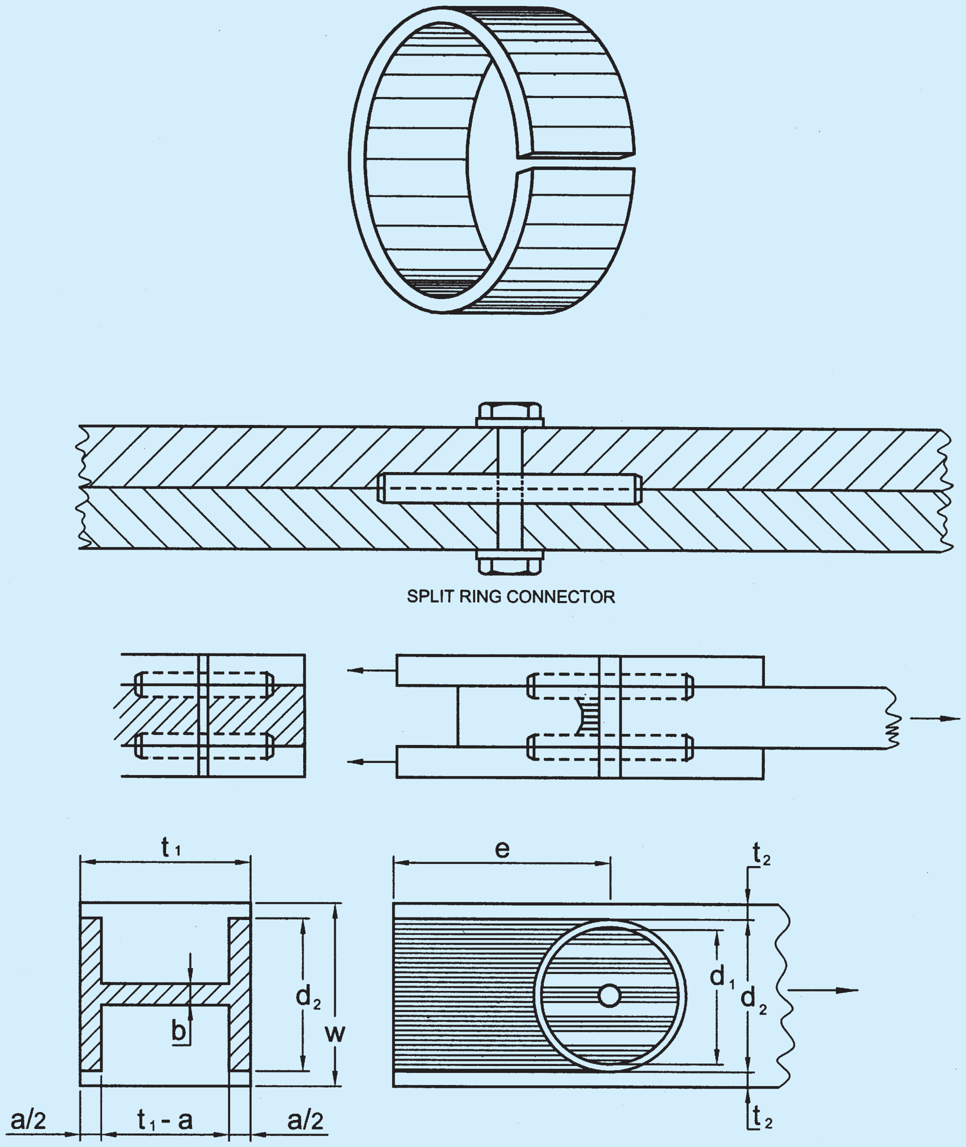


FIG. 8 STRESS DISTRIBUTION IN A SPLIT RING CONNECTOR

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.

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 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 species recommended include:

Babul

Dhaman

Irul

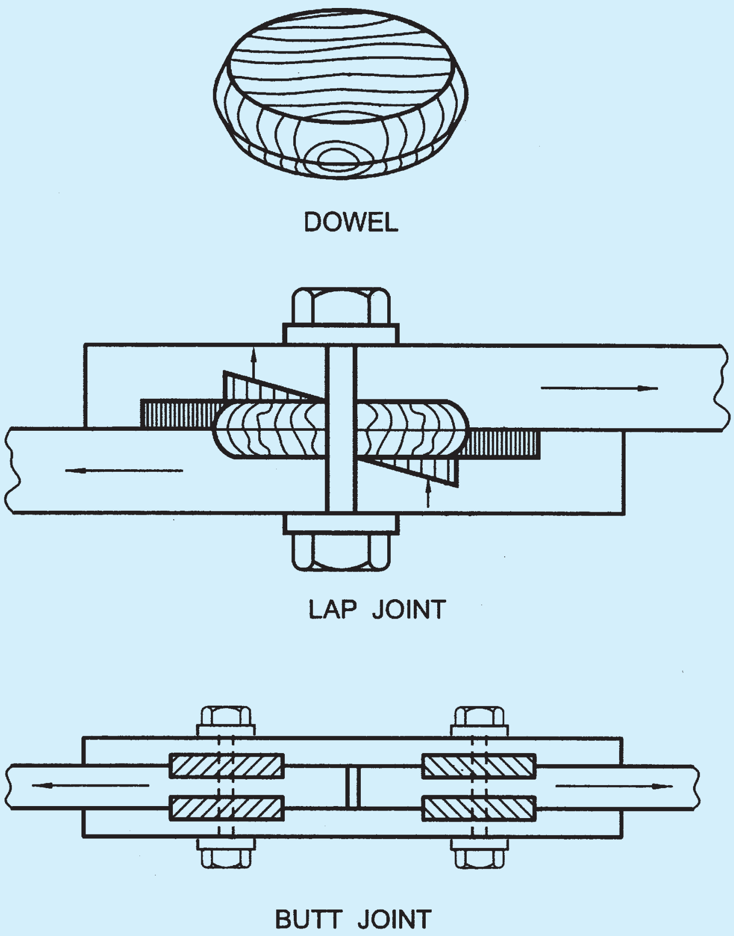
Sissoo
Rose wood
Sandal
Axle-wood
Padauk
Pyinkado
Yon

NOTE — Data on the above species as per Table 1 except for the species *Pyinkado*, which is not an indigenous species.

10.2.4 *Design Considerations*

Figure 9 illustrates the forces on dowel in a lap joint and butt joint. 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 Fig. 9 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



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.

$$\pi d^2/4 \times S = d \times t/2 \times c$$

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. 9 DISTRIBUTION OF FORCES IN DOWEL JOINT 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 kgf for dowels bearing parallel to the grain.

Members	*Dowels*	*62 × 25*	*75 × 25*	*87 × 25*	*100 × 31*	*112 × 37*	*125 × 37*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Sal	Babul	680	1 000	1 360	1 815	2 270	2 810
Sal	Sissoo	545	770	1 045	1 360	1 725	2 130

11 GLUED LAMINATED CONSTRUCTION AND FINGER JOINTS

11.1 Developments in the field of synthetic adhesive have brought glueing 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 gluelam. 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 lengths 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 [6-3A(11)]. However, it is important that only boiling water proof (BWP) grade adhesives shall be used for fabrication of gluelam 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 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.

11.2 Finger joints are glued joints connecting timber members end-to-end (Fig. 10). 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, glueing the interfaces and then meeting 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.

11.2.1 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.

11.2.1.1 The figures 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. Thus a joist is slightly stronger with edge-to-edge finger joints and a plank is stronger with face-to-face finger joint.

11.2.1.2 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 [6-3A(11)].

11.2.2 *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) Planning of finger-jointed planks for smooth surface.

11.2.3 *Strength*

Strength of finger joints depends upon the geometry of the profile for structural purpose; this is generally 50 mm long, 12 mm pitch.

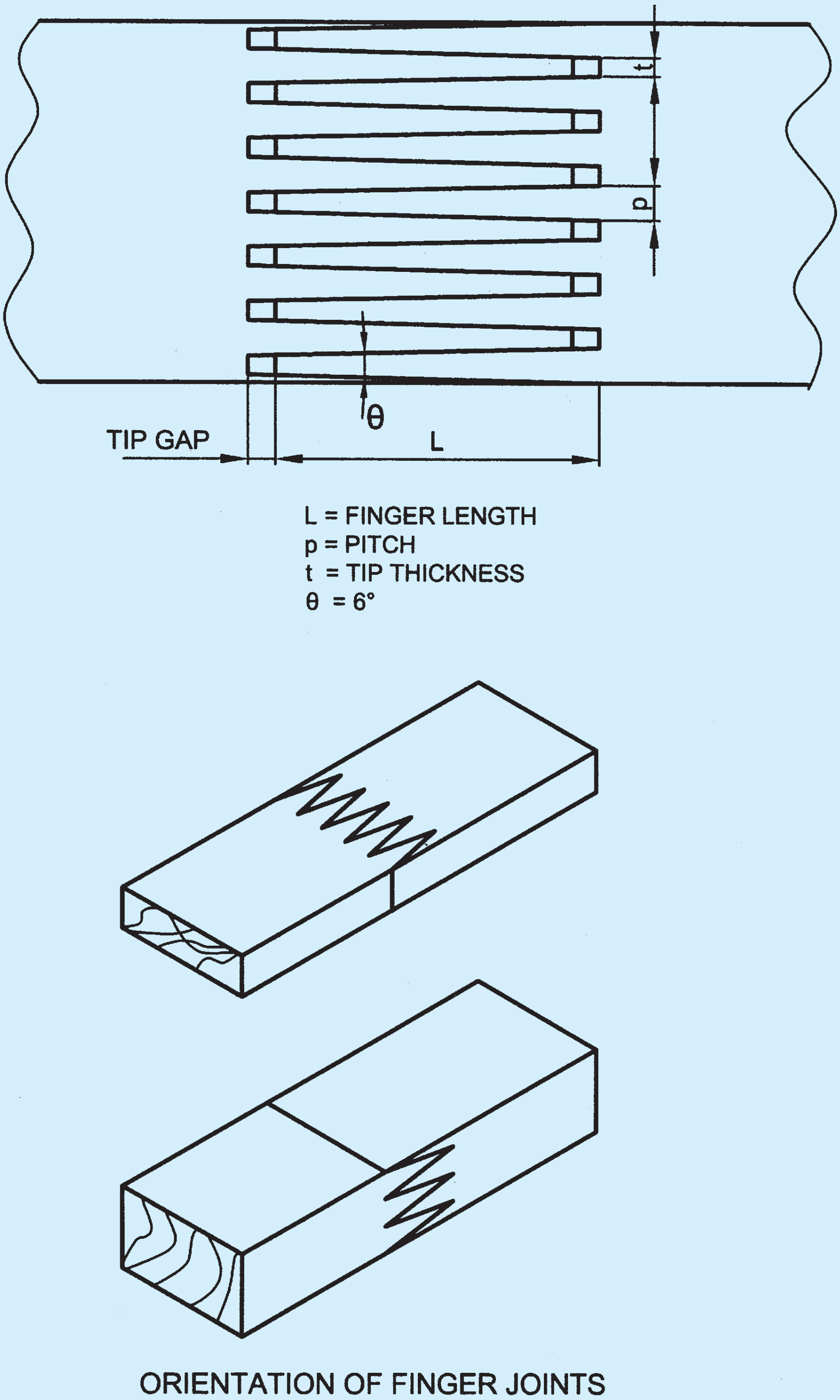


FIG. 10 TYPICAL FINGER JOINT GEOMETRY

11.2.3.1 End joints shall be scattered in adjacent laminations, which shall not be located in very highly stressed outer laminations.

11.2.4 Tip thickness will be as small as practically possible.

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 0.6 to 0.75 which is manufactured in accordance with good practice [6-3A(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*

Jointing gaps — Not more than 3 mm wide, provided they are well staggered in their spacing and position between the successive plies.

Slope of grain — Not exceeding 1 in 10 in the face layers.

Tight knot — Three numbers up to 25 mm diameter in one square metre provided they are spaced 300 mm or more apart.

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 17.

13 DESIGN OF GLUED LAMINATED BEAMS

13.1 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. 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.

13.2 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.

13.3 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 glueing. The moisture content of individual

Table 17 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)	Compressive strength perpendicular to grain:	
	a) Parallel to grain (N/mm^2), *Min*	35
	b) Perpendicular to grain (N/mm^2), *Min*	50
v)	Horizontal shear:	
	a) Parallel to laminae (N/mm^2), *Min*	6
	b) Perpendicular to laminae (N/mm^2), *Min*	8
vi)	Tensile strength parallel to grain (N/mm^2), *Min*	55
vii)	Screw holding power:	
	a) Edge (N), *Min*	2 300
	b) Face (N), *Min*	2 700
viii)	Thickness swelling in 2 h water soaking (percent), *Max*	3

laminations in a structural member shall not differ by more than 3 percent at the time of glueing. Glue shall be of type suitable for the intended service of a structural member.

13.4 Fabrication/Manufacture

In order to assure a well-bonded and well-finished member of true shape and size, all equipments, 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.

13.5 Testing

For examining the quality of glue and its relative strength *vis-à-vis* species of timber in glued laminated construction, it is necessary to conduct block shear and other related tests in accordance with accepted standard [6-3A(11)].

Structural loading tests on prototype sizes provide information on the strength properties, stiffness or rigidity against deflection of a beam.

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 would be manufactured in accordance with accepted standard [6-3A(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), (Fig. 11 and Fig. 12) stressed skin panels (Fig. 13) used for flooring and walling and pre-fabricated houses, cabins, etc. Glueing can be carried out by nail glueing 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 spares. 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.

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 quack 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, 9-12 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-12 mm thick structural plywood can be very well used as nailed or bolded gussets in fixing members of trusses or lattice griders 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.

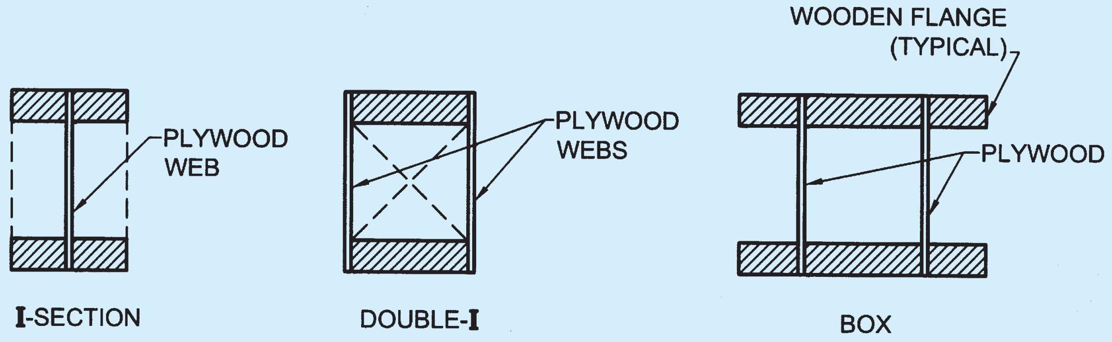


FIG. 11 TYPICAL CROSS-SECTION OF WEB BEAMS

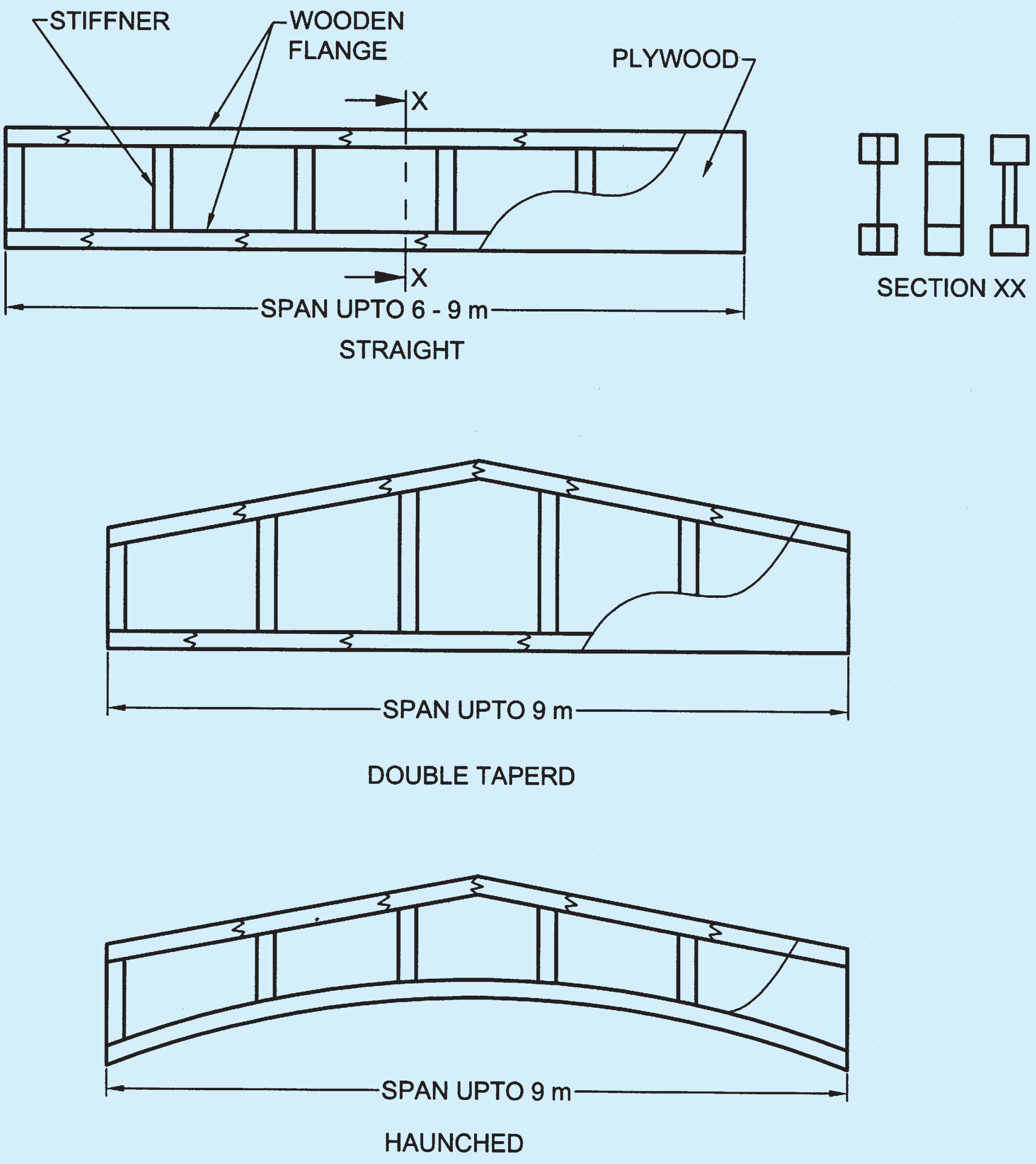


FIG. 12 WEB BEAM CONFIGURATIONS

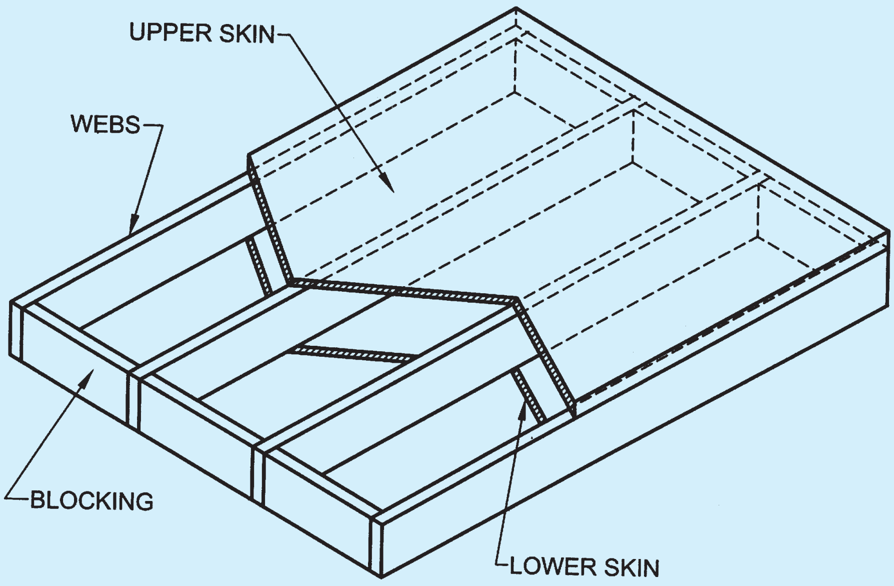


FIG. 13 STRESSED SKIN PANEL CONSTRUCTION (SINGLE SKIN OR DOUBLE SKIN)

Trussed rafters require to be supported only at their ends so that there is no need to provide load bearing internal walls, Purlins, etc are dispensed with and in comparison with traditional methods of construction they use less timber and considerably reduces of site labour, Mass production or reliable units can be carried out under workshop controls.

15.2 Design

Trussed rafter shall be designed to sustain the dead and imposed loads specified in Part 6 'Structural Design: Section 1 Loads, Forces and Effects' 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.

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 to be as per zonal requirements in accordance with **4.4**.

15.4 Plywood

Boiling water resistant (BWR) grade preservative treated plywood shall be used in accordance with accepted standard [6-3A(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 [6-3A(14)].

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 (Fig. 14). The thin facings are usually of strong dense material since that are the principal load carrying members of the construction. The core must 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 glueing 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.

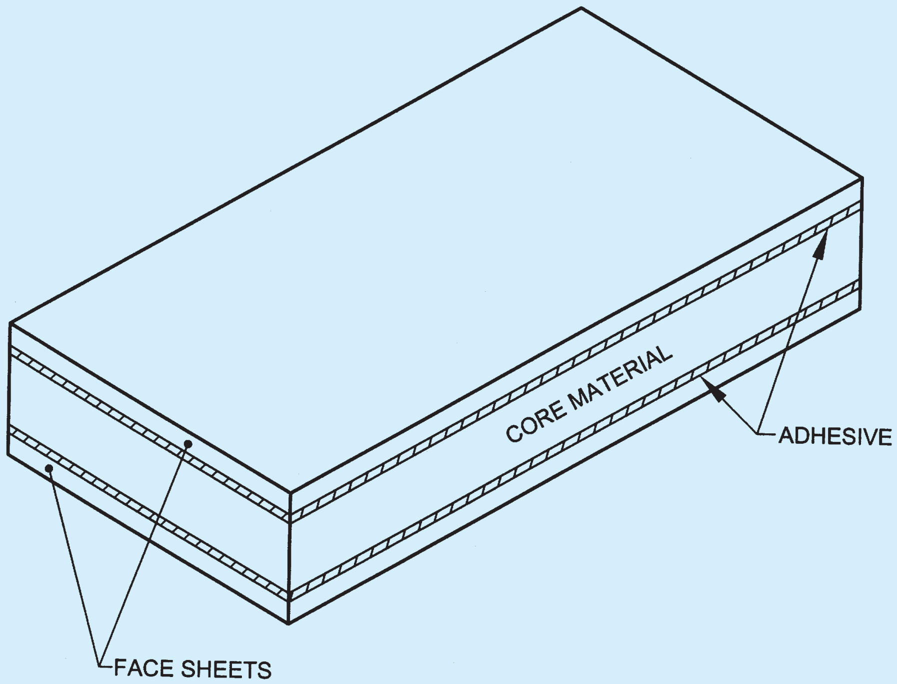


FIG. 14 STANDWICH CONSTRUCTION IN STRUCTURAL APPLICATIONS

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.

16.5 Tests

Panels of sandwich construction shall be subject to testing in accordance with accepted standards [6-3A(15)]. 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 (Fig. 15).

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

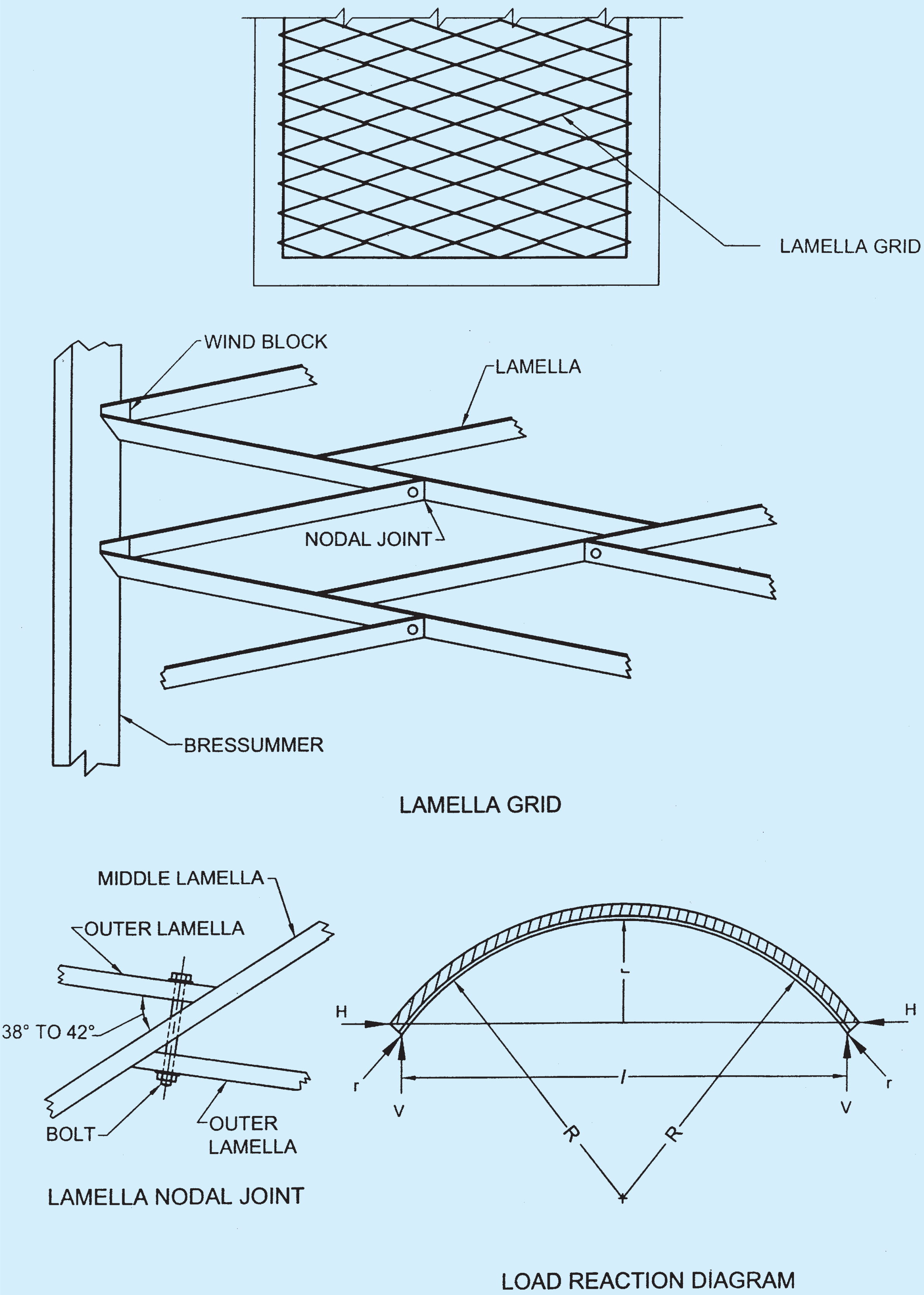


Fig. 15 Typical Arrangement of Lamella Roofing

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 sheathening (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 are 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.

17.3 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.

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 as standard code of practice. 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 standards [6-3A(16)]. 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 standard [6-3A(17)].

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 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.

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 [6-3A(18)].

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 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.

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 the enforcement of the Code. The standards listed may be used by the Authority as a guide in conformance with the requirements of the referred clauses in the Code.

	IS No.	*Title*
(1)	707 : 1976	Glossary of terms applicable to timber technology and utilization (*second revision*)
(2)	1708 (Parts 1 to 18) : 1986	Methods of testing small clear specimens of timber (*second revision*)
	2408 : 1963	Methods of static tests of timbers in structural sizes
	2455 : 1990	Methods of sampling of model trees and logs for timber testing and their conversion (*second revision*)
(3)	4970 : 1973	Key for identification of commercial timbers (*first revision*)
(4)	287 : 1993	Recommendations for maximum permissible moisture content of timber used for different purposes (*third revision*)
(5)	3629 : 1986	Specification for structural timber in building (*first revision*)
(6)	401 : 2001	Code of practice for preservation of timber (*fourth revision*)
(7)	2366 : 1983	Code of practice for nail-jointed timber construction (*first revision*)

IS No.	*Title*
(8) 4983 : 1968	Code of practice for design and construction of nail laminated timber beam (*first revision*)
(9) 11096 : 1984	Code of practice for design construction of bolt-jointed construction
(10) 4907 : 1968	Methods of testing timber connectors
(11) 9188 : 1979	Performance requirements for adhesive for structural laminated wood products for use under exterior exposure condition
(12) 14616 : 1999	Specification for laminated veneer lumber
(13) 10701 : 1983	Specification for structural plywood
4990 : 1993	Specification for plywood for concrete shuttering work (*second revision*)
(14) 12120 : 1987	Code of practice for preservation of plywood and other pane products
(15) 9307 (Parts 1 to 7) : 1979	Methods of test for wood based structural sandwich construction
(16) 723 : 1972	Specification for steel countersunk head wire nails (*first revision*)
(17) 451 : 1999	Technical supply conditions for wood screws (*fourth revision*)
6736 : 1972	Specification for slotted countersunk head wood screw
6739 : 1972	Specification for slotted round head wood screw
6760 : 1972	Specification for slotted countersunk head wood screws
(18) 6313	Code of practice for anti-termite measures in buildings:
(Part 1) : 1981	Constructional measures (*first revision*)
(Part 2) : 2001	Pre-constructional chemical treatment measures (*second revision*)
(Part 3) : 2001	Treatment for existing buildings (*second revision*)

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 3 Timber and Bamboo: 3B Bamboo

BUREAU OF INDIAN STANDARDS

CONTENTS

National Building Code Sectional Committee, CED 46

FOREWORD

Bamboo is versatile resource characterized by high strength to weight ratio and ease in working with simple tools. It has a long and well established tradition as a building material throughout the tropical and sub-tropical regions. It is used in many forms of construction, particularly, for housing in rural areas. But, enough attention had not been paid towards research and development in bamboo as had been in the case with other materials of construction including timber. However, of late bamboo is being given its due importance and realization by national and international organizations. A need is being felt for design and construction code in bamboo for a number of social and trade advantages, engineering recognition and improved respectability. Forest Research Institute, Dehra Dun and some other organizations have been engaged in bamboo research to establish its silviculture, botanical nomenclature, entomological and pathological aspects and utilization base.

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.

Analogous to some constructional timbers, bamboo possesses better strength-to-mass and cost ratio. Resilience coupled with lightness makes bamboo suitable for housing in disaster-prone areas such as areas prone to earthquake. It has the capacity to absorb more energy and show larger deflections before collapse and as such is safer under earth tremors. At present, the application of bamboo as an engineering material is largely based on practical and engineering experience as the design guidelines are inadequate.

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.

In the earlier version of this Code, timber was covered under Section 3 of Part 6 under the title Wood, which did not cover Bamboo. In this revision, this Section has been enlarged and titled as Section 3 Timber and Bamboo. This has been sub-divided into sub-section 3A Timber and sub-section 3B Bamboo. Bamboo has found a place in this draft revision of the Code for the first time. This subsection pertains to bamboo and may be read in conjunction with sub-section 3A Timber.

The information contained in this Section is largely based on the works carried out at Forest Research Institute, Dehra Dun, Indian Plywood Industries Research and Training Institute, Bangalore, INBAR documents and the following Indian Standards:

IS No.	*Title*
6874 : 1973	Method of test for round bamboo
8242 : 1976	Method of test for split bamboo
9096 : 1979	Code of practice for preservation of bamboo for structural purposes
13958 : 1994	Specification for bamboo mat board for general purposes

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.

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 3 Timber and Bamboo: 3B Bamboo

1 SCOPE

This Section relates to the use of bamboo for constructional purposes in structures or elements of the structure, ensuring quality and effectiveness of design and construction using bamboo. It covers minimum strength data, dimensional and grading requirements, seasoning, preservative treatment, design and jointing techniques with bamboo which would facilitate scientific application and long-term performance of structures. It also covers guidelines so as to ensure proper procurement, storage, precautions and design limitations on bamboo.

2 TERMINOLOGY

2.0 For the purpose of this Section, 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 *Bambusoidae*.

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 rhizomer 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 cytoplasma 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.

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.

2.2.2 *Beam* — A structural member which supports load primarily by its internal resistance to bending.

2.2.3 *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.

2.2.4 *Bundle-Column* — A column consisting of three or more number of culm bound as integrated unit with wire or strap type of fastenings.

2.2.5 *Centre Internode* — A test specimen having its centre between two nodes.

2.2.6 *Characteristic Load* — The value of loads which has a 95 percent probability of not exceeding during the life of the structure.

2.2.7 *Characteristic Strength* — The strength of the material below which not more than 5 percent of the test results are expected to fall.

2.2.8 *Cleavability* — The ease with which bamboo can be split along the longitudinal axis. The action of splitting is known as cleavage.

2.2.9 *Column* — A structural member which supports axial load primarily by inducing compressive stress along the fibres.

2.2.10 *Common Rafter* — A roof member which supports roof battens and roof coverings, such as boarding and sheeting.

2.2.11 *Curvature* — The deviation from the straightness of the culm.

2.2.12 *Delamination* — Separation of mats through failure of glue.

2.2.13 *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.

2.2.14 *Flatten 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.

2.2.15 *Full Culm* — The naturally available circular section/shape.

2.2.16 *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.

2.2.17 *Inner Diameter* — Diameter of internal cavity of a hollow piece of bamboo.

2.2.18 *Inside Location* — Position in buildings in which bamboo remains continuously dry or protected from weather.

2.2.19 *Joint* — A connection between two or more bamboo structural elements.

2.2.20 *Joist* — A beam directly supporting floor, ceiling or roof of a structure.

2.2.21 *Length of Internode* — Distance between adjacent nodes.

2.2.22 *Loaded End or Compression End Distance* — The distance measured from the centre of the fastner to the end towards which the load induced by the fastener acts.

2.2.23 *Matchet* — A light cutting and slashing tool in the form of a large knife.

2.2.24 *Mat* — A woven sheet made using thin slivers.

2.2.25 *Mortise and Tenon* — A joint in which the reduced end (tenon) of one member fits into the corresponding slot (mortise) of the other.

2.2.26 *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.

2.2.27 *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.

2.2.28 *Outer Diameter* — Diameter of a cross-section of a piece of bamboo measured from two opposite points on the outer surface.

2.2.29 *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.

2.2.30 *Permissible Stress* — Stress obtained after applying factor of safety to the ultimate or basic stress.

2.2.31 *Principal Rafter* — A roof member which supports purlins.

2.2.32 *Purlins* — A roof member directly supporting roof covering or common rafter and roof battens.

2.2.33 *Roof Battens* — A roof member directly supporting tiles, corrugated sheets, slates or other roofing materials.

2.2.34 *Roof Skeleton* — The skelton 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.

2.2.35 *Slenderness Ratio* — The ratio of the length of member to the radius of gyration is known as slenderness ratio of member. (The length of the member is the equivalent length due to end conditions).

2.2.36 *Splits* — The pieces made from quarters by dividing the quarters radially and cutting longitudinally.

2.2.37 *Taper* — The ratio of difference between minimum and maximum outer diameter to length.

2.2.38 *Unloaded End Distance* — The end distance opposite to the loaded end.

2.2.39 *Wall Thickness* — Half the difference between outer diameter and inner diameter of the piece at any cross-section.

2.2.40 *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. *Bostrychdae*), 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 Definitions 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

3.1 For the purpose of this Section, 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

$$A = \frac{\pi}{4}(D^2 - d^2)$$

D = Outer diameter, mm

d = Inner diameter, mm

E = Modulus of elasticity 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, mm^4

$$= \frac{\pi}{64}(D^4 - d^4)$$

l = Unsupported length of column

M = Moisture content, percent

r = Radius of gyration

$= \sqrt{(I/A)}$

R' = Modulus of rupture, N/mm^2

w = Wall thickness, mm

Z = Section modulus, mm^3

δ = Deflection or deformation, mm.

4 MATERIALS

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. Physical and mechanical properties of 20 species of bamboo so far tested in green and dry conditions in round form are given in Table 1.

4.1.1 *Grouping*

Sixteen species of bamboo found suitable for structural applications and classified into three groups, namely, Group A, Group B and Group C are given in Table 2.

The characteristics of these groups are as given below:

Limiting Strength Values (in Green Condition)

	Modulus of Rupture (R') N/mm^2	*Modulus of Elasticity (E) in Bending* 10^3 N/mm^2
(1)	(2)	(3)
Group A	$R' > 70$	$E > 9$
Group B	$70 \geq R' > 50$	$9 \geq E > 6$
Group C	$50 \geq R' > 30$	$6 \geq E > 3$

4.1.2 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.

4.1.3 *Dendrocalamus strictus* and *Bambusa arundinacea* are the two principal economic species of which the former occupies the largest area and is the most common owing to the vast expanse and suitability as a raw material for industrial uses.

4.2 Species of bamboo other than those listed in the Table 2 may be used, provided the basic strength characteristics are determined and found more than the limits mentioned therein. However, in the absence of testing facilities and compulsion for use of other species, and for expedient designing, allowable stresses may be arrived at by multiplying density with factors as given below:

Allowable Long-Term Stress (N/mm^2) per Unit Density (kg/m^3)

Condition	*Axial Compression* (no buckling)	*Bending*	*Shear*
Green	0.011	0.015	—
Air dry (12%)	0.013	0.020	0.003

NOTE — In the laboratory regime, the density of bamboo is conveniently determined. Having known the density of any species of bamboo, permissible stresses can be worked out using factors indicated above. For example, if green bamboo has a density of 600 kg/m^3, the allowable stress in bending would be $0.015 \times 600 = 9$ N/mm^2.

4.3 Moisture Content in Bamboo

With decrease of moisture content (M) the strength of

Table 1 Physical and Mechanical Properties of Indian Bamboos (in Round Form)

(*Clauses* 4.1 *and* 5.1.2)

Sl No.	Species	Properties							
		In Green Condition				In Air Dry Conditions			
		Density kg/m^3	Modulus of Rupture N/mm^2	Modulus of Elasticity 10^3 N/mm^2	Maximum Compressive Strength N/mm^2	Density kg/m^3	Modulus of Rupture N/mm^2	Modulus of Elasticity 10^3 N/mm^2	Maximum Compressive Strength 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	—	—	—	—
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. polymorpha*	619	28.3	3.12	32.1	659	35.5	4.40	—
ix)	*B. tulda*	658	51.1	7.98	40.7	722	66.7	10.07	68.0
x)	*B. ventricosa*	626	34.1	3.38	36.1	—	—	—	—
xi)	*B. vulgaris*	626	41.5	2.87	38.6	—	—	—	—
xii)	*Cephalostachyum pergracile*	601	52.6	11.16	36.7	640	71.3	19.22	49.4
xiii)	*Dendrocalamus giganteous*	597	17.2	0.61	35.2	—	—	—	—
xiv)	*D. hamiltonii*	515	40.0	2.49	43.4	—	—	—	—
xv)	*D. longispathus*	711	33.1	5.51	42.1	684	47.8	6.06	61.1
xvi)	*D. membranacaus*	551	26.3	2.44	40.5	664	37.8	3.77	—
xvii)	*D. strictus*	631	73.4	11.98	35.9	728	119.1	15.00	69.1
xviii)	*Melocanna baccifera*	817	53.2	11.39	53.8	751	57.6	12.93	69.9
xix)	*Oxytenanthera abyssinicia*	688	83.6	14.96	46.6	—	—	—	—
xx)	*Thyrsostachys oliveri*	733	61.9	9.72	46.9	758	90.0	12.15	58.0

NOTES

1 As the strength of split bamboo is more than that of round bamboo, the results of tests on round bamboo can be safely used for designing with spit bamboo.

2 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.

Table 2 Safe Working Stresses of Bamboos for Structural Designing[1)]

(*Clauses* 4.1.1, 4.2, 5.3 *and* 5.4)

Sl No.	Species	Extreme Fibre Stress in Bending N/mm²	Modulus of Elasticity 10^3 N/mm²	Allowable Compressive Stress N/mm²
(1)	(2)	(3)	(4)	(5)
	GROUP A			
i)	*Bambusa glancescens (syn. B. nana)*	20.7	3.28	15.4
ii)	*Dendrocalamus strictus*	18.4	2.66	10.3
iii)	*Oxytenanthera abyssinicia*	20.9	3.31	13.3
	GROUP B			
iv)	*Bambusa balcooa*	16.4	1.62	13.3
v)	*B. pallida*	13.8	2.87	15.4
vi)	*B. nutans*	13.2	1.47	13.0
vii)	*B. tulda*	12.8	1.77	11.6
viii)	*B. auriculata*	16.3	3.34	10.5
ix)	*B. burmanica*	14.9	2.45	11.4
x)	*Cephalostachyum pergracile*	13.2	2.48	10.5
xi)	*Melocanna baccifera (Syn. M. bambusoides)*	13.3	2.53	15.4
xii)	*Thyrsotachys oliveri*	15.5	2.16	13.4
	GROUP C			
xiii)	*Bambusa arundinacea (Syn. B. bambos)*	14.6	1.32	10.1
xiv)	*B. ventricosa*	8.5	0.75	10.3
xv)	*B. vulgaris*	10.4	0.64	11.0
xvi)	*Dendrocalamus longispathus*	8.3	1.22	12.0

NOTE — The values of stress in N/mm² have been obtained by converting the values in kgf/cm² by dividing the same by 10.

[1)] The values given pertain to testing of bamboo in green condition.

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 good practice [6-3B(1)]. Matured culms shall be seasoned to about 20 percent moisture content before use.

4.3.1 Air seasoning of split or half-round bamboo does not pose much problem but care has to be taken to prevent fungal discolouration and decay. However, rapid drying in open sun can control decay due to fungal and insect attack. Seasoning in round form presents considerable problem for several of Indian species of bamboo as regards mechanical degrade due to drying defects.

NOTE — A general observation has been that immature bamboo gets invariably deformed in cross-section during seasoning and thick walled immature bamboo generally collapses. Thick mature bamboo tends to crack on the surface, with the cracks originating at the nodes and at the decayed points. Moderately thick immature and thin and moderately thick mature bamboos season with much less degrade. Bamboo having poor initial condition on account of decay, borer holes, etc generally suffers more drying degrades.

4.3.2 Accelerated air seasoning method gives good results. In this method, the nodal diaphragm (septa) are punctured to enable thorough passage of hot air from one end of the resulting bamboo tube to the other end.

NOTE — For details, reference may be made to relevant publications of Forest Research Institute, Dehra Dun.

4.4 Grading of Structural Bamboo

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,
e) Wall thickness,
f) Density and strength, and
g) Durability and seasoning.

One of the above characteristics or sometimes combination of 2 or 3 characteristics form the basis of grading. The culms shall be segregated species-wise.

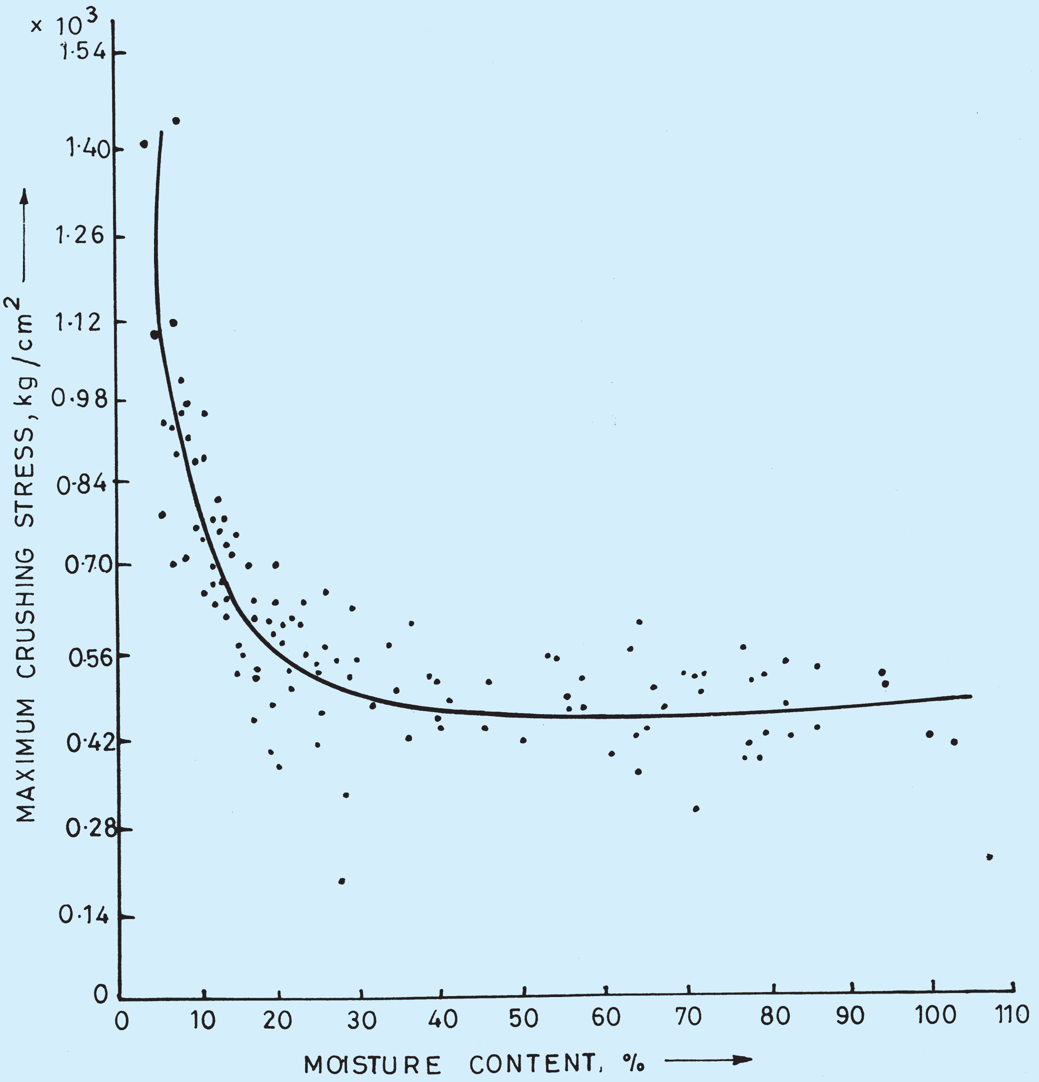


FIG. 1 MOISTURE STRENGTH RELATIONSHIP BAMBUSA NUTANS (BAMBOO)

4.4.2 *Diameter and Length*

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:

Special Grade	70 mm < Diameter ≤ 100 mm
Grade I	50 mm < Diameter ≤ 70 mm
Grade II	30 mm < Diameter ≤ 50 mm
Grade III	Diameter ≤ 30 mm

For structural Group C species culms shall be segregated in steps of 20 mm of mean outer diameter

Grade I	80 mm < Diameter ≤ 100 mm
Grade II	60 mm < Diameter ≤ 80 mm
Grade III	Diameter ≤ 60 mm

4.4.2.2 The minimum length of culms shall be preferably 6 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) of bamboo in any grade of bamboo.

4.4.4 *Curvature*

The maximum curvature shall not be more than 75 mm in a length of 6 m of any grade of bamboo.

4.4.5 *Wall Thickness*

Preferably minimum wall thickness of 8 mm shall be used for load bearing members.

4.4.6 *Defects and Permissible Characteristics*

4.4.6.1 Dead and immature bamboos, bore/*GHOON* holes, decay, collapse, checks more than 3 mm in depth, shall be avoided.

4.4.6.2 Protruded portion of the nodes shall be flushed smooth. Bamboo shall be used after at least six weeks of felling. Bamboo shall be properly treated in accordance with good practice [6-3B(2)].

4.4.6.3 Broken, damaged and discoloured bamboo shall be rejected.

4.4.6.4 Matured bamboo of at least 4 years of age shall be used.

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 biodeterioration 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. For making bamboo durable, suitable treatment shall be given.

4.5.2 *Treatability*

Due to difference in the anatomical structure of bamboo as compared to timber, bamboo behaves entirely differently from wood during treatment with preservative. Bamboos are difficult to treat by normal preservation methods in dry condition and therefore treatment is best carried out in green condition in accordance with good practice [6-3B(2)].

4.5.2.1 *Boucherie Process*

In this process of preservative treatment, water borne preservative is applied to end surface of green bamboo through a suitable chamber and forced through the bamboo by hydrostatic or other pressure.

4.5.2.2 *Performance of treated bamboo*

Trials with treated bamboos have indicated varied durability depending upon the actual location of use. The performance in partially exposed and under covered conditions is better.

4.5.2.3 For provisions on safety of bamboo structures against fire, *see* Part 7 'Constructional Practices and Safety'.

5 PERMISSIBLE STRESSES

5.1 Basic stress values of different species and groups of bamboo shall be determined according to good practice [6-3B(3)]. These values shall then be divided by appropriate factors of safety to obtain permissible stresses.

5.1.1 The safety factor for deriving safe working stresses of bamboo shall be as under:

Extreme fibre stress in beams	–	4
Modulus of elasticity	–	4.5
Maximum compressive stress parallel to grain/fibres	–	3.5

5.1.2 The coefficient of variation (in percent), which has been arrived based on data of test-results of at least 15 consignments of bamboo in green conditions, shall be as under:

Property	*Mean*	*Range*	*Maximum Expected Value*
(1)	(2)	(3)	(4)
Modulus of rupture	15.9	5.7 – 28.3	23.4
Modulus of elasticity	21.1	12.7 – 31.7	27.4
Maximum compressive stress	14.9	7.6 – 22.8	20.0

The maximum expected values of coefficient of variation which are the upper confidence limits under normality assumption such that with 97.5 percent confidence the actual strength of the bamboo culm will be at least 53 percent of the average reported value of modulus of rupture in Table 1.

5.2 Solid bamboos or bamboos whose wall thickness (w) is comparatively more and bamboos which are generally known as male bamboos having nodes very closer and growing on ridges are often considered good for structural purposes.

5.3 The safe working stresses for 16 species of bamboos are given in Table 2.

5.4 For change in duration of load other than continuous (long-term), the permissible stresses given in Table 2 shall be multiplied by the modification factors given below:

For imposed or medium term loading	–	1.25
For short-term loading	–	1.50

6 DESIGN CONSIDERATIONS

6.1 All structural members, assemblies or framework in a building shall be capable of sustaining, without exceeding the limits of stress specified, the worst combination of all loadings. 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. Accepted principles of mechanics for analysis and specified design procedures shall be applied (*see* Part 6 'Structural Design, Sub-section 3A Timber').

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 and the same may be adopted to bamboo with the limitations that practical experience rather than precise calculations generally govern the detailing.

6.3 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.

6.4 Loads

6.4.1 The loads shall be in accordance with Part 6 'Structural Design, Section 1 Loads, Forces and Effects'.

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 Structural Forms

6.5.1 Main structural components in bamboo may include roof and floor diaphragms, shear walls, wall panellings, beams, piles, columns, etc. Both from the point of view of capacity and deformation, trusses and framed skeltons are much better applications of bamboo.

6.5.2 *Schematization of Bamboo as a Structural Material*

This shall be based on the principles of engineering mechanics involving the following assumptions and practices:

a) The elastic behaviour of bamboo, till failure; (plastic behaviour being considered insignificant);
b) Bamboo culms are analysed on mean wall thickness basis as hollow tube structure (not perfectly straight) member on mean diameter basis;
c) The 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; and
d) Bamboo structures be designed like any other conventional structural elements taking care of details with regards to supports and joints; they shall be considered to generally act as a hinge, 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.

6.6.2 The deflection shall be within the prescribed limits. The tendency of bamboo beams to acquire a large deflection under long continuous loadings due to possible plastic flow, if any shall be taken care of. Permanent load may be doubled for calculation of deflection under sustained load (including creep) in case of green bamboo having moisture content exceeding 15 percent.

6.6.3 Bamboo is not naturally reinforced for shear, because, compared to reinforced cement concrete beam, the stirrups are located on the longitudinal instead of the transverse direction in a bamboo culm.

6.6.4 The moment of inertia, *I* shall be determined as follows:

a) The 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. (For each cross-section the diameter shall be taken twice, in direction perpendicular to each other and so the wall thickness shall be taken as four times, in the same places as the diameter has been taken twice.)
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.

6.6.4.1 The maximum bending stress shall be calculated and compared with the allowable stress.

6.6.4.2 The deflection shall be calculated, and compared with the allowable deflection. The initial curvature shall be considered in the calculation of the deflection.

6.6.4.3 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. For shear checks, conventional design procedure in accordance with Part 6 'Structural Design, Sub-section 3A Timber' shall be followed.

NOTE — The basic shear stress values (N/mm^2) for at least five species of bamboo in split form in green condition have been determined as under:

Bambusa pallida	9.77
B. Vulgaris	9.44
Dedrocalamus giganteous	8.86
D.hamiltonii	7.77
Oxytenanthera abyssinicia	11.2

6.6.4.4 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 Bamboo 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.
b) Calculations, based on the following:
   1) The moment of inertia shall be as per **6.6.4**.
   2) For bamboo columns the best available straight bamboo culms shall be selected. Structural bamboo components in compression should be kept under a slenderness ratio of 50.
   3) The bending stresses due to initial curvature, eccentricities and induced deflection shall be taken into account, in addition to those due to any lateral load.

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 provided it is not less than 0.6 percent.

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 Assemblies, Roof Trusses

6.8.1 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.

6.8.2 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 are 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).

6.8.3 Truss shall be analysed from principles of structural mechanics for the determination of axial forces in members. For the influence of eccentricities, due allowance shall be made in design.

6.8.4 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.

6.8.5 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.

6.8.6 For strength verification of members in compression and connections, the calculated axial forces should be increased by 10 percent.

6.8.7 The spacing of trusses shall be consistent with use of bamboo purlins (2 m to 3 m).

6.8.8 The ends in open beams, joists, rafters, purlins shall be suitably plugged. Bamboo roof coverings shall be considered as non-structural in function. The common roof covering shall include bamboo mat board, bamboo mat corrugated sheet, bamboo tiles/ strings, plastered bamboo reeds, thatch, corrugated galvanized iron sheeting, plain clay tiles or pan tiles, etc.

7 DESIGN AND TECHNIQUES OF JOINTS

7.1 Connecting the load-bearing elements together for effective transfer of stress is one of the serious problems confronted by the engineers. The size of the members in a structure depends not only on the direct loads they are required carry, but also on the ability to join the members together. Joints are quite critical in assemblies, and these should be stable in relation to time. The main objective is to achieve continuity between elements with controlled displacements. As joints are a source of weakness in any bamboo structure, they have to be made as strong and rigid as possible.

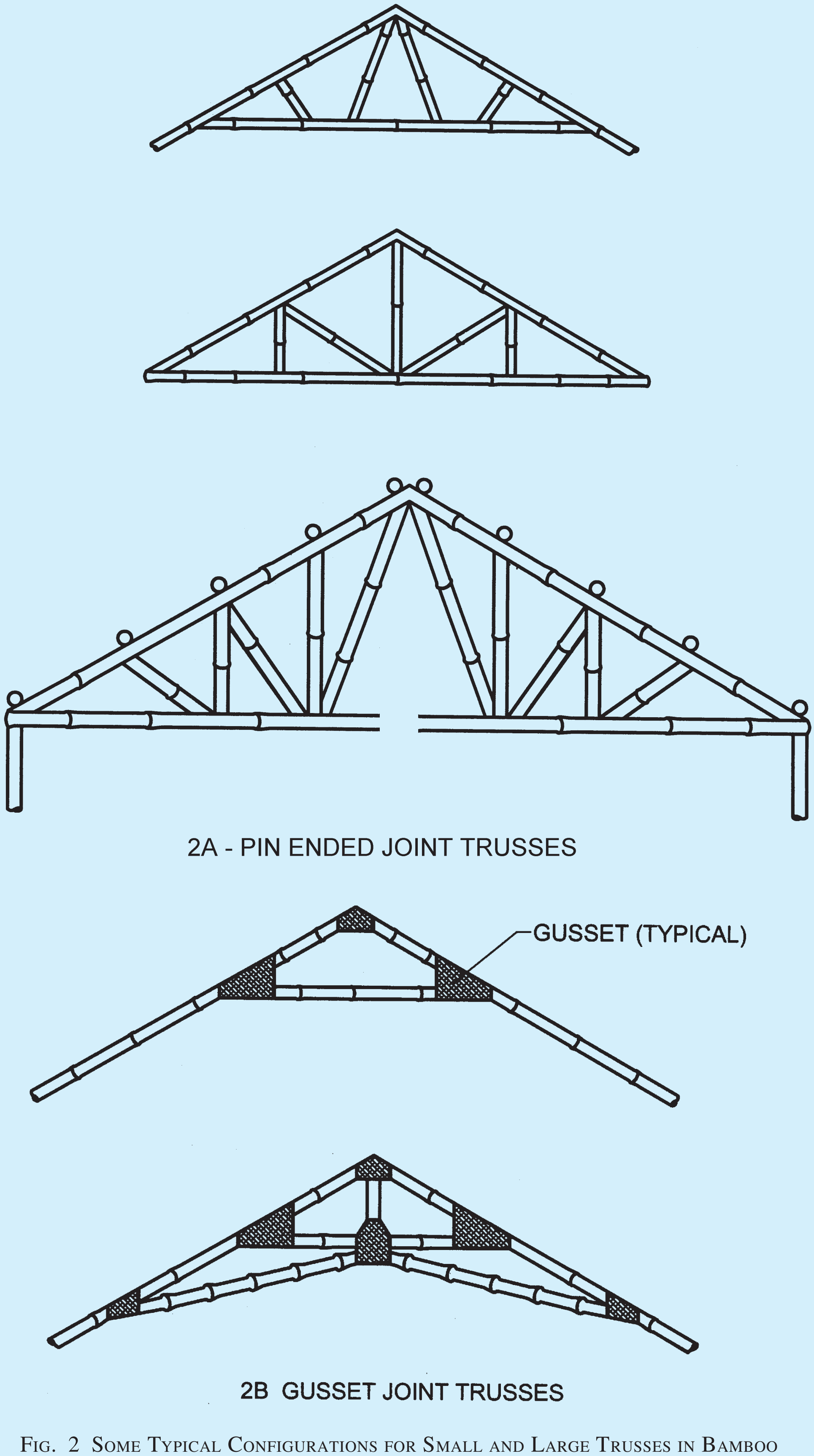


FIG. 2 SOME TYPICAL CONFIGURATIONS FOR SMALL AND LARGE TRUSSES IN BAMBOO

7.2 Bamboo Joints

Efficient jointing is basic to the structural adequacy of a framed construction, may it be of any cellulosic material. Round, tubular form of bamboo requires an approach different to that used for sawn timber. 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.

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.

7.2.1.1 *Lengthening joints* (*end jointing*)

7.2.1.1.1 *Lap joint*

In this case, 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. In half lapping, culms shall preferably be of similar diameter and cut longitudinally to half depth over at least one internode length and fastened as per full lap joint (*see* Fig. 3).

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. This joint transfers both compressive and tensile forces equally well (*see* Fig. 4).

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 (*see* Fig. 5).

7.2.1.1.4 *Scarf joints*

A scarf joint is formed by cutting a sloping plane 1 in 4 to 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. Using hooked splays adds to the strength and proper location of joints (*see* Fig. 6).

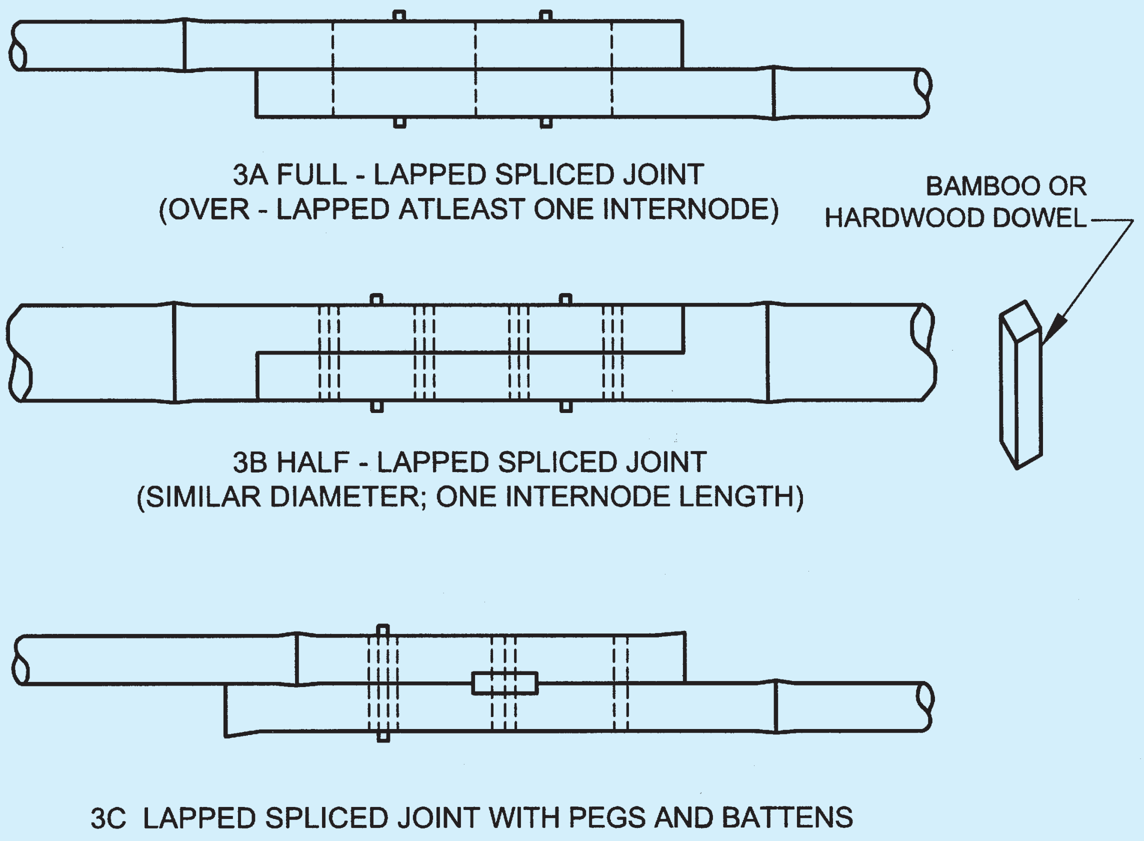


Fig. 3 Lap Joints in Bamboo

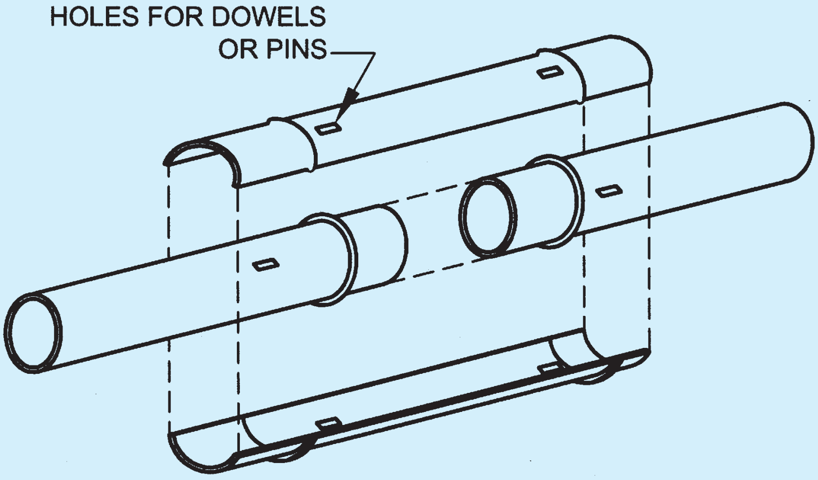


FIG. 4 BUTT JOINT WITH SIDE PLATES IN BAMBOO

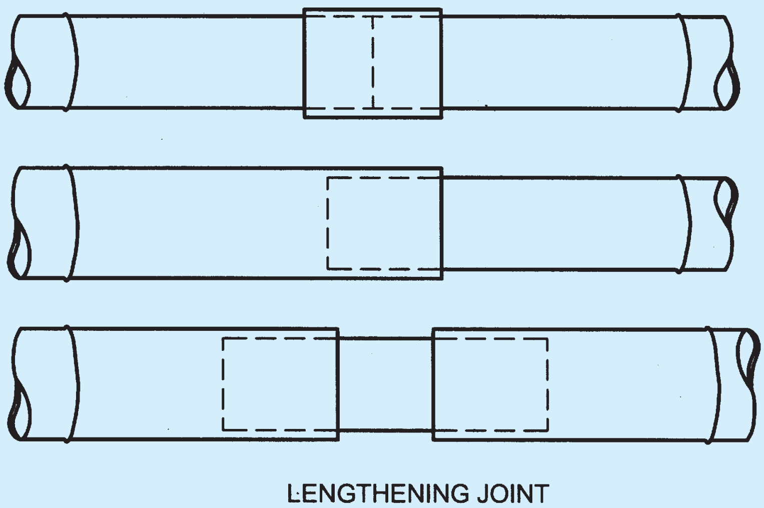


FIG. 5 SLEEVES AND INSERTS FOR BAMBOO JOINTS

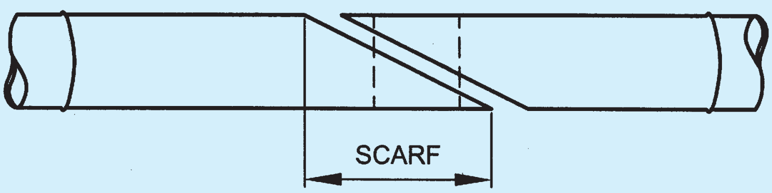


FIG. 6 SCARF JOINT

7.2.1.2 *Bearing joints*

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 *Butt 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 (*see* Fig. 7).

7.2.1.2.2 *Tenon joint*

It is formed by cutting a projection (tenon) in walls of one piece of bamboo and filling it into corresponding holes (mortise) in another and keyed. It is a neat and versatile joint for maximum strength and resistance to separation (*see* Fig. 8).

7.2.1.2.3 *Cross over joint*

It is formed when two or more members cross at right angles and its function is to locate the members and to provide lateral stability. In case of the joint connecting floor beam to post, it may be load bearing

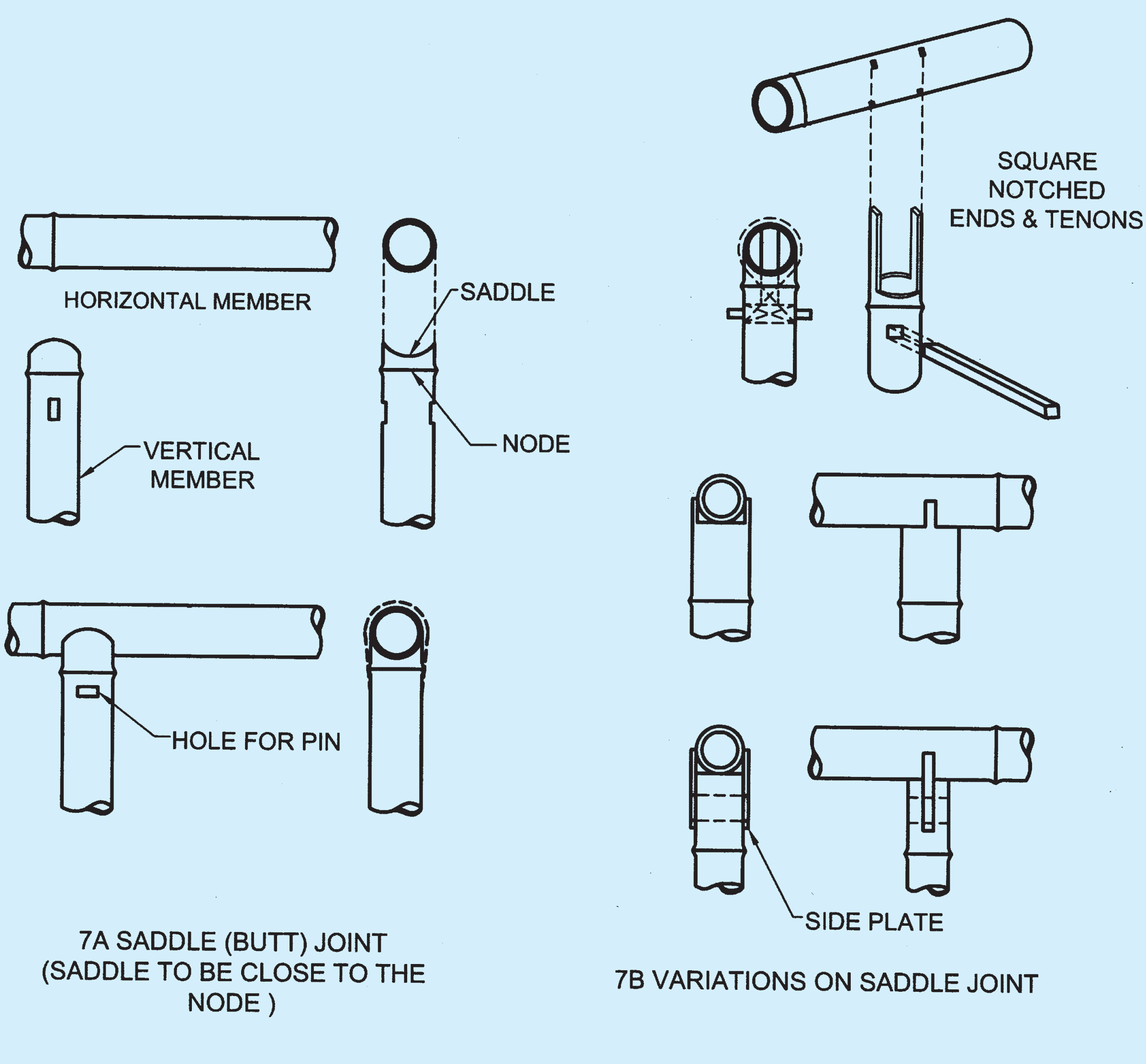


FIG. 7 BUTT JOINTS IN BAMBOO

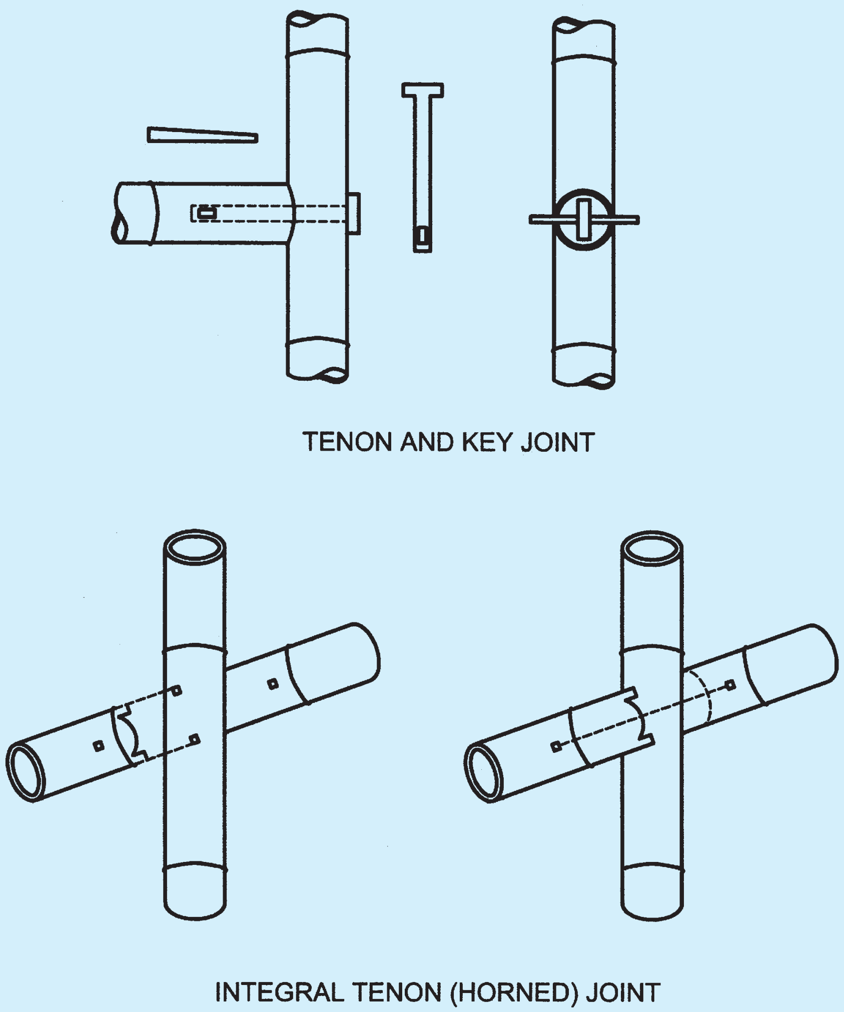


FIG. 8 TENON JOINT

(*see* Fig. 9). Such joints are also used to transmit angle thrust.

7.2.1.3 *Angled joints*

When two or more members meet or cross other than at right angles, angled joints are formed. For butt joints, the ends of the members may be shaped to fit in as saddle joints. Tenons would help in strengthening such joints (*see* Fig. 10).

7.2.2 *Modern Practices* (*see* Fig. 11)

Following are some of the modern practices for bamboo jointing:

a) 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 bolts. Hollow cavities of bamboo need to be stuffed with wooden plugs.

b) 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 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 increase strength and dowels pass right through the member, usually at right angles.

7.2.3.1 Normally 1.60 mm diameter galvanized iron wire may be used for tight lashing.

7.2.3.2 *Wire bound joints*

Usually galvanized iron 2.00 mm 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.

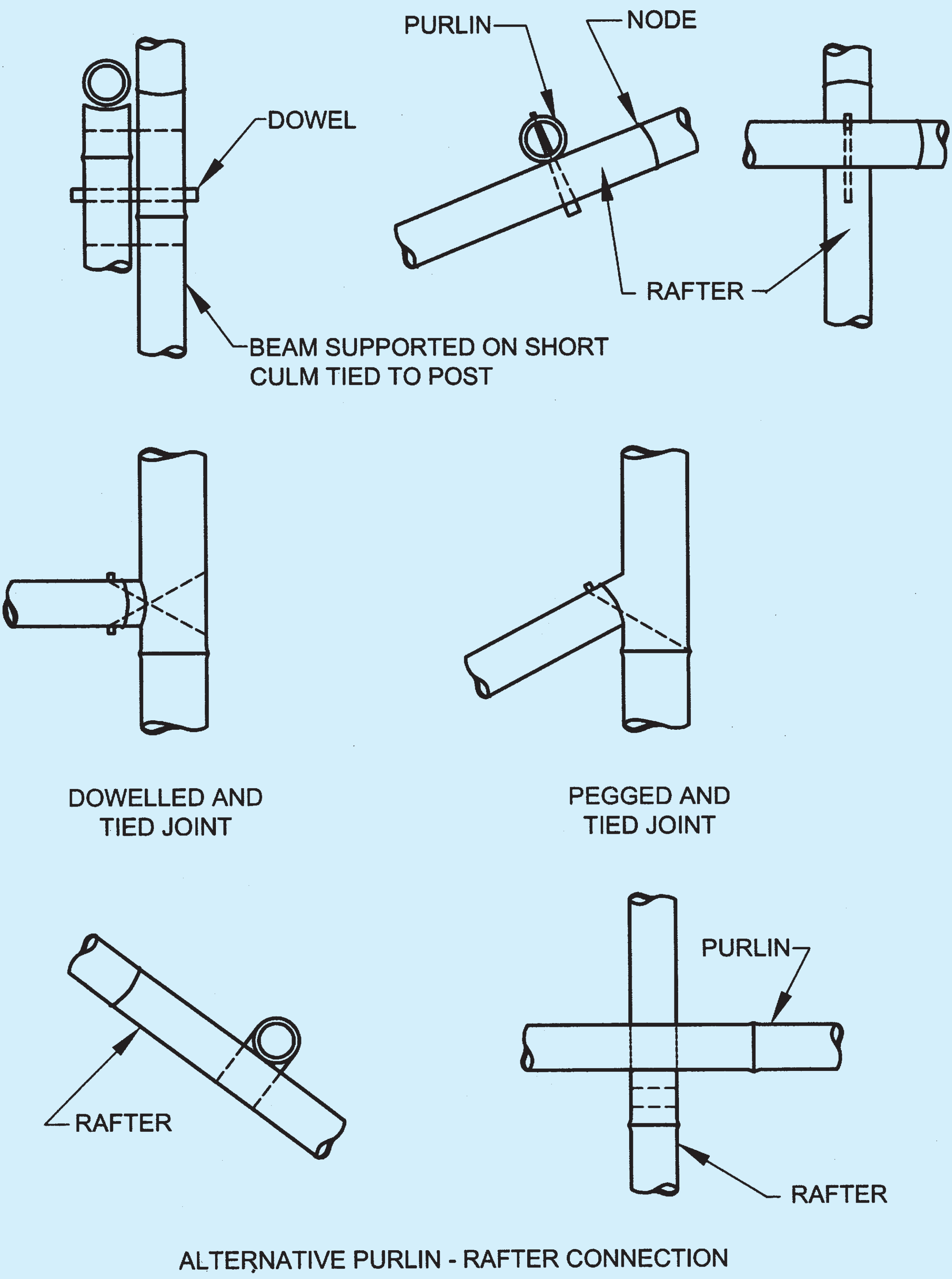


FIG. 9 CROSS OVER JOINTS (BEARING JOINTS)

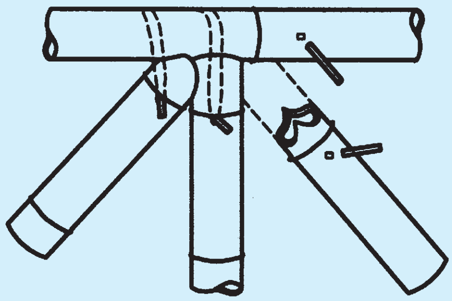


FIG. 10 ANGLED JOINT WITH INTEGRAL TENONS

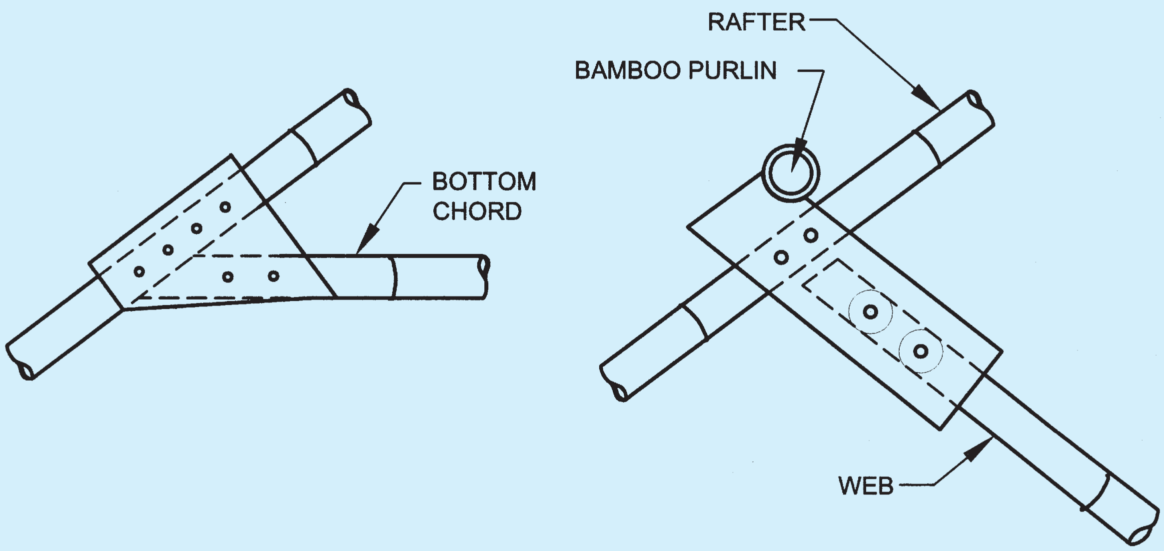


FIG. 11 GUSSET PLATED JOINTS

7.2.3.3 *Pin and wire bound joints*

Generally 12 mm dia bamboo pins are fastened to culms and bound by 2.00 mm diameter galvanized iron wire.

7.2.3.4 *Fish plates/gusset plated joints*

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.

7.2.3.5 *Horned joints*

Two tongues made at one end of culm may be fastened with a cross member with its mortise grooves to receive horns, the assembly being wire bound.

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 disaster high wind and seismic areas, good construction practice shall be followed taking care of joints, their damping and possible ductility. Bracings in walls shall be taken care of in bamboo structures.

8 STORAGE OF BAMBOO

Procurement and storage of bamboo stocks are essential for any project work and shall be done in accordance with Part 7 'Constructional Practices and Safety'.

ANNEX A

(*Foreword*)

SOURCE AND LOCAL NAMES OF SOME OF THE SPECIES OF BAMBOO

Sl No.	*Species*	*Source/Local Names*
(1)	(2)	(3)
1.	*Bambusa auriculata*	Assam, Bangladesh, Myanmar; introduced in Calcutta Botanic Garden.
2.	*B. balcooa*	Asm — Baluka; Ben — Balku bans; Duars — Bora bans; Garo — Wamnah, beru; Tripura — Barak.
3.	*B. bambos (Syn. B. arundinacea)*	Asm — Kotoba; Ben — Baroowa, behor; ketuas; ketwa Manip — Saneibi; Mah — Katang bamboo, oowga; Oriya — Daba, katuig; Tel — Mulkas veduru, Mullu vedurn; English — Spiny bamboo.
4.	*B. burmanica*	Asm — Thaikawa.
5.	*B. multiplex* *Syn. B. glancescens (Syn. B. nana)*	Sans — keu-fa; Burmese — Pa-lau-pinan-wa; Malay — Bamboo tjeenah; China — Bamboo hower tjeenah.
6.	*B. nutans*	Assam — 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 (Banglaesh) — Peechli; Tripura — Kali
7.	*B. pallida*	Asm — Bijli, jowa, makal, walkthai; Cachar — Bakhal, burwal; Khasi — Seskien, skhen, ineng, usker; Lepcha — Pashipo, pshi, pushee; Mikir — Loto; Naga — Tesero, watoi; Tripura — Makal.
8.	*B. polymorpha*	Asm — Jama betwa, betwa; Ben — Batua, jaibarouwa, jama; Burma — Kyathaung-wa; MP-Korku — Narangi bhas; Tripura — Basi.
9.	*B. tulda*	Asm — Wamunna, wagi, nal-bans; Beng — Tulda, jowa; Duars — Karanti, matela; Garo — Watti; Hin — Peka; Kamrup — Bijuli, jati, joo, ghora; Tripura — Mirtinga

(1)	(2)	(3)
10.	*B. vulgaris*	Ben and Manipuri — Bakal; Oriya — Sunarkania bans.
11.	*B. Wamin Syn. B. ventricosa* *(Syn. B. Vulgaris var. Wamin)*	Common name — Pitcher bamboo.
12.	*Cephalostachyum pergacile*	MP — Bhalan bans; Manipuri — Wootang; Naga — Latang; Oriya — Darrgi.
13.	*Dendrocalamus giganteous*	English — Giant Bamboo; Asm — Worra; Manipuri — Maroobeb.
14.	*D. hamiltonii*	Nep — Tamo; Asm — Kokwa; Tripura — Pecha.
15.	*D. longispathus*	Tripura — Rupai.
16.	*D. membranaceus*	Native of Myanmar; introduced in Kerala.
17.	*D. strictus*	English — Male bamboo; Ben — Karail; Guj — Nakur bans; Kan — Kiri bidiru; Mah — Male bamboo, nanvel; Oriya — Salia; Tam — Kalmungil; Tel — Sadanapa vedur; Tripura — Lathi bans; Hin — Bans Kaban, nav bans;
18.	*Melocanna baccifera*	Asm — Tarai; Ben — Muli; Cachar — Wati; Garo — Watrai; Manipuri — Moubi; Mikir — Artem; Naga — Turiah.
19.	*Oxytenanthera abyssinicia*	Native of tropical Africa; cultivated at FRI, Dehra Dun.
20.	*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
MP	—	Madhya Pradesh
Nep	—	Nepali
Sans	—	Sanskrit

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.

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 Code. The latest version of a standard shall be adopted at the time of enforcement of the Code. The standards listed may be used by the Authority as a guide in conformance with the requirements of the referred clauses in the Code.

	IS No.	*Title*
(1)	6874 : 1973	Method of test for round bamboo
(2)	9096 : 1979	Code of practice for preservation of bamboo for structural purposes
(3)	6874 : 1973	Method of test for round bamboo
	8242 : 1976	Method of test for split bamboo

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 4 Masonry

BUREAU OF INDIAN STANDARDS

C O N T E N T S

National Building Code Sectional Committee, CED 46

FOREWORD

This Section primarily covers the structural design of unreinforced masonry elements in buildings. However, provisions on reinforced brick and reinforced brick concrete floors and roofs have also been included.

This Section was first published in 1970 and revised in 1983. Subsequently 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 this 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.

As a result of experience gained in the implementation of this Section and feedback received, as well as in view of revision of IS 4326 'Code of practice for earthquake resistant design and construction of buildings' and formulation of some new standards in this field, a need to revise this Section has been felt. This revision has, therefore, been prepared to take care of these aspects. The significant changes incorporated in this revision include the following:

a) The provision of special considerations in earthquake zones have been aligned in line with IS 4326 : 1993.
b) A new clause covering guidelines for improving earthquake resistance of low strength masonry buildings has been added.
c) Reference to design of reinforced brick and reinforced brick concrete floors and roofs has been included.
d) Reference to all the concerned Indian Standards have been updated.

Structural design requirements of this Section are based on IS 1905 : 1987 'Code of practice for structural use of unreinforced masony (*third revision*)' and IS 4326 : 1993 'Code of practice for earthquake resistant design and construction of buildings (*second revision*)'.

A reference to SP 20 : 1991 'Handbook on masonry design and construction (*first revision*)' may be useful.

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.

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 4 Masonry

1 SCOPE

1.1 This Section primarily covers the structural design aspects of unreinforced load bearing and non-load bearing walls, constructed with masonry units permitted in accordance with this Section. This, however, also covers provisions for design of reinforced brick and reinforced brick concrete floors and roofs. It also covers guidelines regarding earthquake resistance of low strength masonry buildings.

1.2 The recommendations of the Section do not apply to walls constructed in mud mortars.

2 TERMINOLOGY

2.1 For the purpose of this Section, the following definitions shall apply.

2.1.1 *Bed Block* — A block bedded on a wall, column or pier to disperse a concentrated load on a masonry element.

2.1.2 *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.

2.1.3 *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.

2.1.4 *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.

2.1.5 *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).

2.1.6 *Effective Height* — The height of a wall or column, to be considered for calculating slenderness ratio.

2.1.7 *Effective Length* — The length of a wall to be considered for calculating slenderness ratio.

2.1.8 *Effective Thickness* — The thickness of a wall or column to be considered for calculating slenderness ratio.

2.1.9 *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.

2.1.10 *Grout* — Mortar of pourable consistency.

2.1.11 *Joint* — A junction of masonry units.

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.
c) *Wall joint* — A vertical joint parallel to the face of the wall.

2.1.12 *Leaf* — Inner or outer section of a cavity wall.

2.1.13 *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.

2.1.14 *Load Bearing Wall* — A wall designed to carry an imposed vertical load in addition to its own weight, together with any lateral load.

2.1.15 *Masonry* — An assemblage of masonry units properly bonded together with mortar.

2.1.16 *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.

2.1.17 *Partition Wall* — An interior non-load bearing wall, one storey or part storey in height.

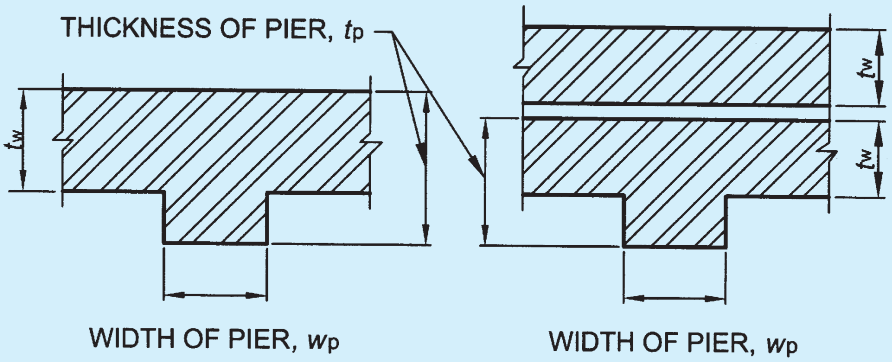


FIG. 1 DEFINITION OF PIER

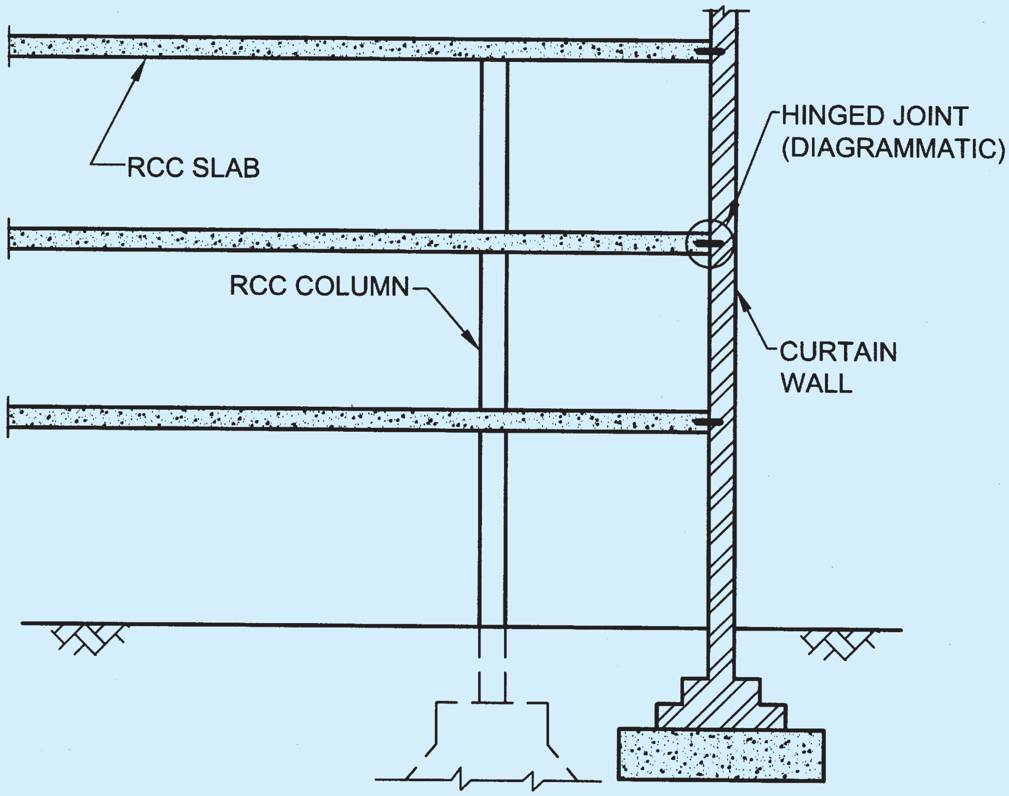


FIG. 2 MASONRY CURTAIN WALL

2.1.18 *Panel Wall* — An exterior non-load bearing wall in framed construction, wholly supported at each storey but subjected to lateral loads.

2.1.19 *Shear Wall* — A wall designed to carry horizontal forces acting in its plane with or without vertical imposed loads.

2.1.20 *Slenderness Ratio* — Ratio of effective height or effective length to effective thickness of a masonry element.

2.1.21 *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).

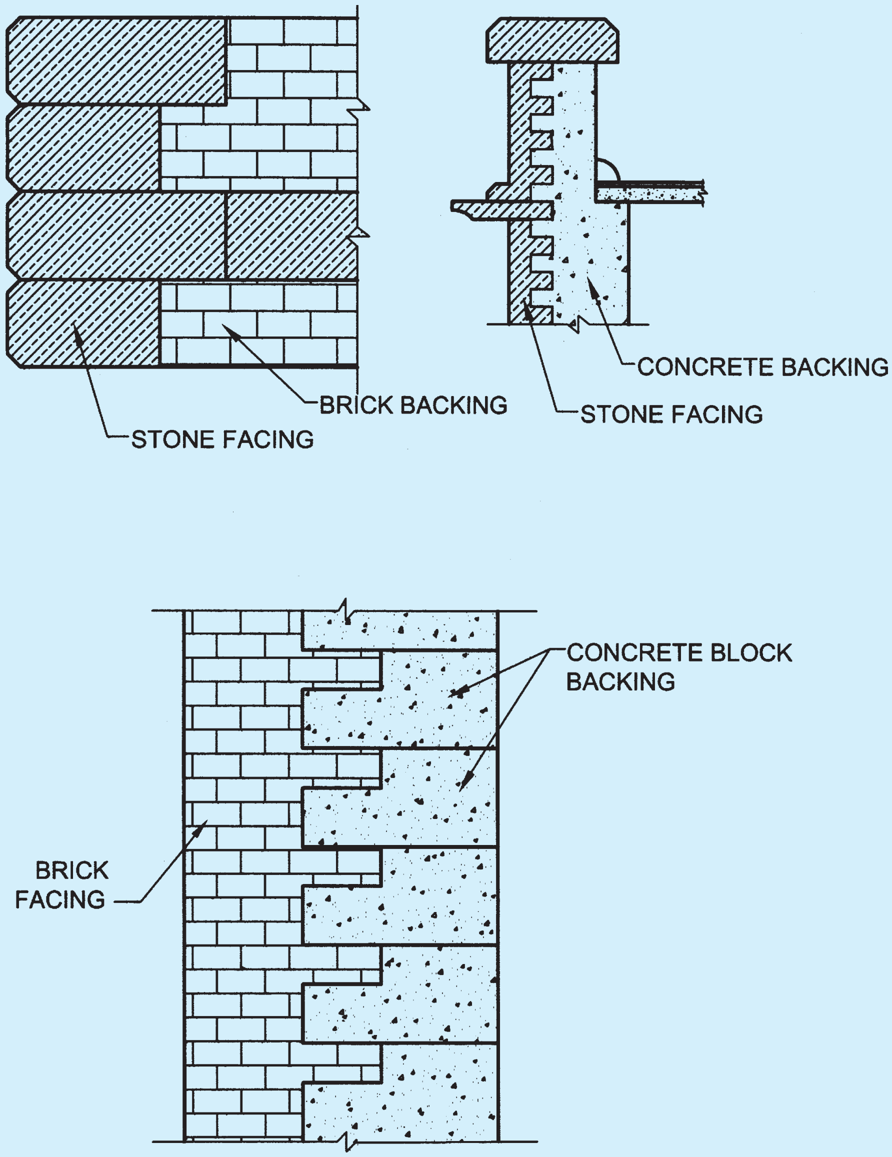


FIG. 3 TYPICAL FACED WALL

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.

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.

3 MATERIALS

3.1 General

The materials used in masonry construction shall be in accordance with Part 5 'Building Materials'.

3.2 Masonry Units

Masonry units used in construction shall conform to accepted standards [6-4(1)].

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) Stones (in regular sized units),
e) Sand-lime bricks,
f) Concrete blocks (solid and hollow),
g) Lime based blocks,
h) Burnt clay hollow blocks,
j) Gypsum partition blocks,
k) Autoclaved cellular concrete blocks, and
m) 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, precise stone blocks, fly-ash-lime-gypsum bricks, stabilized mud 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 re-used 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 [6-4(2)].

3.3.1 Mix proportions and compressive strengths of some of the commonly used mortars are given in Table 1.

Table 1 Mix Proportions and Strength of Mortars for Masonry

(*Clause* 3.3.1)

Sl No.	Grade of Mortar	Mix Proportions (by Loose Volume)					Minimum Compressive Strength at 28 Days in N/mm²
		Cement	Lime	Lime Pozzolana Mixture	Pozzolana	Sand	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	H1	1	¼ C or B	0	0	3	10
2(a)	H2	1	¼ C or B	0	0	4	7.5
2(b)		1	½ C or B	0	0	4½	6.0
3(a)	M1	1		0	0	5	5.0
3(b)		1	1 C or B	0	0	6	3.0
3(c)		0	0	1 (LP-40)	0	1½	3.0
4(a)	M2	1	0	0	0	6	3.0
4(b)		1	2 B	0	0	9	2.0
4(c)		0	1 A	0	0	2	2.0
4(d)		0	1 B	0	1	1	2.0
4(e)		0	1 C or B	0	2	0	2.0
4(f)		0	0	1 (LP-40)	0	1¾	2.0
5(a)	M3	1	0	0	0	7	1.5
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
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)	L1	1	0	0	0	8	0.7
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	2¼	0.7
6(e)		0	0	1 (LP-20)	0	1½	0.7
7(a)	L2	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	1½	0.5

NOTES

1 Sand for making mortar should be well graded. In case sand is not well graded, its proportion shall be reduced in order to achieve the minimum specified strength.

2 For mixes in Sl No. 1 and 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 it 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 volume) 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), 4(e), 5(d), 5(e), 6(b), 6(c), 7(a) and 7(b) are prepared by grinding in a mortar mill.

6 Mix in Sl No. 2(b) has been classified to be of same grade as that of Sl No. 2(a), mixes in Sl No. 3(b) and 3(c) same as that in Sl No. 3(a), mixes in Sl No. 4(b) to 4(f) same as that in Sl No. 4(a), even though their compressive strength is less. This is from consideration of strength of masonry using different mix proportions.

7 A, B and C denote eminently hydraulic lime, semi-hydraulic lime and fat lime respectively, as specified in appropriate standards listed in Part 5 'Building Materials'.

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 multi-storey 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.

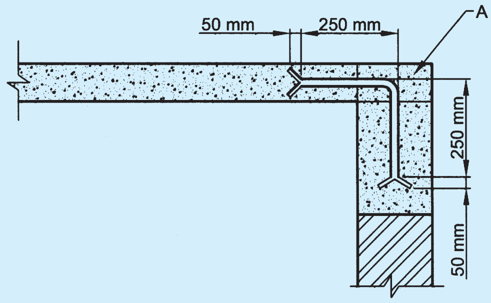
4.2.1.1 Lateral support may be in the vertical or horizontal direction, the former consisting of floor/roof bearing on the wall or properly anchored to the same and latter consisting of cross walls, piers or buttresses.

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. 4; and



A = Cement concrete only at places where anchors are provided (200 mm in width in the direction perpendicular to the plane of paper)

FIG. 4 ANCHORING OF RCC SLAB WITH MASONRY WALL (WHEN SLAB DOES NOT BEAR ON WALL)

   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. 5 and Fig. 6. The

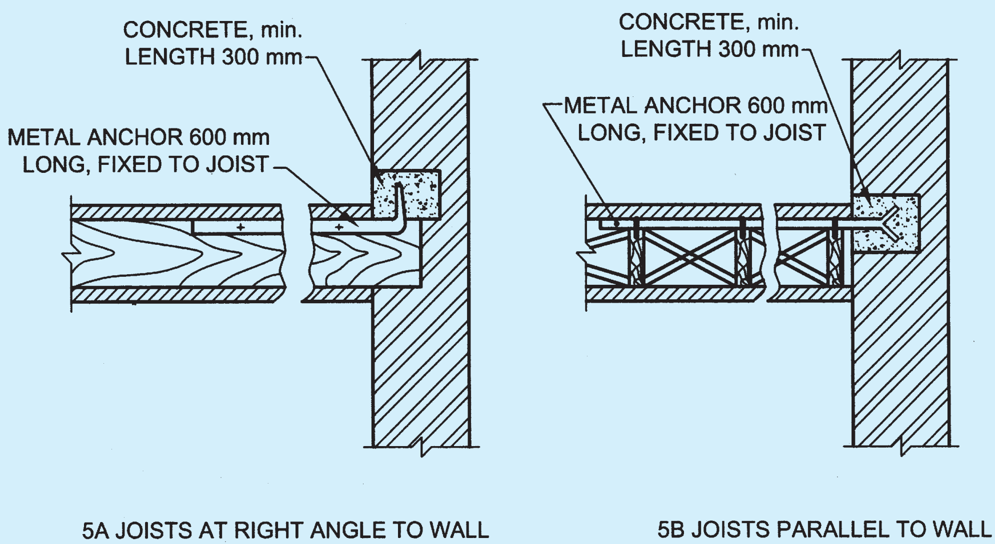


FIG. 5 TYPICAL DETAILS FOR ANCHORAGE OF SOLID WALLS

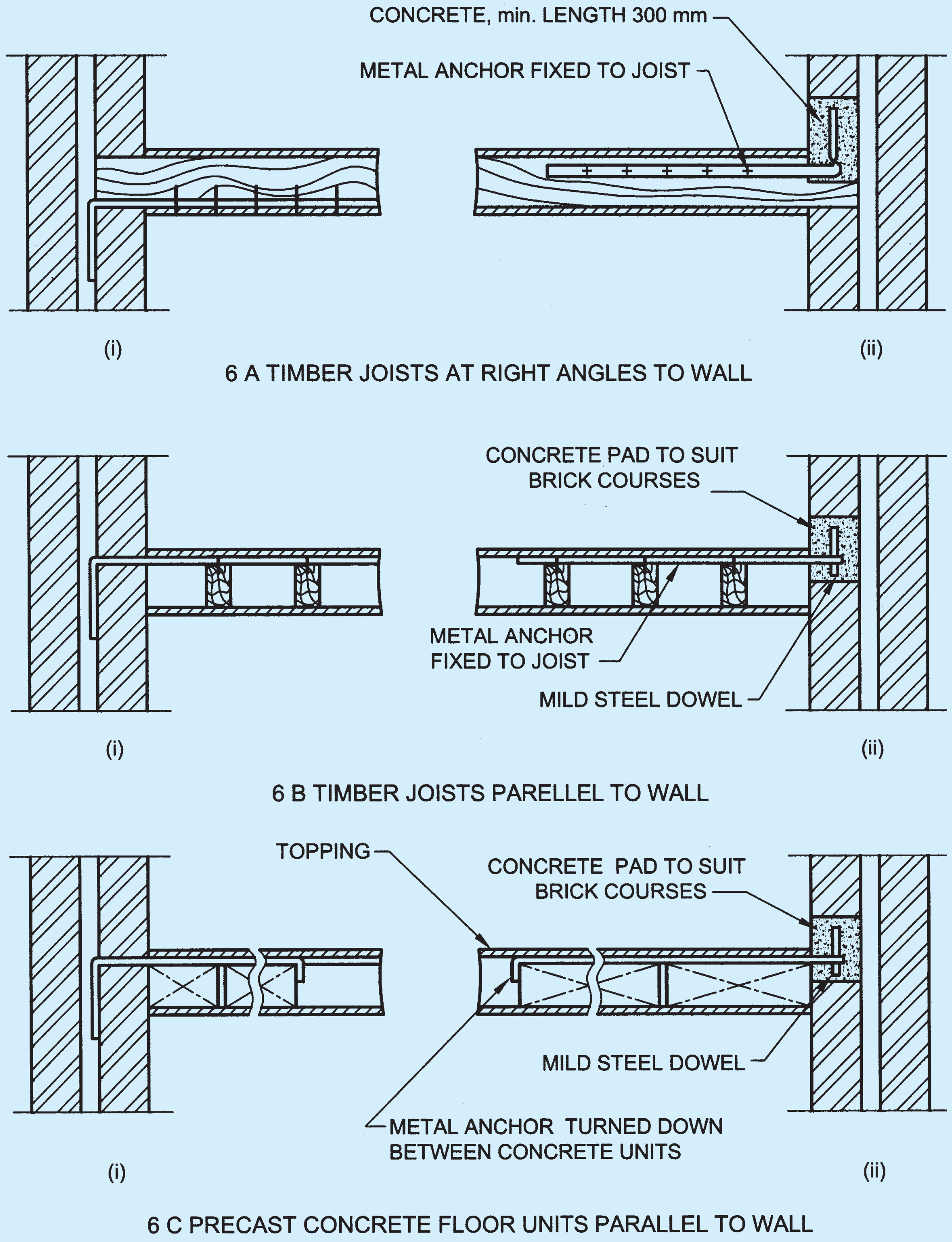


FIG. 6 TYPICAL DETAILS FOR ANCHORAGE OF CAVITY WALLS

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. 7) and bonded to it according to provision of **4.2.2.2** (d);

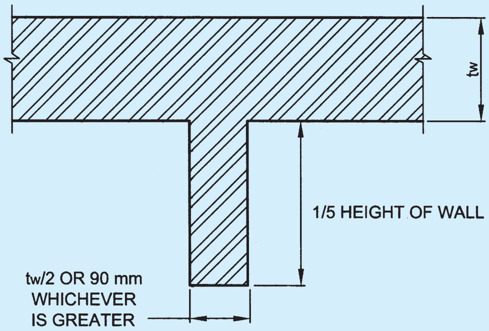


FIG. 7 MINIMUM DIMENSION FOR MASONRY WALL OR BUTTRESS EFFECTIVE LATERAL SUPPORT

c) In case of a column, an RCC or timber beam/R S 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.

4.2.2 *Stability*

A wall or column subject 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 2 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. 8;

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

Table 2 Thickness and Spacing of Stiffening Walls

[*Clause* 4.2.2.2(b)]

Sl No.	Thickness of Load Bearing Wall to be Stiffened	Height [1] of Storey Not to Exceed	Stiffening Wall [1]		
			Thickness not less than		Maximum Spacing
			1 to 3 storeys	4 to 6 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.

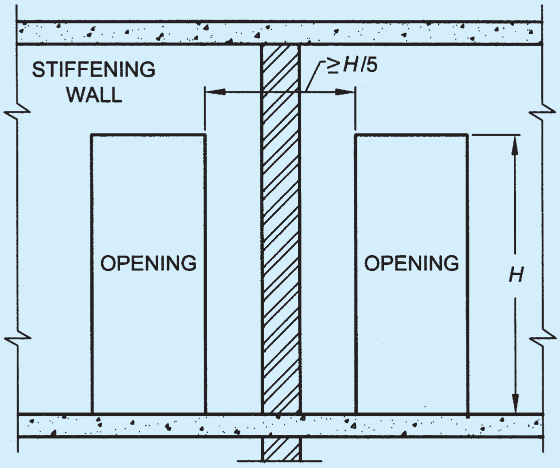


FIG. 8 OPENING IN STIFFENING WALL

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. 9).

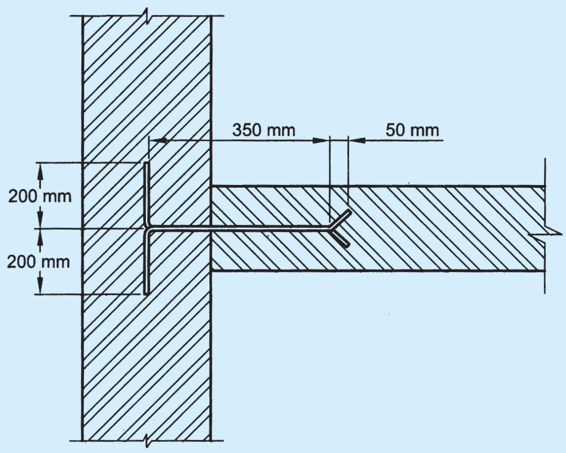


FIG. 9 ANCHORING OF STIFFENING WALL WITH SUPPORT WALL

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 12 t or H/6, whichever is less in case of T/I shaped walls, and 6 t 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. 10.

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 3.

NOTE — In case there is surcharge on basement walls from adjoining buildings, thickness of basement walls shall be based on structural analysis.

Table 3 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 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

4.2.2.7 *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

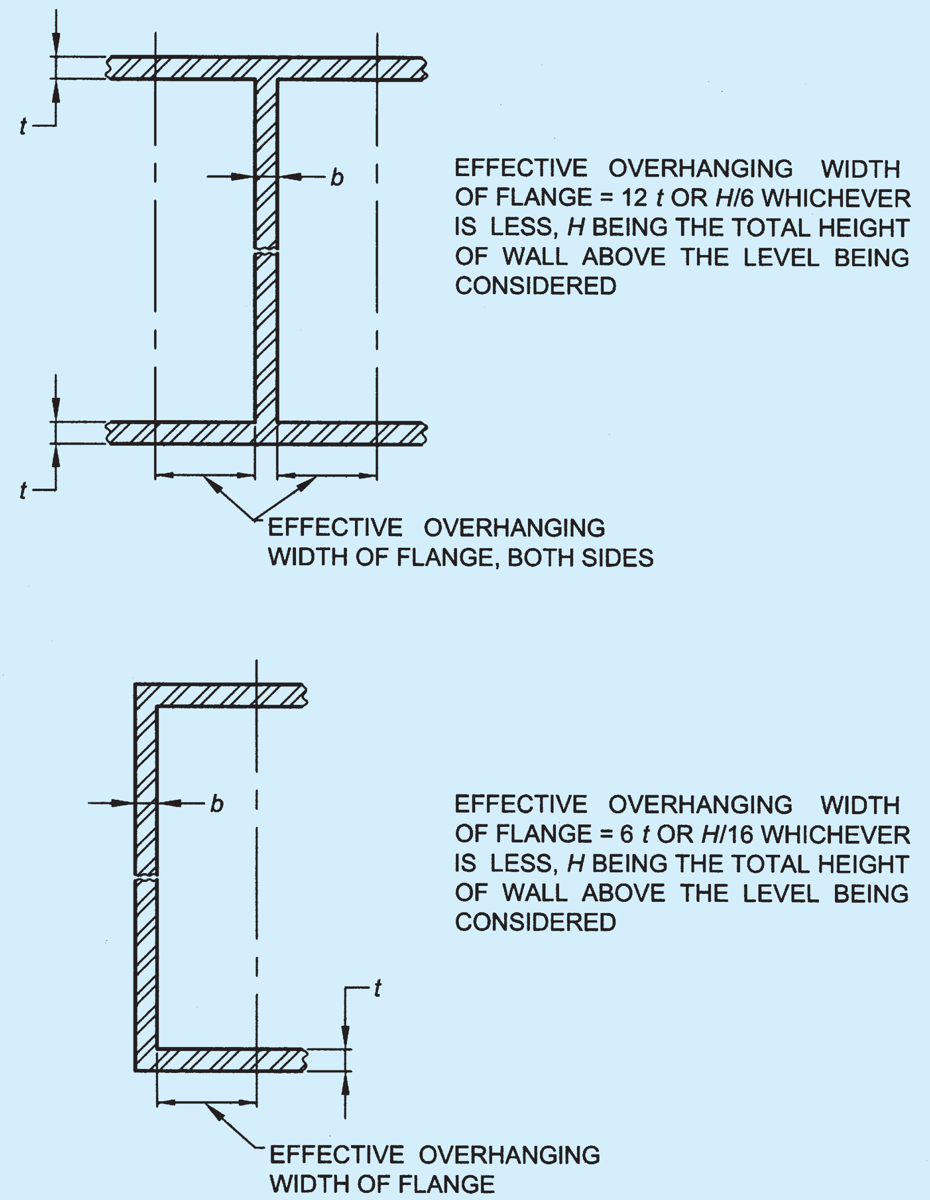


FIG. 10 TYPICAL DETAILS FOR ANCHORAGE OF SOLID WALLS

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.

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**).

4.3 Effective Height

4.3.1 *Wall*

Effective height of a wall shall be taken as shown in Table 4 (*see* Fig. 11).

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.

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 twice the actual height for the direction it is not laterally supported (*see* Fig. 12).

Table 4 Effective Height of Walls

(*Clause* 4.3.1)

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 9 cm), 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*

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. 11.

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 memberane 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*.

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.

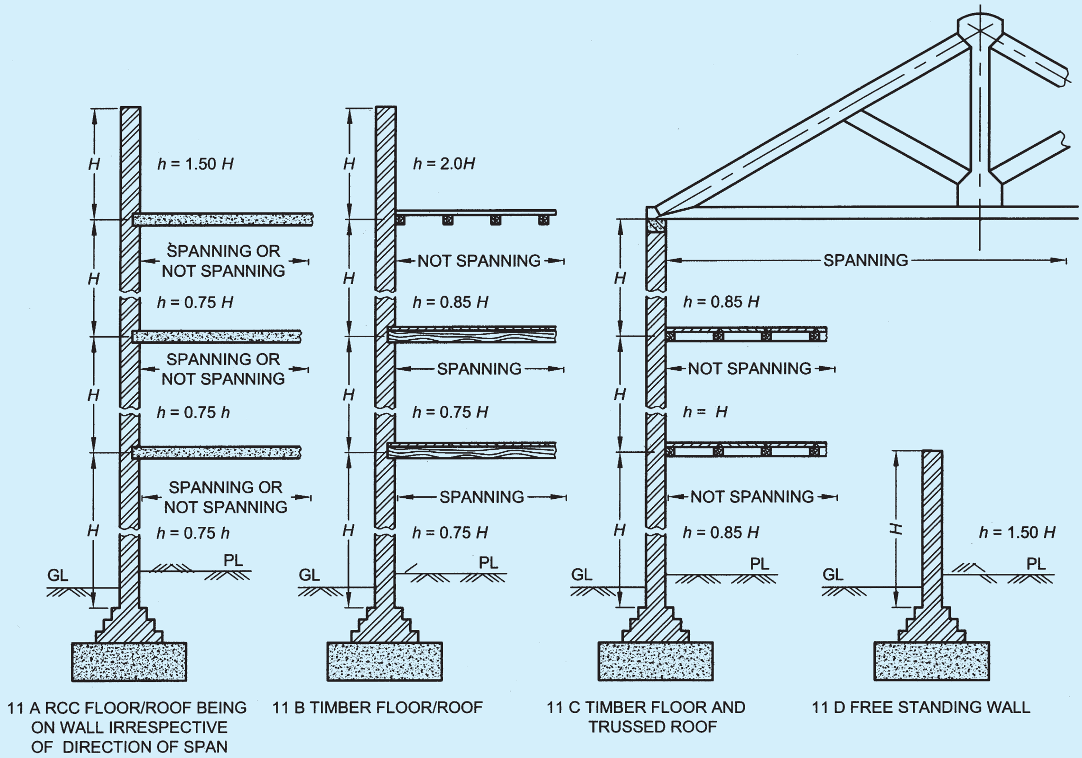
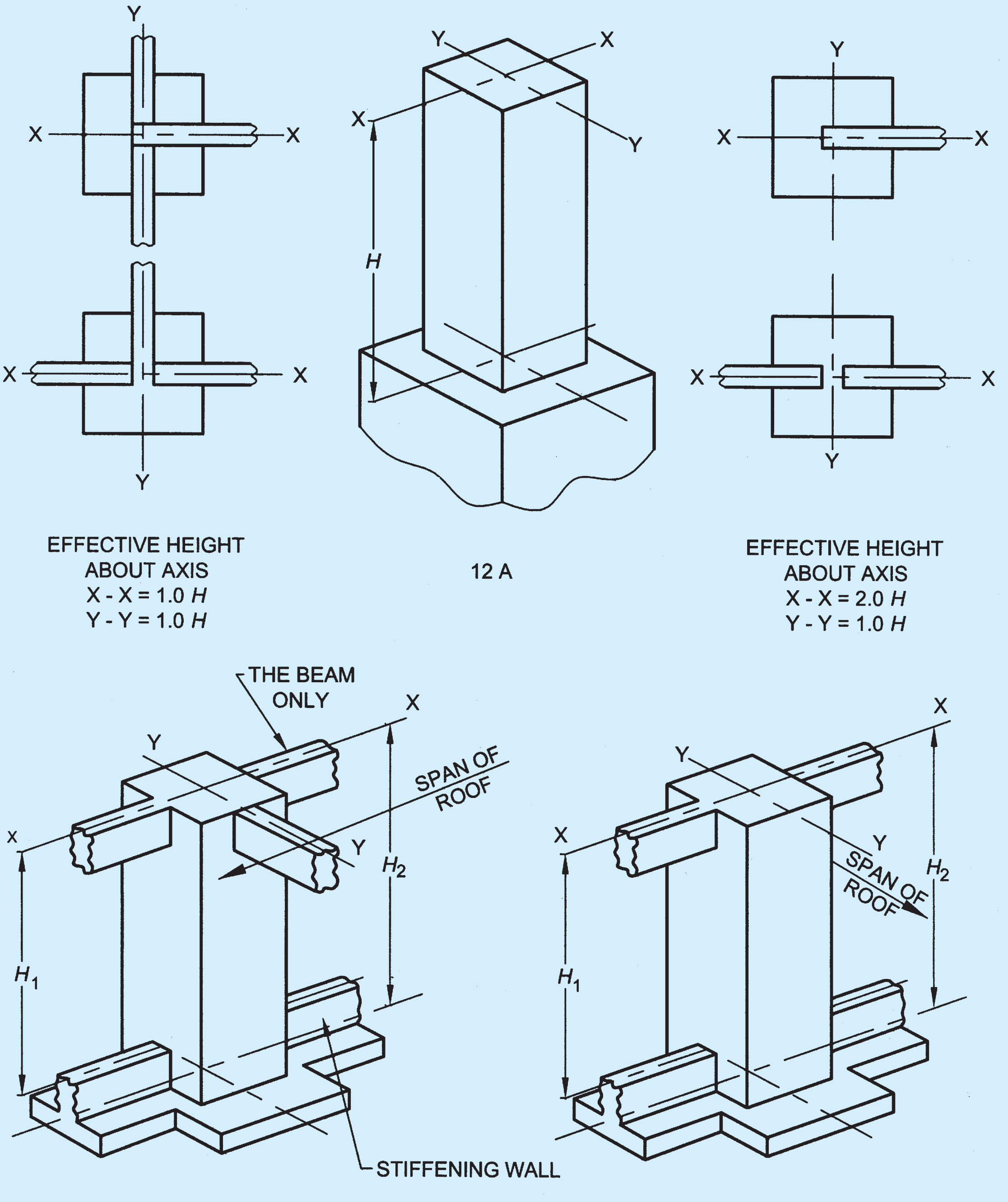


FIG. 11 EFFECTIVE HEIGHT OF WALL



Roof Construction	*Effective Height About Axis Fig. 12B*	*Effective Height About Axis Fig. 12C*
With precast concrete units of *in-situ* concrete floor or roof	$X - X = 1.0\ H_2$ $Y - Y = 1.0\ H_1$ $Y - Y = 1.5\ H_1$ (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. 12 EXAMPLE OF EFFECTIVE HEIGHT OF COLUMNS

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 5.

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 6. No modification in effective thickness, however, shall be made when slenderness ratio is to be based on effective length of walls.

4.5.3 For solid walls or faced walls stiffened by cross walls, appropriate stiffening coefficient may be

Table 5 Effective Length of Walls

(*Clause* 4.4)

Sl No.	Conditions of Support (*See* Fig. 13)	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. 13) 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 **4.2.1.3**(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)	2.0 L

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.

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 **4.2.2.1**(d).

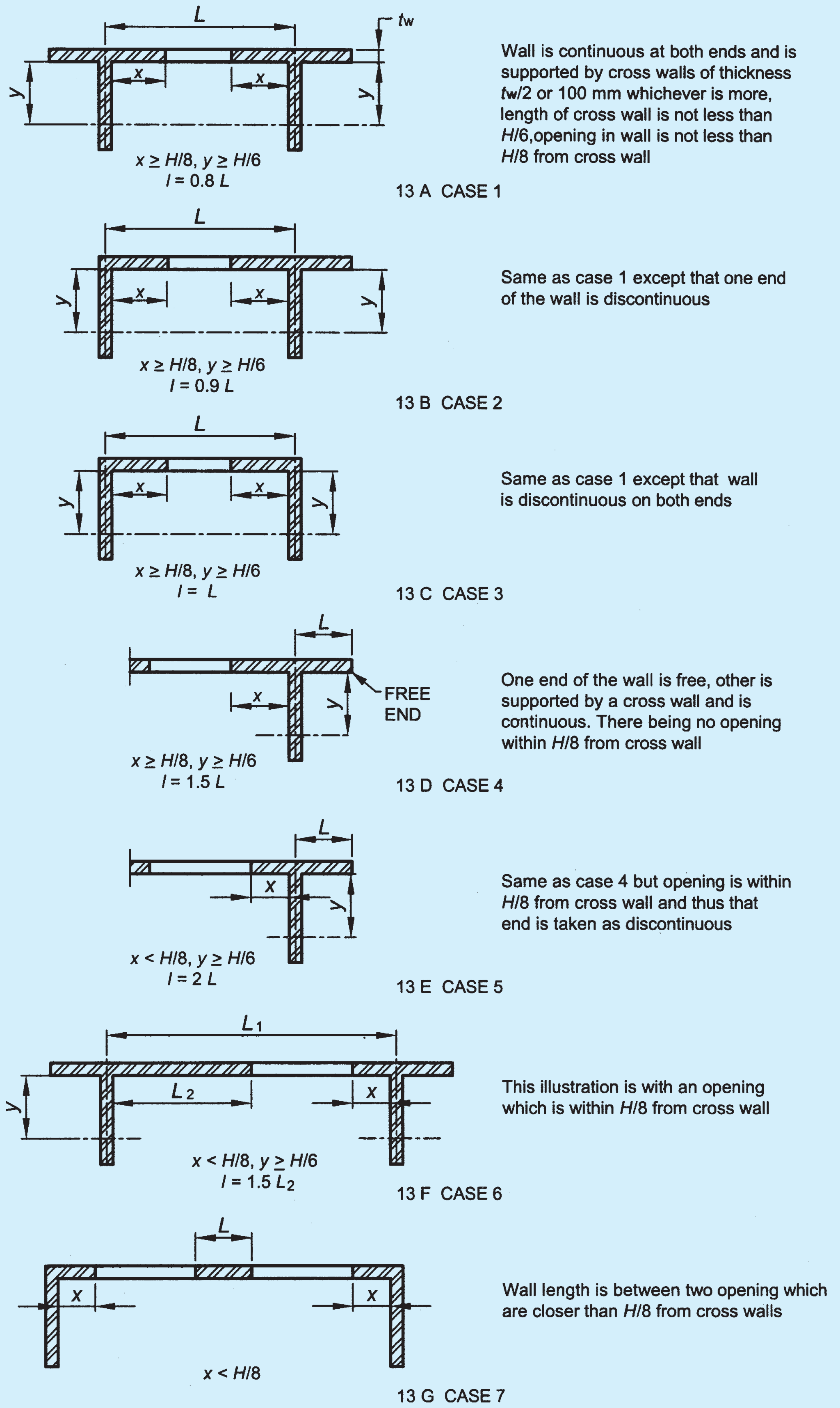


Fig. 13 Effective Length of Wall

determined from Table 6 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.

Table 6 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 **2.3.2** (*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.

4.5.4 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.

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 7.

Table 7 Maximum Slenderness Ratio for a Load Bearing Wall

(*Clause* 4.6.1)

Number of Storeys	Maximum Slenderness Ratio	
	Using Portland Cement or Portland Pozzolana Cement in Mortar	Using Lime Mortar
(1)	(2)	(3)
Not exceeding 2	27	20
Exceeding 2	27	13

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 under any particular circumstances has, therefore, to be decided according to the best judgement of the designer. Some guidelines for assessment of eccentricity are given in Annex A.

5 STRUCTURAL DESIGN

5.1 General

The building as a whole shall be analyzed 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) live 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 judgement of the designer (*see also* Part 6 'Structural Design, Section 1 Loads, Forces and Effects').

5.2.1 The design loads and other forces to be taken for the design of masonry structures shall conform to Part 6 'Structural Design, Section 1 Loads, Forces and Effects'.

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 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 8 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 8 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.

5.4.1.1 *Stress reduction factor*

This factor, as given in Table 9, 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 m^2. The factor $k_a = 0.7 + 1.5A$, A being the area of section in m^2.

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 10. This factor is applicable for units for crushing strength up to 15 N/mm^2.

Table 8 Basic Compressive Stresses for Masonry (After 28 days)

(*Clauses* 5.4.1 *and* 6.3.1)

Sl No.	Mortar Type (Ref Table 1)	Basic Compressive Stresses in N/mm^2 Corresponding to Masonry Units of which Height to Width Ratio does not Exceed 0.75 and Crushing Strength, in N/mm^2, 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 mud block having thickness 100 mm or more, reference to specialist literature may be made.

Table 9 Stress Reduction Factor for Slenderness Ratio and Eccentricity
(*Clause* 5.4.1.1)

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)
6	1.00	1.00	1.00	1.00	1.00	1.00
8	0.95	0.95	0.94	0.93	0.92	0.91
10	0.89	0.88	0.87	0.85	0.83	0.81
12	0.84	0.83	0.81	0.78	0.75	0.72
14	0.78	0.76	0.74	0.70	0.66	0.66
16	0.73	0.71	0.68	0.63	0.58	0.53
18	0.67	0.64	0.61	0.55	0.49	0.43
20	0.62	0.59	0.55	0.48	0.41	0.34
22	0.56	0.52	0.48	0.40	0.32	0.24
24	0.51	0.47	0.42	0.33	0.24	—
26	0.45	0.40	0.35	0.25	—	—
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.

Table 10 Shape Modification Factor for Masonry Units
(*Clause* 5.4.1.3)

Height to Width Ratio of Units (as Laid)	Shape Modification Factor (k_p) for Units Having Crushing Strength in N/mm^2 is			
	5.0	7.5	10.0	15.0
(1)	(2)	(3)	(4)	(5)
Up to 0.75	1.0	1.0	1.0	1.0
1.0	1.2	1.1	1.1	1.0
1.5	1.5	1.3	1.2	1.1
2.0 to 4.0	1.8	1.5	1.3	1.2

NOTE — Linear interpolation between values is permissible.

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.
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 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 judgement of the designer. Some guidelines in this regard are given in Annex C.

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:

Grade M1 or better mortar – 0.07 N/mm² for bending in the vertical direction where tension developed is normal to bed joints.

– 0.14 N/mm² 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/mm².

Grade M2 mortar – 0.05 N/mm² for bending in the vertical direction where tension developed is normal to bed joints.

– 0.10 N/mm² 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/mm².

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/mm² for M1 mortar and 0.07 N/mm² for M2 mortar in case of boundry 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/mm²:

$$f_s = 0.1 + f_d/6$$

f_d = Compressive stress due to dead loads in N/mm², and

f_s = Permissible shear stress in N/mm².

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.

5.5 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.
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.
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.)
   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 11 provided the mortar used is of grade not leaner than M1.
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 11 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 11 reckoning the height from the lower level at which the wall is restrained laterally.

Table 11 Height to Thickness Ratio of Free Standing Walls Related to Wind Speed

(*Clause* 5.5.2.1)

Design Wind Pressure N/mm^2 (1)	Height to Thickness Ratio (2)
Up to 285	10
575	7
860	5
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.

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 [6-4(3)].

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*

No special provisions on construction are necessary for buildings constructed in Zones II. Special features of construction for earthquake resistant masonry buildings in Zones III, IV and V shall be applicable according to good practice [6-4(3)].

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 Code, 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 8 would hold good for commercially obtainable standards of workmanship with reasonable degree of supervision. If the work is inadequately supervised, strength should be reduced to three-fourth or less at the discretion of the designer.

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 relaid 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 [6-4(4)].

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.
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 [6-4(5)].

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.

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 a 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 SPECIAL CONSIDERATION IN EARTHQUAKE ZONES

7.0 Special features of design and construction for earthquake resistant masonry buildings are given in **7.2** to **7.8.2.** Reference may also be made to good practice [6-4(6)] for detailed information.

7.1 Categories of Buildings

For the purpose of specifying the earthquake resistant features in masonry and wooden buildings, the buildings have been categorized in five categories A to E based on the seismic zone and the importance of building *I*,

where

I = Importance factor applicable to the building (*see* Table 35 of Part 6, Section 1)

7.1.1 The building categories are given in Table 12.

Table 12 Building Categories for Earthquake Resisting Features

(*Clauses* 7.1.1 *and* 8.1.2)

Importance Factor	Seismic Zone			
	II	III	IV	V
(1)	(2)	(3)	(4)	(5)
1.0	A	B	C	D
1.5	B	C	D	E

7.2 Masonry Units

Bricks/Blocks as per the accepted standards [6-4(1)] having a crushing strength not less than 3.5 MPa shall be used. However, higher strength of masonry units may be required depending upon number of storeys and thickness of walls in accordance with provisions of this Section.

7.3 Mortar

7.3.1 Mortars, such as those given in Table 13 or of equivalent specification, shall preferably be used for masonry construction for various categories of buildings.

Table 13 Recommended Mortar Mixes
(*Clauses* 7.3.1 *and* 7.4.6)

Category of Construction[1]	Proportion of Cemet-Lime-Sand[2]
(1)	(2)
A	M2 (Cement-sand 1:6) or M3 (Lime-cinder [3] 1:3) or richer
B, C	M2 (Cement-lime-sand 1:2:9 or Cement-sand 1:6) or richer
D, E	H2 (Cement-sand 1:4) or M1 (Cement-lime-sand 1:1:6) or richer

NOTE — Though the equivalent mortar with lime will have less strength at 28 days, their strength after one year will be comparable to that of cement mortar.

[1] Category of construction is defined in Table 12.

[2] Mortar grades and specification for types of limes etc, shall be in accordance with Table 1.

[3] In this case some other pozzolanic material like *SURKHI* (burnt brick fine powder) may be used in place of cinder.

7.3.2 Where steel reinforcing bars are provided in masonry the bars shall be embedded with adequate cover in cement sand mortar not leaner than 1:3 (minimum clear cover 10 mm) or in cement concrete of grade M15 (minimum clear cover 15 mm or bar diameter whichever more), so as to achieve good bond and corrosion resistance.

7.4 Walls

7.4.1 Masonry bearing walls built in mortar, as specified in **7.3.1** unless rationally designed as reinforced masonry shall not be built of greater height than 15 m subject to a maximum of four storeys when measured from the mean ground level to the roof slab or ridge level. The masonry bearing walls shall be reinforced in accordance with **7.6.1**.

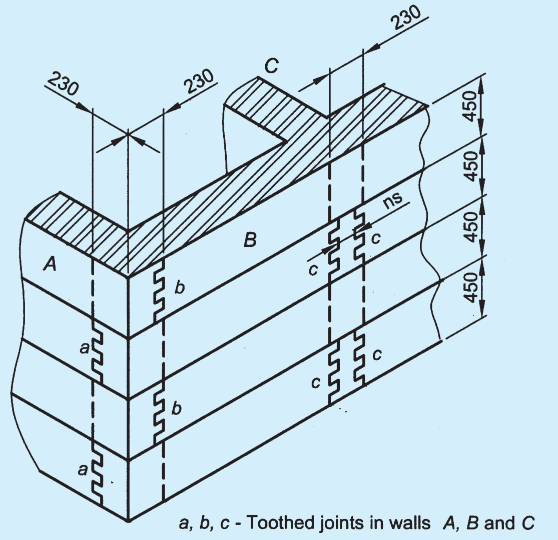
7.4.2 The bearing walls in both directions shall be straight and symmetrical in plan as far as possible.

7.4.3 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.

NOTE — For panel walls of 200 mm or larger thickness having a storey height not more than 3.5 m and laterally supported at the top, this check need not be exercised.

7.4.4 *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 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. 14).



All dimensions in millimetres.

FIG. 14 ALTERNATING TOOTHED JOINTS IN WALLS AT CORNER AND T-JUNCTION

7.4.5 Ignoring tensile strength, free standing walls shall be checked against overturning under the action of design seismic coefficient α_h allowing for a factor of safety of 1.5.

7.4.6 Panel or filler walls in framed buildings shall be properly bonded to surrounding framing members by means of suitable mortar (*see* Table 13) or connected through dowels. If the walls are so bonded they shall be checked according to **7.4.3** otherwise as in **7.4.5**.

7.5 Openings in the Bearing Walls

7.5.1 Door and window openings in walls reduce their lateral load resistance and hence, should preferably be small and more centrally located. The guidelines on the size and position of opening are given in Table 14 and Fig. 15.

7.5.2 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.

7.5.3 Where openings do not comply with the guidelines of Table 14, they should be strengthened by providing reinforced concrete or reinforcing the brickwork, as shown in Fig. 16 with high strength deformed steel bars of 8 mm diameter but the quantity of steel shall be increased at the jambs to comply with **7.6.9**, if so required.

7.5.4 If a window or ventilator is to be projected out, the projection shall be in reinforced masonry or concrete and well anchored.

Table 14 Size and Position of Openings in Bearing Walls

(*Clause* 7.5.1 and *Fig.* 15)

Sl No.	Position of Opening	Details of Opening for Building Category		
		A and B	C	D and E
(1)	(2)	(3)	(4)	(5)
i)	Distance b_5 from the inside corner of outside wall, *Min*	0	230 mm	450 mm
ii)	For total length of openings, the ratio $(b_1 + b_2 + b_3)/ l_1$ or $(b_6 + b_7)/ l_2$ shall not exceed:			
	a) one-storeyed building	0.60	0.55	0.50
	b) two-storeyed building	0.50	0.46	0.42
	c) three or four-storeyed building	0.42	0.37	0.33
iii)	Pier width between consecutive openings b_4, *Min*	340 mm	450 mm	560 mm
iv)	Vertical distance between two openings one above the other h_3, *Min*	600 mm	600 mm	600 mm
v)	Width of opening of ventilator b_8, *Max*	900 mm	900 mm	900 mm

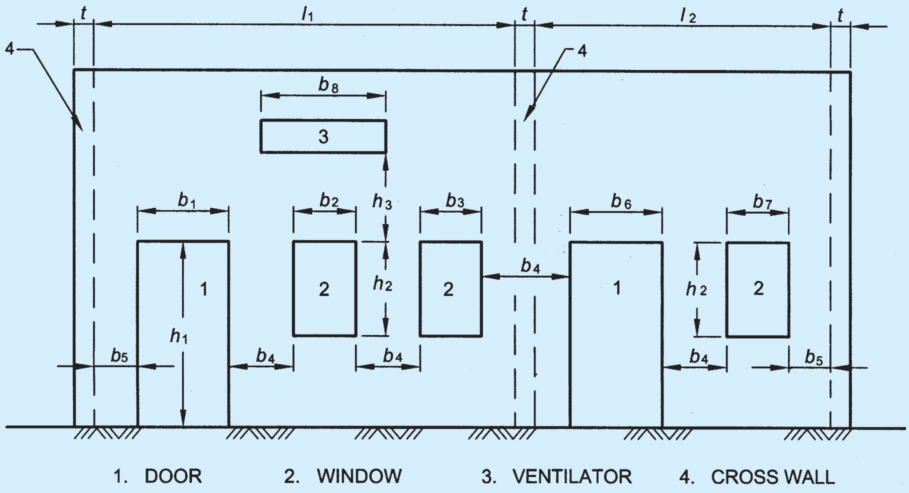


FIG. 15 RECOMMENDED DIMENSIONS OF OPENINGS AND PIERS (*see* Table 14)

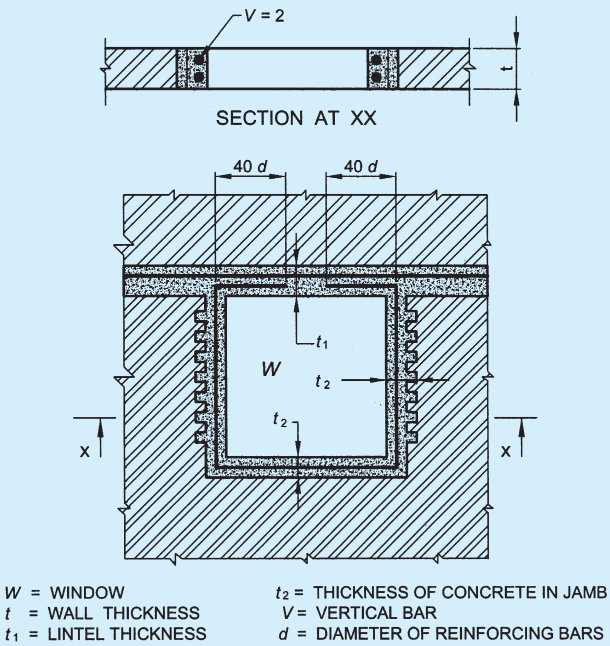


FIG. 16 STRENGTHENING MASONARY AROUND OPENING

7.5.5 If an opening is tall from bottom to almost top of a storey, thus dividing the wall into two portions, these 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.

7.5.6 The use of arches to span over the openings is a source of weakness and shall be avoided. Otherwise, steel ties should be provided.

7.6 Seismic Strengthening Arrangements

7.6.1 All masonry buildings shall be strengthened by the methods, as specified for various categories of buildings, as listed in Table 15, and detailed in subsequent clauses. Figures 17 and 18 show, schematically, the overall strengthening arrangements to be adopted for category 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 opening.

Table 15 Strengthening Arrangements Recommended for Masonry Buildings (Rectangular Masonry Units)

(*Clause* 7.6.1)

Building Category	Number of Storeys	Strengthening to be Provided in all Storeys
(1)	(2)	(3)
A	i) 1 to 3	*a*
	ii) 4	*a, b, c*
B	i) 1 to 3	*a, b, c, f, g*
	ii) 4	*a, b, c, d, f, g*
C	i) 1 and 2	*a, b, c, f, g*
	ii) 3 and 4	*a* to *g*
D	i) 1 and 2	*a* to *g*
	ii) 3 and 4	*a* to *h*
E	1 to 3[1)]	*a* to *h*

where

a – Masonry mortar (*see* **7.3**)

b – Lintel band (*see* **7.6.2**)

c – Roof band and gable band where necessary (*see* **7.6.3** and **7.6.4**),

d – Vertical steel at corners and junctions of walls (*see* **7.6.8**)

e – Vertical steel at jambs of openings (*see* **7.6.9**)

f – Bracing in plan at tie level of roofs

g – Plinth band where necessary (*see* **7.6.6**), and

h – Dowel bars (*see* **7.6.7**)

1) Fourth storey not allowed in category E.

NOTE — In case of four storey buildings of category B, the requirements of vertical steel may be checked through a seismic analysis using a design seismic coefficient equal to four times the one given in good practice [6-4(8)] (this is because the brittle behaviour of masonry in the absence of a vertical steel results in much higher effective seismic force than that envisaged in the seismic coefficient, provided in the Code). If this analysis shows that vertical steel is not required the designer may take the decision accordingly.

7.6.2 Lintel band is a band provided at lintel level on all load bearing internal, external longitudinal and cross walls. The specifications of the band are given in **7.6.3**.

NOTE — Lintel band if provided in panel or partition walls also will improve their stability during severe earthquake.

7.6.3 Roof band is a band provided immediately below the roof or floors. The specifications of the band are given in **7.6.5**. 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 ¾ of the wall thickness.

7.6.4 Gable band is a band provided at the top of gable masonry below the purlins. The specifications of the band are given in **7.6.5**. This band shall be made continuous with the roof band at the eaves level.

7.6.5 *Section and Reinforcement of Band*

The band shall be made of reinforced concrete of grade not leaner than M15 or reinforced brick work in cement mortar not leaner than 1:3. The bands shall be of the full width of the wall, not less than 75 mm in depth and reinforced with steel, as indicated in Table 16.

NOTE — In coastal areas, the concrete grade shall be M20 concrete and the filling mortar of 1:3 (cement sand with water proofing admixture).

Table 16 Recommended Longitudinal Steel in Reinforced Concrete Bands

(*Clauses* 7.6.5 *and* 7.8.1 and *Table* 17)

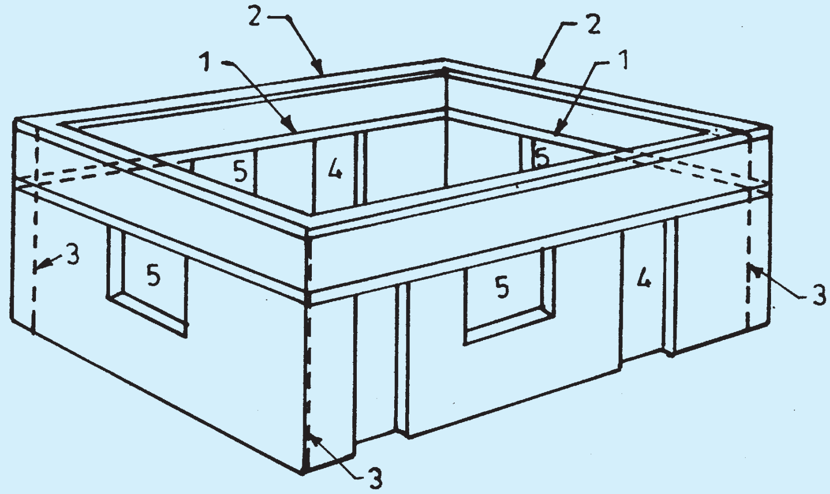
Span	Building Category B		Building Category C		Building Category D		Building Category E	
	No. of Bars	Dia	No. of Bars	Dia	No. of Bars	Dia	No. of Bars	Dia
m		mm		mm		mm		mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
5 or less	2	8	2	8	2	8	2	10
6	2	8	2	8	2	10	2	12
7	2	8	2	10	2	12	4	10
8	2	10	2	12	4	10	4	12

NOTES

1 Span of wall will be the distance between centre lines of its cross walls or buttresses. For spans greater than 8 m it will be desirable to insert pilasters or buttresses to reduce the span or special calculations shall be made to determine the strength of wall and section of band.

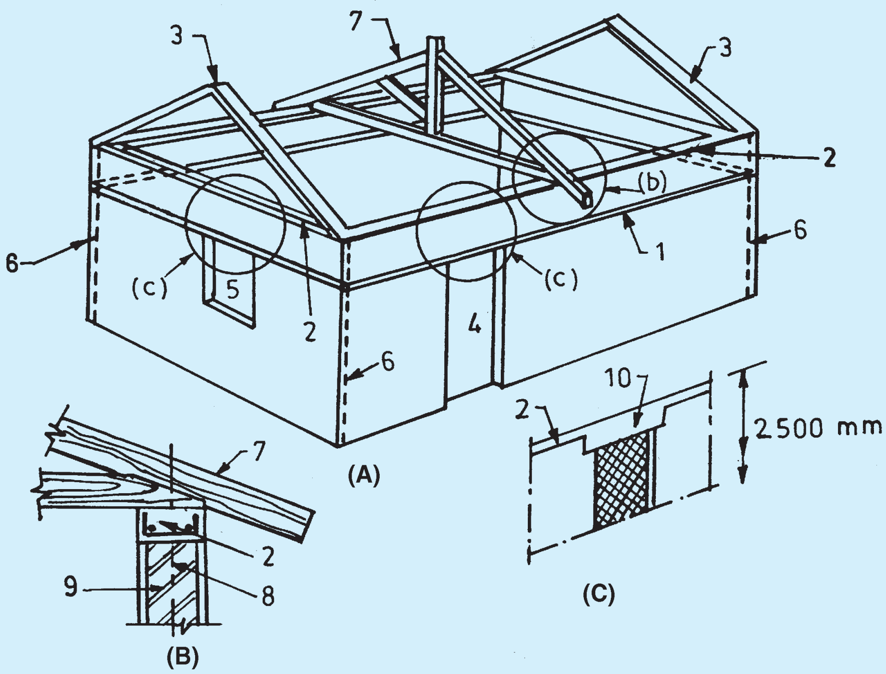
2 The number and diameter of bars given above pertain to high strength deformed bars. If plain mild steel bars are used keeping the same number, the following diameters may be used:

High strength deformed steel bar diameter	8	10	12	16	20



1. LINTEL BAND
2. ROOF FLOOR BAND
3. VERTICAL BAR
4. DOOR
5. WINDOW

FIG. 17 OVERALL ARRANGEMENT OF REINFORCING MASONRY BUILDINGS



1. LINTEL BAND
2. EAVE LEVEL (ROOF) BAND
3. CABLE BAND
4. DOOR
5. WINDOW
6. VERTICAL STEEL BAR
7. RAFTER
8. HOLDING DOWN BOLT
9. BRICK/STONE WALL
10. DOOR LINTEL INTEGRATE WITH ROOF BAND

18 A PERSPECTIVE VIEW
18 B DETAILS OF TRUSS CONNECTION WITH WALL
18 C DETAILS OF INTEGRATING DOOR LINTEL WITH ROOF BAND

FIG. 18 OVERALL ARRANGEMENT OF REINFORCING MASONRY BUILDING HAVING PITCHED ROOF

Table 16 — *Concluded*

Mild steel plain deformed bar diameter	10	12	16	20	25

3 Width of RC band is assumed same as the thickness of the wall. Wall thickness shall be 200 mm minimum. A clear cover of 20 mm from face of wall will be maintained.

4 The vertical thickness of RC band be kept 75 mm minimum, where two longitudinal bars are specified, one on each face; and 150 mm, where four bars are specified.

5 Concrete mix shall be of grade M15 or 1:2:4 by volume.

6 The longitudinal steel bars shall be held in position by steel links or stirrups 6 mm dia spaced at 150 mm apart.

7.6.5.1 In case of reinforced brickwork, the thickness of joints containing steel bars shall be increased so as to have a minimum mortar cover of 10 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.

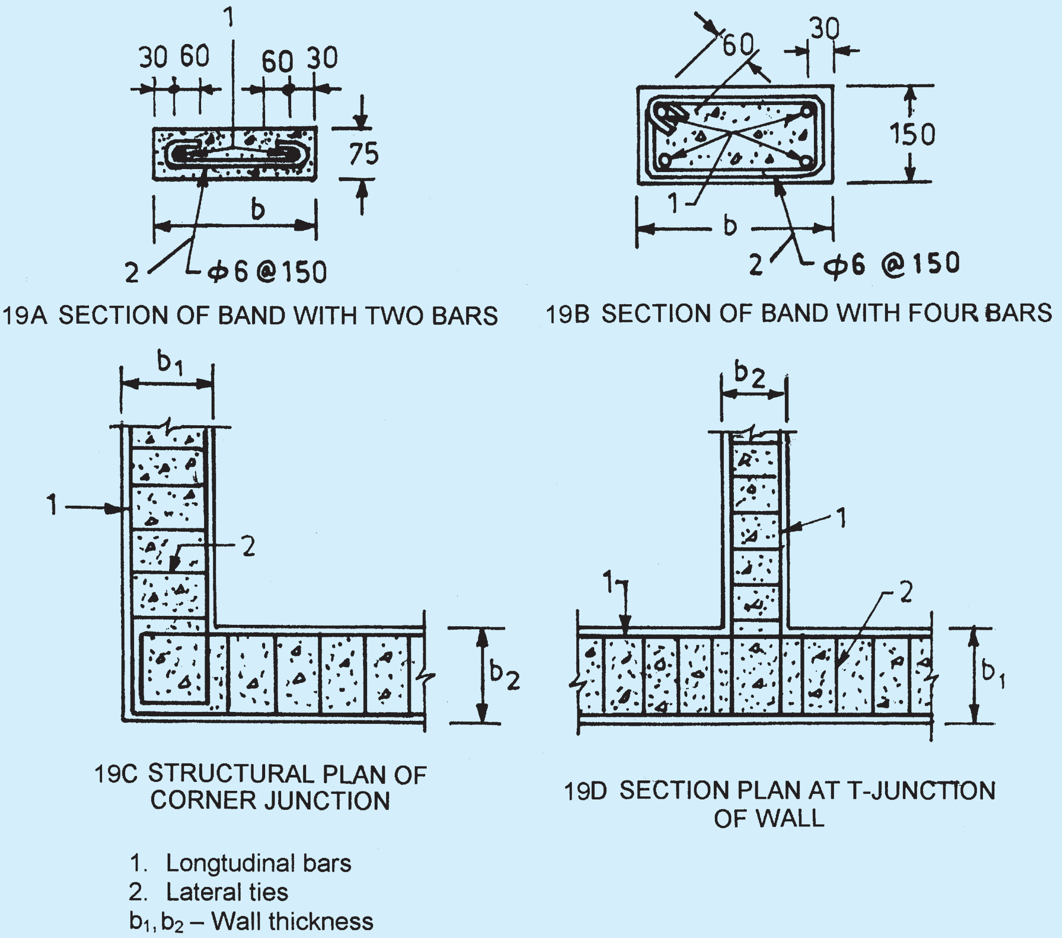
7.6.5.2 For full integrity of walls at corners and junctions of walls and effective horizontal bending resistance of bands continuity of reinforcement is essential. The details as shown in Fig. 19 are recommended.

7.6.6 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 **7.6.5**. This band will serve as damp proof course as well.

7.6.7 In category D and E buildings, to further iterate 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 a length of 900 mm from the inside corner in each wall. Such dowel may be in the form of U stirrups of 8 mm diameter. Where used, such bars shall be laid in 1:3 cement-sand-mortar with a minimum cover of 10 mm on all sides to minimize corrosion.

7.6.8 *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 17. For walls thicker than 340 mm, the area of the bars shall be proportionately increased.



All dimensions in millimetres.

FIG. 19 REINFORCEMENT AND BENDING DETAIL IN R.C. BAND

Table 17 Vertical Steel Reinforcement in Masonry Walls with Rectangular Masonry Units

(*Clauses* 7.6.8, 7.6.9 *and* 8.7.2)

No. of Storeys	Storey	Diameter of HSD Single Bar in mm at Each Critical Section			
		Category B	Category C	Category D	Category E
(1)	(2)	(3)	(4)	(5)	(6)
One	—	Nil	Nil	10	12
Two	Top	Nil	Nil	10	12
	Bottom	Nil	Nil	12	16
Three	Top	Nil	10	10	12
	Middle	Nil	10	12	16
	Bottom	Nil	12	12	16
Four	Top	10	10	10	Four storeyed building not permitted
	Third	10	10	12	
	Second	10	12	16	
	Bottom	12	12	20	

NOTES

1 The diameters given above are for high strength deformed steel bars. For mild steel plain bars, use equivalent diameters as given in Table 16 (Note 2).

2 The vertical bars will be covered with concrete M 15 or mortar 1:3 grade in suitably created pockets around the bars. This will ensure their safety from corrosion and good bond with masonry.

3 In case of floors/roofs with small precast components, also refer good practice [6-4(8)] for floor/roof band details.

For earthquake resistant framed wall construction, (*see* **7.7**). No vertical steel need be provided in category A buildings.

7.6.8.1 The vertical reinforcement shall 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 shall be passing through the lintel bands and floor level bands in all storeys.

Bars in different storeys may be welded or suitably lapped.

NOTE — Typical details of providing vertical steel in brickwork masonry with rectangular solid units at corners and T-junctions are shown in Fig. 20.

7.6.9 Vertical reinforcement at jambs of window and door openings shall be provided as per Table 17. It may start from foundation of floor and terminate in lintel band (*see* Fig. 21).

7.7 Framing of Thin Load Bearing Walls (*see* Fig. 21)

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 will 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 will 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 M15 (1:2:4) concrete to fill the columns against the walls using wood forms only on two sides. The column 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.

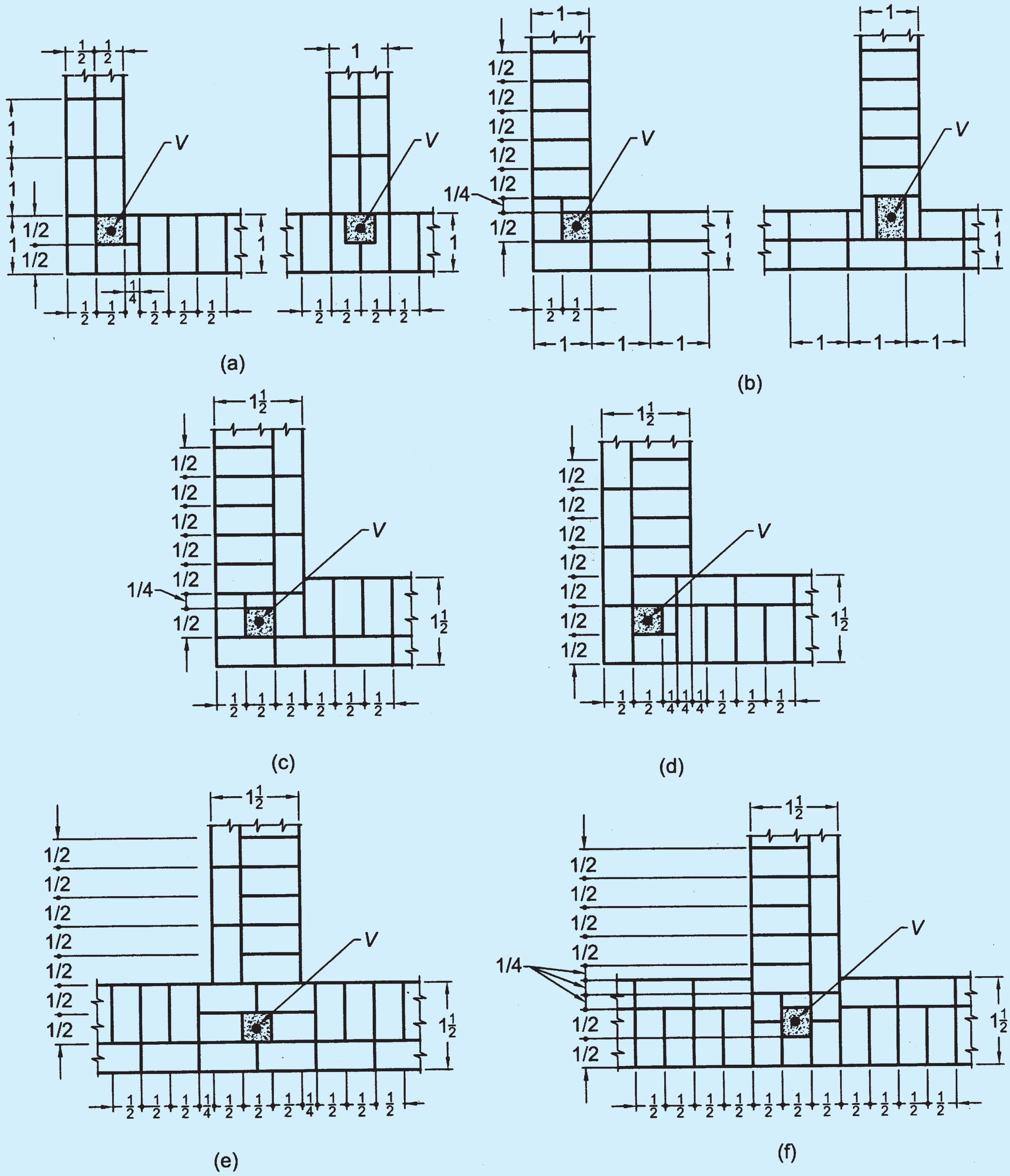
Such construction may be limited to only two storeys maximum in view of its vertical load carrying capacity. The horizontal length of walls between cross walls shall be restricted to 7 m and the storey height to 3 m.

7.8 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.

7.8.1 *Horizontal Band*

U-shaped blocks may be used for construction of horizontal bands in various levels of the storeys as shown in Fig. 22, where the amount of horizontal reinforcement shall be taken 25 percent more than that given in Table 16 and provided by using four bars and 6 mm dia stirrups. Other continuity details shall be followed, as shown in Fig. 19.



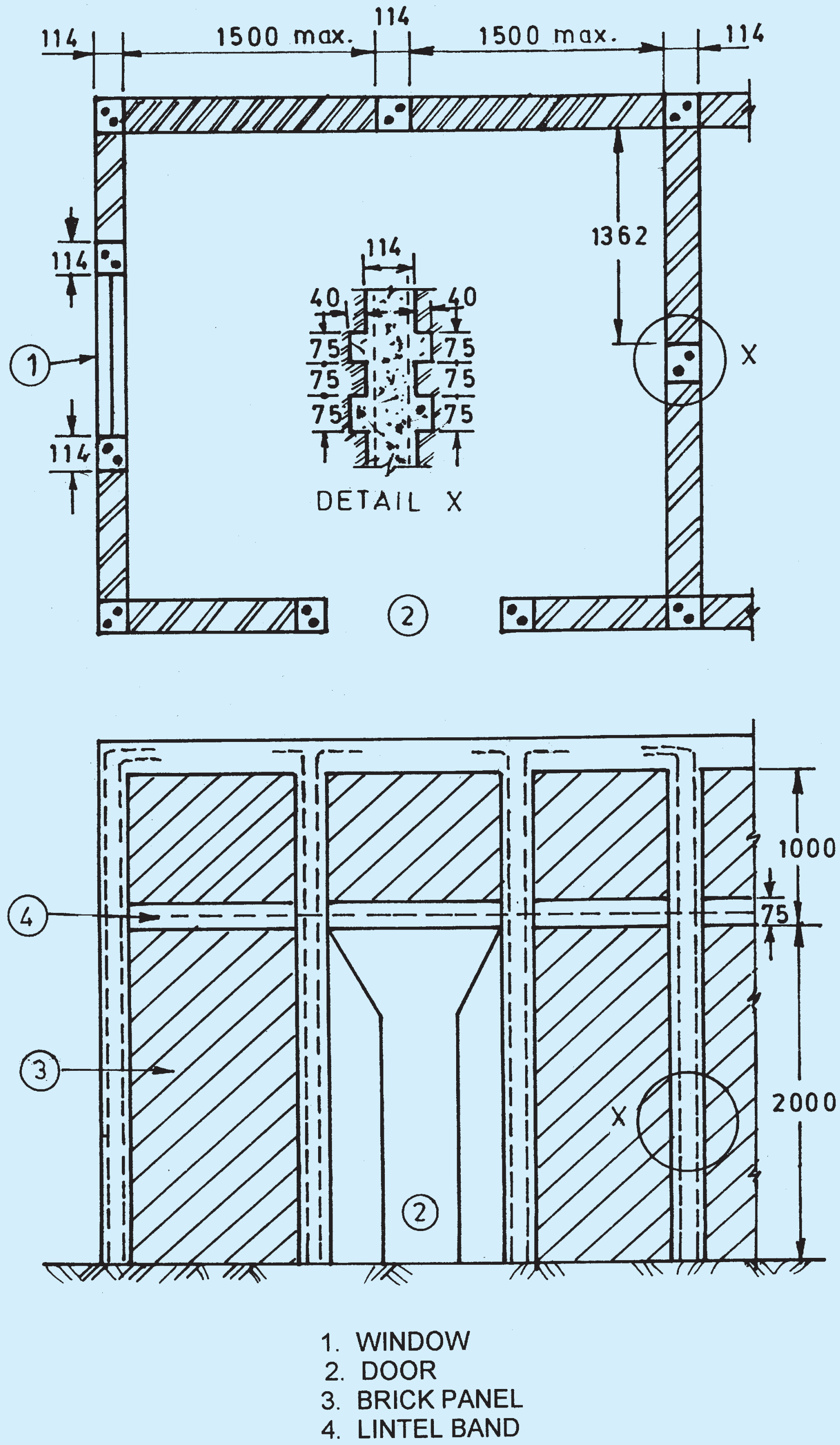
1 — One-brick length, ½ — Half brick length, *V* — Vertical steel bar with mortar/concrete filling in pocket

(a) and (b) — Alternate courses in one brick

(c) and (d) — Alternate courses at corner junction of 1½-brick wall

(e) and (f) — Alternate courses at T-junction of 1½-brick wall

FIG. 20 TYPICAL DETAILS OF PROVIDING VERTICAL STEEL BARS IN BRICK MASONRY



All dimensions in millimetres.

FIG. 21 FRAMING OF THIN LOAD-BEARING BRICK WALLS

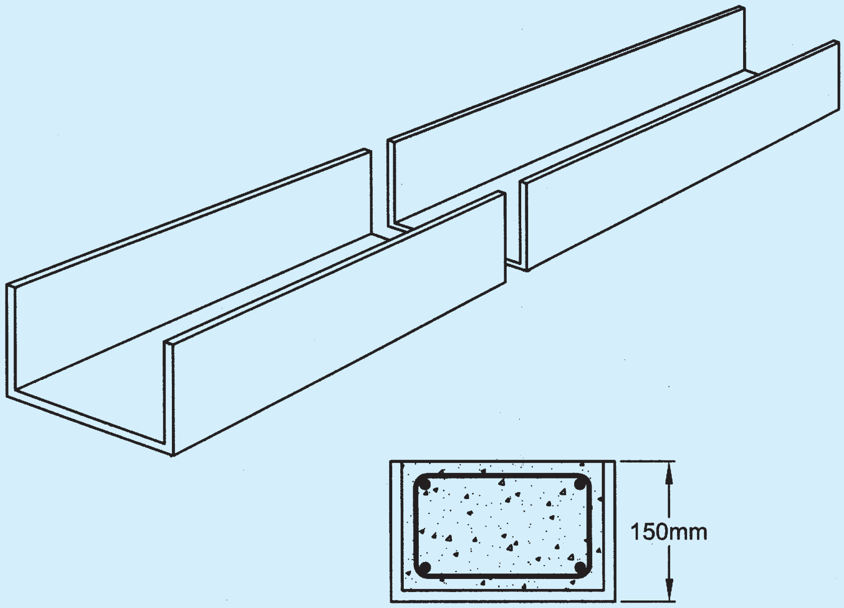


FIG. 22 U-BLOCKS FOR HORIZONTAL BANDS

7.8.2 *Vertical Reinforcement*

Bars, as specified in Table 17 shall be located inside the cavities of the hollow blocks, one bar in each cavity (*see* Fig. 23). 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. 23 which will avoid lifting and threading of bars into the hollows.

8 GUIDELINES FOR IMPROVING EARTHQUAKE RESISTANCE OF LOW STRENGTH MASONRY BUILDINGS

8.0 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 **8.1** to **8.4.7.** Reference may also be made to good practice [6-4(9)] for detailed information.

8.1 General

8.1.1 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.

8.1.2 These constructions should not be permitted for important buildings with $I \geq 1.5$ and should preferably be avoided for building category D and shall not be used for category E (*see* Table 12).

8.1.3 It will be useful to provide damp-proof course at plinth level to stop the rise of pore water into the superstructure.

8.1.4 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.

8.1.5 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.

8.1.6 Ignoring tensile strength, free standing walls should be checked against overturning under the action of design seismic coefficient, a_h, allowing for a factor of safety of 1.5.

8.2 Brickwork in Weak Mortars

8.2.1 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.

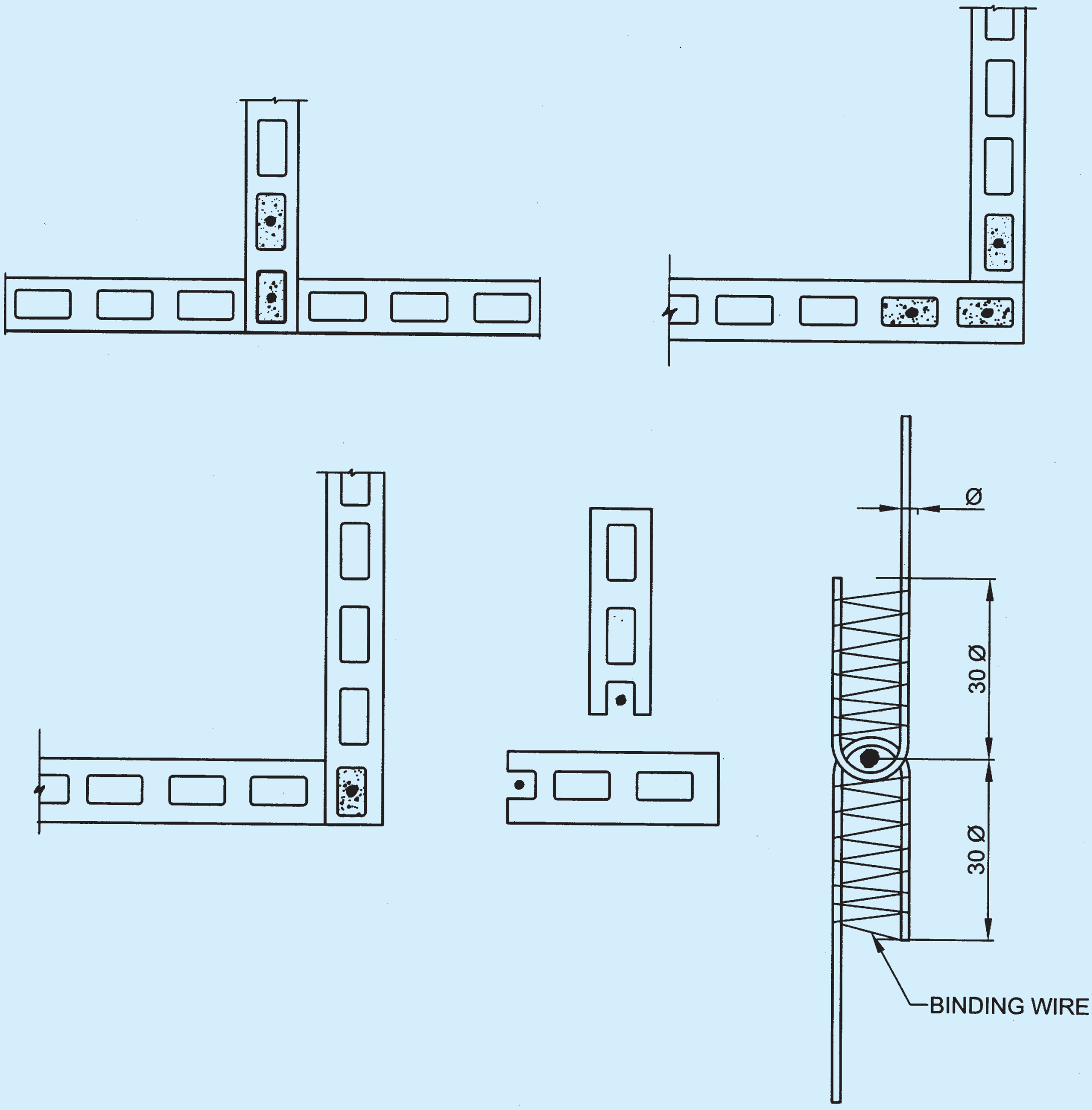


FIG. 23 VERTICAL REINFORCEMENT IN CAVITIES

8.2.2 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 1½ × 1½ brick size depending on wall thickness should preferably be built using 1:6 cement-sand mortar.

8.2.3 The minimum wall thickness shall be one brick in one storey construction, and one brick in top storey and 1½ 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.

8.2.4 The height of the building shall be restricted to the following, where each storey height shall not exceed 3.0 m:

a) *For Categories A, B and C* — three storeys with flat roof; and two storeys plus attic for pitched roof.

b) *For Category D* — two storeys with flat roof; and one storey plus attic for pitched roof.

8.2.5 *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. 14).

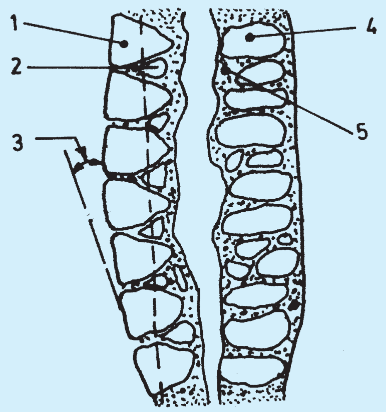
8.3 Stone Masonry (Random Rubble or Half-Dressed)

8.3.1 The construction of stone masonry of random rubble or dressed stone type should generally follow good practice [6-4(3)].

8.3.2 The mortar should be cement-sand (1:6), lime sand (1:3) or clay mud of good quality.

8.3.3 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. 24) and buckle separately under vertical load leading to complete collapse of the wall and the building.



1. Half-cressed conical stone
2. Small alignment stone
3. Rotation of wythe
4. Random rubble
5. Mud or weak lime mortar

FIG. 24 WALL DELAMINATED WITH BUCKLED WYTHES

8.3.4 The masonry should preferably be brought to courses at not more than 600 mm lift.

8.3.5 '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. 25).

8.3.6 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. 25). 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.

8.3.7 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.

8.3.8 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 categories A and 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.

b) *For categories C and D* — Two stroreys 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.

8.3.9 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.

8.4 Opening in Bearing Walls

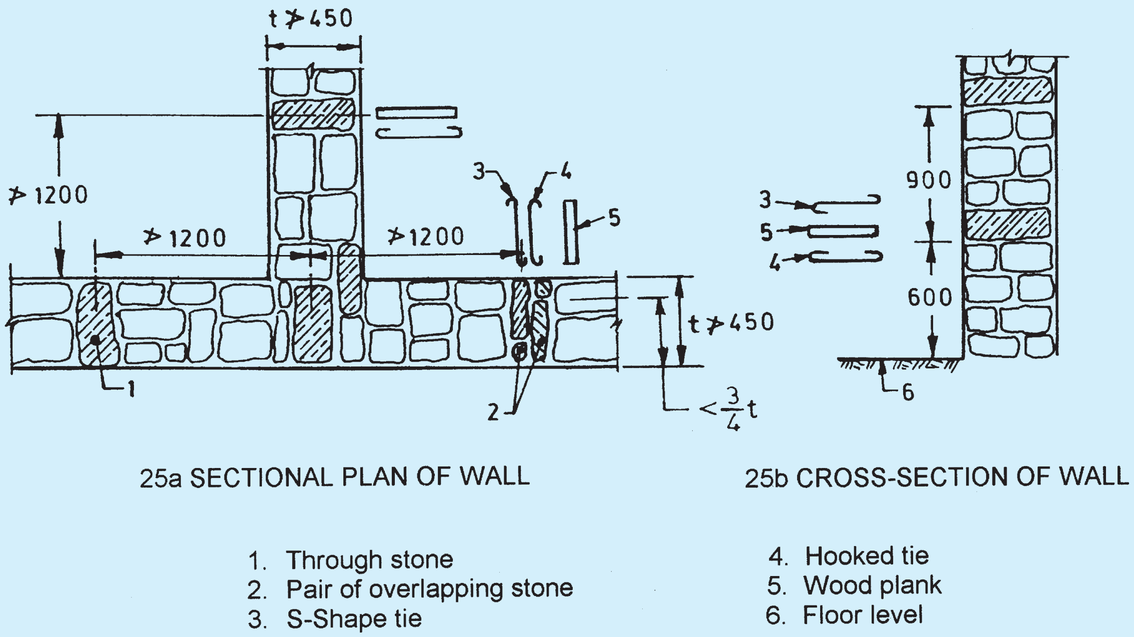
8.4.1 Door and window openings in walls reduce their lateral load resistance and hence should preferably, be small and more centrally located. The size and position of openings shall be as given in Table 18 and Fig. 15.

8.4.2 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.

Table 18 Size and Position of Openings in Bearing Walls (*see* Fig. 15)

(*Clause* 8.4.1)

Sl No.	Description	Building Category	
		A, B & C	D
(1)	(2)	(3)	(4)
i)	Distance b_5 from the inside corner of outside wall, *Min*	230 mm	600 mm
ii)	Total length of openings, ratio; *Max*: $(b_1 + b_2 + b_3)/l_1$ or $(b_6 + b_7)/l_2$	0.46	0.42
	1) one storeyed building	0.37	0.33
	2) 2 and 3 storeyed building		
iii)	Pier width between consecutive openings b_4	450 mm	560 mm
iv)	Vertical distance between two openings one above the other, h_3, *Min*	600 mm	600 mm



All dimensions in millimetres.

FIG. 25 THROUGH STONE AND BAND ELEMENTS

8.4.3 Where openings do not comply with the guidelines of Table 18, they should be strengthened by providing reinforced concrete lining as shown in Fig. 16 with 2 high strength deformed steel bars of 8 mm diameter.

8.4.4 The use of arches to span over the openings is a source of weakness and shall be avoided, otherwise, steel ties should be provided.

8.5 Seismic Strengthening Arrangements

8.5.1 All buildings to be constructed of masonry shall be strengthened by the methods as specified for various categories of buildings, listed in Table 19 and detailed in subsequent clauses. Fig. 17 and Fig. 18 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.

8.5.2 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 **8.5.5**.

8.5.3 Roof band is a band provided immediately below the roof or floors. The details of the band are given in **8.5.5**. 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.

8.5.4 Gable band is a band provided at the top of gable

Table 19 Strengthening Arrangements Recommended for Low Strength Masonry Buildings

(*Clause* 8.5.1)

Building Category	Number of Storeys	Strengthening to be Provided
(1)	(2)	(3)
A	1	*b, c, f, g*
	2 and 3	*b, c, f, g*
B	1 and 2	*b, c, f, g*
	3	*b, c, d, f, g* (*see* Note 1)
C	1	*b, c, f, g*
	2 and 3	*b, c, d, f, g*
D	1 and 2	*b, c, d, f, g*

Strengthening Method

b – Lintel band (*see* **8.5.2**)

c – Roof band and gable band where necessary (*see* **8.5.3** *and* **8.5.4**)

d – Vertical steel at corners and junctions of walls (*see* **8.5.7**)

f – Bracing in plan at tie level of pitched roofs (*see* Note 2)

g – Plinth band where necessary (*see* **8.5.6**)

NOTES

1 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.

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.

masonry below the purlins. The details of the band are given in **8.5.5**. This band shall be made continuous with the roof band at the eaves level.

8.5.5 *Details of Band*

8.5.5.1 *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. 19.

NOTES

1 In coastal areas, the concrete grade shall be of grade in accordance with Part 6 'Structural Design, Section 5 Concrete' 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. 19 are recommended.

8.5.5.2 *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. 26.

8.5.6 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 **8.5.5.1**. This band serves as damp proof course as well.

8.5.7 *Vertical Reinforcement*

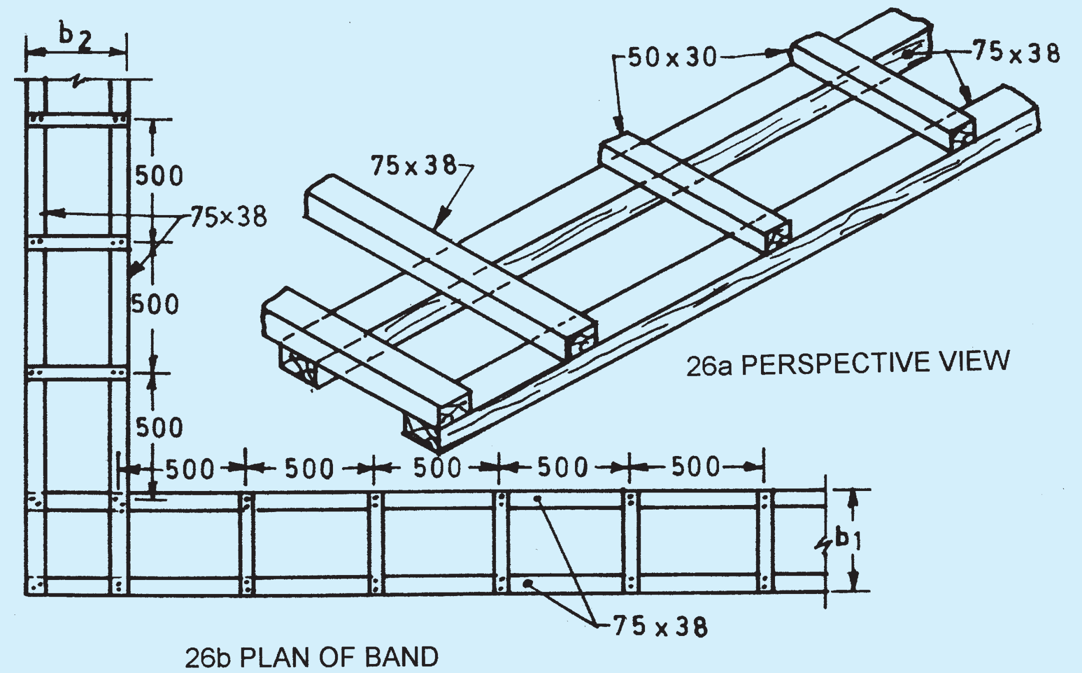
Vertical steel at corners and junctions of walls which are up to 350 mm thick should be provided as specified in Table 20. For walls thicker than 350 mm, the area of the bars should be proportionately increased.

8.5.7.1 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. 20.

2 For providing vertical bar in stone masonry, use of a casing pipe is recommended around which masonry be built to height



All dimensions in millimetres.

Fig. 26 Wooden Band for Low Strength Masonry Buildings

Table 20 Vertical Steel Reinforcement in Low Strength Masonry Walls

(*Clause* 8.5.7)

No. of Storeys	Storey	Diameter of HSD Single Bar; in mm, at Each Critical Section for			
		Category A	Category B	Category C	Category D
(1)	(2)	(3)	(4)	(5)	(6)
One	—	Nil	Nil	Nil	10
Two	Top	Nil	Nil	10	10
	Bottom	Nil	Nil	10	12
Three	Top	Nil	10	10	10
	Middle	Nil	10	10	12
	Bottom	Nil	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. This will ensure their safety from corrosion and good bond with masonry.

3 For category B two storey stone masonry buildings, *see* Note 1 under Table 19.

of 600 mm (see Fig. 27). 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.

9 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 [6-4(10)].

10 NOTATIONS AND SYMBOLS

The various notations and letter symbols used in the text of this Section of the Code shall have the meaning as given in Annex E.

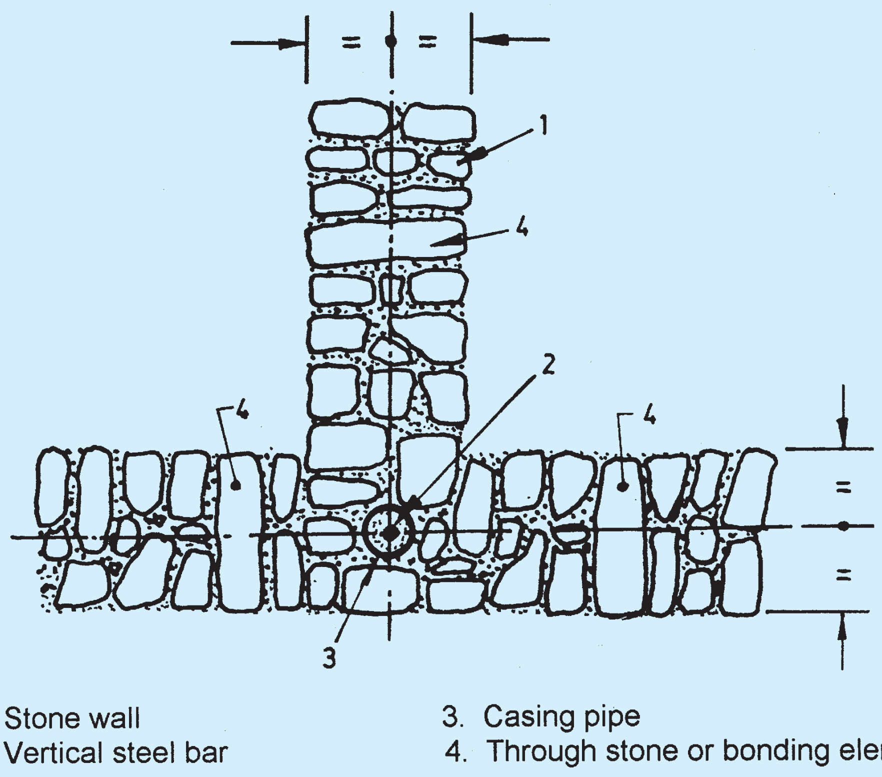


FIG. 27 TYPICAL CONSTRUCTION DETAIL FOR INSTALLING VERTICAL STEEL BAR IN RANDOM RUBBLE STONE MASONRY

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

(*Clause* 5.4.1)

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 21.

Table 21 Correction Factors for Different *h/t* Ratios

(*Clause* B-1.1)

Ratio of Height to Thickness (*h/t*)	2.0	2.5	3.0	3.5	4.0	5.0
Correction Factors for Brickwork[1)]	0.73	0.80	0.86	0.91	0.95	1.00
Correction Factors for Blockwork[1)]	1.00	—	1.20	—	1.30	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 of 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 M15 Grade) capable of taking tension.

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 M15 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 M15 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 would 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 22.

Table 22 Bending Moments in Laterally Loaded Panel Walls, Free at Top Edge and Supported on Other Three Edges

$\frac{\text{Height of Panel, } H}{\text{Length of Panel, } L}$	0.30	0.50	0.75	1.00	1.25	1.50	1.75
Bending Moment	$\frac{P.L}{25}$	$\frac{P.L}{18}$	$\frac{P.L}{14}$	$\frac{P.L}{12}$	$\frac{P.L}{11}$	$\frac{P.L}{10.5}$	$\frac{P.L}{10}$

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 $PL/8$.

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 $\mu = 0.50$, are given in Table 23.

Table 23 Bending Moments in Laterally Loaded Panel Walls Supported on All Four Edges

$\frac{\text{Height of Panel, } H}{\text{Length of Panel, } L}$	0.30	0.50	0.75	1.00	1.25	1.50	1.75
Bending Moment	$\frac{P.L}{72}$	$\frac{P.L}{36}$	$\frac{P.L}{24}$	$\frac{P.L}{18}$	$\frac{P.L}{15}$	$\frac{P.L}{13}$	$\frac{P.L}{12}$

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; or
   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; or
   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.

ANNEX E

(*Clause* 10)

NOTATIONS, SYMBOLS AND ABBREVIATIONS

E-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 the Code:

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)
GL	=	Ground level
H	=	Actual height between lateral supports
H'	=	Height of opening
$H1$, $H2$	=	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
$L1$, $L2$	=	Lower strength mortars
$M1$, $M2$	=	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.

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 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 by the Authority as a guide in 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.

	IS No.	*Title*
(1)	1077 : 1992	Specification for common burnt clay building bricks (*fifth revision*)
	2180 : 1988	Specification for heavy duty burnt clay building bricks (*third revision*)
	2185	Specification for concrete masonry units:
	(Part 1) : 1979	Hollow and solid concrete blocks (*second revision*)

IS No.	Title
(Part 2) : 1983	Hollow and solid light weight concrete blocks (*first revision*)
(Part 3) : 1984	Autoclaved cellular aerated concrete blocks (*first revision*)
2222 : 1991	Specification for burnt clay perforated building bricks (*fourth revision*)
2849 : 1983	Specification for non-load bearing gypsum partition blocks (solid and hollow types) (*first revision*)
3115 : 1992	Specification for lime based blocks (*second revision*)
3316 : 1974	Specification for structural granite (*first revision*)
3620 : 1979	Specification for laterite stone block for masonry (*first revision*)
3952 : 1988	Specification for burnt clay hollow blocks for walls and partitions (*second revision*)
4139 : 1989	Specification for calcium silicate bricks (*second revision*)
12440 : 1988	Specification for precast concrete stone masonry blocks
12894 : 2002	Specification for pulverized fuel ash lime bricks (*first revision*)
13757 : 1993	Specification for burnt clay fly ash building bricks
(2) 2250 : 1981	Code of practice for preparation and use of masonry mortars (*first revision*)
(3) 1597	Code of practice for construction of stone masonry:
(Part 1) : 1992	Rubble stone masonry (*first revision*)
(Part 2) : 1992	Ashlar masonry (*first revision*)
2110 : 1980	Code of practice for *in-situ* construction of walls, in building with soil-cement (*first revision*)

IS No.	Title
2212 : 1991	Code of practice for brickwork (*first revision*)
2572 : 1963	Code of practice for construction of hollow concrete block masonry
2849 : 1983	Specification for non-load bearing gypsum partition blocks (solid and hollow types) (*first revision*)
3630 : 1992	Code of practice for construction of non-load bearing gypsum block partitions (*first revision*)
4326 : 1993	Code of practice for earthquake resistant design and construction of buildings (*second revision*)
6041 : 1985	Code of practice for construction of autoclaved cellular concrete block masonry (*first revision*)
6042 : 1969	Code of practice for construction of lightweight concrete block masonry
(4) 2212 : 1991	Code of practice for brickwork (*first revision*)
(5) 3414 : 1968	Code of practice for design and installation of joints in buildings
(6) 4326 : 1993	Code of practice for earthquake resistant design and construction of buildings (*second revision*)
(7) 1905 : 1987	Code of practice for structural use of un-reinforced masonry (*third revision*)
(8) 1893 (Part 1) : 2002	Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings (*fifth revision*)
(9) 13828 : 1993	Guidelines for improving earthquake resistance of low strength masonry buildings
(10) 10440 : 1983	Code of practice for construction of RB and RBC floors and roofs

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 5 Concrete: 5A Plain and Reinforced Concrete

BUREAU OF INDIAN STANDARDS

CONTENTS

SECTION 5A (e) STRUCTURAL DESIGN (LIMIT STATE METHOD)

National Building Code Sectional Committee, CED 46

FOREWORD

Section 5 of Part 6 of the Code covers plain and reinforced concrete as also the prestressed concrete. The Section has been subdivided into the following sub-sections:

5 A Plain and Reinforced Concrete

5 B Prestressed Concrete

This sub-section 5A covers the structural design aspects of plain and reinforced concrete.

This sub-section 5A was first published in 1970 and was subsequently revised in 1983, to bring it in line with revised version of IS 456 : 1978 on which this chapter was based. Now this revision is intended to bring this subsection in line with the revised version of IS 456 : 2000.

This revision incorporates a number of important changes. The major thrust in the revision is on the following lines:

a) In recent years, durability of concrete structures have become the cause of concern to all concrete technologists. This has led to codify the durability requirements world over. In this revision of the Code, in order to introduce in-built protection from factors affecting a structure, earlier clause on durability has been elaborated and a detailed clause covering different aspects of design of durable structure has been incorporated.

b) Sampling and acceptance criteria for concrete have been revised. With this revision acceptance criteria has been simplified in line with the provisions given in BS 5328 (Part 4) : 1990 'Concrete: Part 4 Specification for the procedures to be used in sampling, testing and assessing compliance of concrete'.

Some of the significant changes incorporated in Section 5A (b) are as follows:

a) All the three grades of ordinary Portland cement, namely 33 grade, 43 grade and 53 grade and sulphate resisting Portland cement have been included in the list of types of cement used (in addition to other types of cement).

b) The permissible limits for solids in water have been modified keeping in view the durability requirements.

c) The clause on admixtures has been modified in view of the availability of new types of admixtures including superplasticizers.

d) In Table 2 'Grades of Concrete', grades higher than M 40 have been included.

e) It has been recommended that minimum grade of concrete shall be not less than M 20 in reinforced concrete work (*see also* **5.1.3**).

f) The formula for estimation of modulus of elasticity of concrete has been revised.

g) In the absence of proper correlation between compacting factor, vee-bee time and slump, workability has now been specified only in terms of slump in line with the provisions in BS 5328 (Parts 1 to 4).

h) Durability clause has been enlarged to include detailed guidance concerning the factors affecting durability. The table on 'Environmental Exposure Conditions' has been modified to include 'very severe' and 'extreme' exposure conditions. This clause also covers requirements for shape and size of member, depth of concrete cover, concrete quality, requirement against exposure to aggressive chemical and sulphate attack, minimum cement requirement and maximum water cement ratio, limits of chloride content, alkali silica reaction, and importance of compaction, finishing and curing.

j) A clause on 'Quality Assurance Measures' has been incorporated to give due emphasis to good practices of concreting.

k) Proper limits have been introduced on the accuracy of measuring equipments to ensure accurate batching of concrete.

m) The clause on 'Construction Joints' has been modified.

n) The clause on 'Inspection' has been modified to give more emphasis on quality assurance.

The significant changes incorporated in Section 5A (c) are as follows:

a) Requirements for 'Fire Resistance' have been further detailed.
b) The figure for estimation of modification factor for tension reinforcement used in calculation of basic values of span to effective depth to control the deflection of flexural member has been modified.
c) Recommendations regarding effective length of cantilever have been added.
d) Recommendations regarding deflection due to lateral loads have been added.
e) Recommendations for adjustments of support moments in restrained slabs have been included.
f) In the determination of effective length of compression members, stability index has been introduced to determine sway or no sway conditions.
g) Recommendations have been made for lap length of hooks for bars in direct tension and flexural tension.
h) Recommendations regarding strength of welds have been modified.
j) Recommendations regarding cover to reinforcement have been modified. Cover has been specified based on durability requirements for different exposure conditions. The term 'nominal cover' has been introduced. The cover has now been specified based on durability requirement as well as for fire requirements.

The significant change incorporated in Section 5A (d) is the modification of the clause on Walls. The modified clause includes design of walls against horizontal shear.

In Section 5 on limit state method a new clause has been added for calculation of enhanced shear strength of sections close to supports. Some modifications have also been made in the clause on Torsion. Formula for calculation of crack width has been added (separately given in Annex F).

Working stress method has now been given in Annex B so as to give greater emphasis to limit state design. In this Annex, modifications regarding torsion and enhanced shear strength on the same lines as in Section 5 have been made.

Whilst the common methods of design and construction have been covered in this Code, special systems of design and construction of any plain or reinforced concrete structure not covered by this Code may be permitted on production of satisfactory evidence regarding their adequacy and safety by analysis or test or both (*see* **18**).

In this Code it has been assumed that the design of plain and reinforced cement concrete work is entrusted to a qualified engineer and that the execution of cement concrete work is carried out under the direction of a qualified and experience supervisor.

This Section also introduces self-compacting concrete (*see* Annex A).

In the formulation of this subsection, assistance has been derived from the following publications:

BS 5328 (Part 1) : 1991 Concrete: Part 1 Guide to specifying concrete, British Standards Institution

BS 5328 (Part 2) : 1991 Concrete: Part 2 Methods for specifying concrete mixes, British Standards Institution

BS 5328 (Part 3) : 1990 Concrete: Part 3 Specification for the procedures to be used in producing and transporting concrete, British Standards Institution

BS 5328 (Part 4) : 1990 Concrete: Part 4 Specification for the procedures to be used in sampling, testing and assessing compliance of concrete, British Standards Institution

BS 8110 (Part 1) : 1985 Structural use of concrete: Part 1 Code of practice for design and construction, British Standards Institution

BS 8110 (Part 2) : 1985 Structural use of concrete: Part 2 Code of practice for special circumstances, British Standards Institution

ACI 318 : 1995 Building code requirements for reinforced concrete, American Concrete Institute

AS 3600 : 1988 Concrete structures, Standards Association of Australia

DIN 1045 July 1988 Structural use of concrete, design and construction, Deutsches Institut für Normung E.V.

CEB-FIP Model Code 1990, Comite Euro International Du Belon

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.

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 5 Concrete: 5A Plain and Reinforced Concrete

SECTION 5A (a) GENERAL

1 SCOPE

1.1 This Section deals with the general structural use of plain and reinforced concrete.

1.1.1 For the purpose of this Section, plain concrete structures are those where reinforcement, if provided is ignored for determination of strength of the structure.

1.2 Design of special requirements of structures, such as shells, folded plates, arches, bridges, chimneys, blast resistant structures, hydraulic structures, liquid retaining structures and earthquake resistant structures, shall be done in accordance with good practice [6-5A(1)].

2 TERMINOLOGY

For the purpose of this Section, the definitions given in accepted standards [6-5A(2)].

3 SYMBOLS

For the purpose of this standard, the following letter symbols shall have the meaning indicated against each; where other symbols are used, they are explained at the appropriate place:

A – Area
b – Breadth of beam, or shorter dimension of a rectangular column
b_{ef} – Effective width of slab
b_f – Effective width of flange
b_w – Breadth of web or rib
D – Overall depth of beam or slab or diameter of column; dimension of a rectangular column in the direction under consideration
D_f – Thickness of flange
DL – Dead load
d – Effective depth of beam or slab
d' – Depth of compression reinforcement from the highly compressed face
E_c – Modulus of elasticity of concrete
EL – Earthquake load
E_s – Modulus of elasticity of steel
e – Eccentricity
f_{ck} – Characteristic cube compressive strength of concrete
f_{cr} – Modulus of rupture of concrete (flexural tensile strength)
f_{ct} – Splitting tensile strength of concrete
f_d – Design strength
f_y – Characteristic strength of steel
H_w – Unsupported height of wall
H_{we} – Effective height of wall
I_{ef} – Effective moment of inertia
I_{gr} – Moment of inertia of the gross section excluding reinforcement
I_r – Moment of intertia of cracked section
K – Stiffness of member
k – Constant or coefficient or factor
L_d – Development length
LL – Live load or imposed load
l – Length of a column or beam between adequate lateral restraints or the unsupported length of a column
l_{ef} – Effective span of beam or slab or effective length of column
l_{ex} – Effective length about x-x axis
l_{ey} – Effective length about y-y axis
l_n – Clear span, face-to-face of supports
l'_{ef} – l'_{ef} for shorter of the two spans at right angles
l_x – Length of shorter side of slab
l_y – Length of longer side of slab
l_0 – Distance between points of zero moments in a beam
l_1 – Span in the direction in which moments are determined, centre-to-centre of supports
l_2 – Span transverse to l_1, centre-to-centre of supports
l'_2 – l_2 for the shorter of the continuous spans
M – Bending moment
m – Modular ratio
n – Number of samples
P – Axial load on a compression member
q_o – Calculated maximum bearing pressure
q_0 – Calculated maximum bearing pressure of soil
r – Radius
s – Spacing of stirrups or standard deviation
T – Torsional moment
t – Wall thickness
V – Shear force

W – Total load
WL – Wind load
w – Distributed load per unit area
w_d – Distributed dead load per unit area
w_l – Distributed imposed load per unit area
x – Depth of neutral axis
Z – Modulus of section
z – Lever arm
α, β – Angle or ratio
γ_f – Partial safety factor for load
γ_m – Partial safety factor for material
δ_m – Percentage reduction in moment
ε_{cc} – Creep strain of concrete
σ_{cbc} – Permissible stress in concrete in bending compression
σ_{cc} – Permissible stress in concrete in direct compression
σ_{mc} – Permissible stress in metal in direct compression
σ_{sc} – Permissible stress in steel in compression
σ_{st} – Permissible stress in steel in tension
σ_{sv} – Permissible stress in shear reinforcement
τ_{bd} – Design bond stress
τ_c – Shear stress in concrete
$\tau_{c,max}$ – Maximum shear stress in concrete with shear reinforcement
τ_v – Nominal shear stress
ϕ – Diameter of bar

SECTION 5A (b) MATERIALS, WORKMANSHIP, INSPECTION AND TESTING

4 MATERIALS

4.1 Cement

The cement used shall be any of the following conforming to accepted standards [6-5A(3)] and the type selected should be appropriate for the intended use:

a) 33 grade ordinary Portland cement
b) 43 grade ordinary Portland cement
c) 53 grade ordinary Portland cement
d) Rapid hardening Portland cement
e) Portland slag cement
f) Portland pozzolana cement (fly ash based)
g) Portland pozzolana cement (calcined clay based)
h) Hydrophobic cement
j) Low heat Portland cement
k) Sulphate resisting Portland cement

Other combinations of Portland cement with mineral admixture (*see* **4.2**) of quality conforming with relevant Indian Standards laid down may also be used in the manufacture of concrete provided that there are satisfactory data on their suitability, such as performance test on concrete containing them.

4.1.1 Low heat Portland cement conforming to accepted standards [6-5A(3)] shall be used with adequate precautions with regard to removal of formwork, etc.

4.1.2 High alumina cement or supersulphated cement conforming to accepted standards [6-5A(4)] may be used only under special circumstances with the prior approval of the engineer-in-charge. Specialist literature may be consulted for guidance regarding the use of these types of cements.

4.1.3 The attention of the engineer-in-charge and users of cement is drawn to the fact that quality of various cements mentioned in **4.1** is to be determined on the basis of its conformity to the performance characteristics given in the respective Indian Standard Specification for that cement. Any trade-mark or any trade name indicating any special features not covered in the standard or any qualification or other special performance characteristics sometimes claimed/indicated on the bags or containers or in advertisements alongside the 'Statutory Quality Marking' or otherwise have no relation whatsoever with the characteristics guaranteed by the Quality Marking as relevant to that cement. Consumers are, therefore, advised to go by the characteristics as given in the corresponding Indian Standard Specification or seek specialist advise to avoid any problem in concrete making and construction.

4.2 Mineral Admixtures

4.2.1 *Pozzolanas*

Pozzolanic materials conforming to relevant Indian Standards may be used with the permission of the engineer-in-charge, provided uniform blending with cement is ensured.

4.2.1.1 *Fly ash (pulverized fuel ash)*

Fly ash conforming to Grade 1 of accepted standards [6-5A(5)] may be used as part replacement of ordinary Portland cement provided uniform blending with cement is ensured.

4.2.1.2 *Silica fume*

Silica fume conforming to a standard approved by the deciding authority may be used as part replacement of cement provided uniform blending with the cement is ensured.

NOTE — The silica fume (very fine non-crystalline silicon dioxide) is a by-product of the manufacture of silicon, ferrosilicon or the like, from quartz and carbon in electric are furnace. It is usually used in proportion of 5 to 10 percent of the cement content of a mix.

4.2.1.3 *Rice husk ash*

Rice husk ash giving required performance and uniformity characteristics may be used with the approval of the deciding authority.

NOTE — Rice husk ash is produced by burning rice husk and contain large proportion of silica. To achieve amorphous state, rice husk may be burnt at controlled temperature. It is necessary to evaluate the product from a particular source for performance and uniformity since it can be as deleterious as silt when incorporated in concrete. Water demand and drying shrinkage should be studied before using rice husk.

4.2.1.4 *Metakaoline*

Metakaoline having fineness between 700 to 900 m^2/kg may be used as pozzolanic material in concrete.

NOTE — Metakaoline is obtained by clacination of pure or refined kaolintic clay at a temperature between 650°C and 850°C, followed by grinding to achieve a fineness of 700 to 900 m^2/kg. The resulting material has high pozzolanicity.

4.2.2 *Ground Granulated Blast Furnace Slag*

Ground granulated blast furnace slag obtained by grinding granulated blast furnace slag conforming to accepted standards [6-5A(6)] may be used as part replacement of ordinary Portland cements provided uniform blending with cement is ensured.

4.3 Aggregates

Aggregates shall comply with the requirements of accepted standards [6-5A(7)]. As far as possible preference shall be given to natural aggregates.

4.3.1 Other types of aggregates such as slag and crushed overburnt brick or tile, which may be found suitable with regard to strength, durability of concrete and freedom from harmful effects may be used for plain concrete members, but such aggregates should not contain more than 0.5 percent of sulphates as SO_3 and should not absorb more than 10 percent of their own mass of water.

4.3.2 Heavy weight aggregates or light weight aggregates such as bloated clay aggregates and sintered fly ash aggregates may also be used provided the engineer-in-charge is satisfied with the data on the properties of concrete made with them.

NOTE — Some of the provisions of the Code would require modification when these aggregates are used; specialist literature may be consulted for guidance.

4.3.3 *Size of Aggregate*

The nominal maximum size of coarse aggregate should be as large as possible within the limits specified but in no case greater than one-fourth of the minimum thickness of the member, provided that the concrete can be placed without difficulty so as to surround all reinforcement thoroughly and fill the corners of the form. For most work, 20 mm aggregate is suitable. Where there is no restriction to the flow of concrete into sections, 40 mm or larger size may be permitted. In concrete elements with thin sections, closely spaced reinforcement or small cover, consideration should be given to the use of 10 mm nominal maximum size.

Plums above 160 mm and up to any reasonable size may be used in plain concrete work up to a maximum limit of 20 percent by volume of concrete when specifically permitted by the engineer-in-charge. The plums shall be distributed evenly and shall be not closer than 150 mm from the surface.

4.3.3.1 For heavily reinforced concrete members as in the case of ribs of main beams, the nominal maximum size of the aggregate should usually be restricted to 5 mm less than the minimum clear distance between the main bars or 5 mm less than the minimum cover to the reinforcement whichever is smaller.

4.3.4 Coarse and fine aggregate shall be batched separately. All-in-aggregate may be used only where specifically permitted by the engineer-in-charge.

4.4 Water

Water use for mixing and curing shall be clean and free from injurious amounts of oils, acids, alkalis, salts, sugar, organic materials or other substances that may be deleterious to concrete or steel.

Potable water is generally considered satisfactory for mixing concrete. As a guide the following concentrations represent the maximum permissible values:

a) To neutralize 100 ml sample of water, using phenolphthalein as an indicator, it should not require more than 5 ml of 0.02 normal NaOH. The details of test are given in **7.1** of good practice [6-5A(8)].
b) To neutralize 100 ml sample of water, using mixed indicator, it should not require more than 25 ml of 0.02 normal H_2SO_4. The details of test shall be as given in **7** of good practice [6-5A(8)].
c) Permissible limits for solids shall be as given in Table 1.

Table 1 Permissible Limit for Solids

(*Clause* 4.4)

Sl No.		Tested as per	Permissible Limit, *Max*
(1)	(2)	(3)	(4)
i)	Organic	Good practice [6-5A(8)]	200 mg/l
ii)	Inorganic	Good practice [6-5A(8)]	3 000 mg/l
iii)	Sulphates (as SO_3)	Good practice [6-5A(8)]	400 mg/l
iv)	Chlorides (as Cl)	Good practice [6-5A(8)]	2 000 mg/l for concrete not containing embedded steel and 500 mg/l for reinforced concrete work
v)	Suspended matter	Good practice [6-5A(8)]	2 000 mg/l

4.4.1 In case of doubt regarding development of strength, the suitability of water for making concrete shall be ascertained by the compressive strength and initial setting time tests specified in **4.4.1.2** and **4.4.1.3**.

4.4.1.1 The sample of water taken for testing shall represent the water proposed to be used for concreting, due account being paid to seasonal variation. The sample shall not receive any treatment before testing other than that envisaged in the regular supply of water proposed for use in concrete. The sample shall be stored in a clean container previously rinsed out with similar water.

4.4.1.2 Average 28 days compressive strength of at least three 150 mm concrete cubes prepared with water proposed to be used shall not be less than 90 percent of the average of strength of three similar concrete cubes prepared with distilled water. The cubes shall be prepared, cured and tested in accordance with good practice [6-5A(9)].

4.4.1.3 The initial setting time of test block made with the appropriate cement and the water proposed to be used shall not be less than 30 min and shall not differ by ± 30 min from the initial setting time of control test block prepared with the same cement and distilled water. The test blocks shall be prepared and tested in accordance with the good practice [6-5A(10)].

4.4.2 The *p*H value of water shall be not less than 6.

4.4.3 *Sea Water*

Mixing or curing of concrete with sea water is not recommended because of presence of harmful salts in sea water. Under unavoidable circumstances sea water may be used for mixing or curing in plain concrete with no embedded steel after having given due consideration to possible disadvantages and precautions including use of appropriate cement system.

4.4.4 Water found satisfactory for mixing is also suitable for curing concrete. However, water used for curing should not produce any objectionable stain or unsightly deposit on the concrete surface. The presence of tannic acid or iron compounds is objectionable.

4.5 Chemical Admixtures

4.5.1 Admixture, if used shall comply with accepted standards [6-5A(11)]. 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.

4.5.2 Admixtures should not impair durability of concrete nor combine with the constituent to form harmful compounds nor increase the risk of corrosion of reinforcement.

4.5.3 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.

4.5.4 The relative density of liquid admixtures shall be checked for each drum containing admixtures and compared with the specified value before acceptance.

4.5.5 The chloride content of admixtures shall be independently tested for each batch before acceptance.

4.5.6 If two or more admixtures are used simultaneously in the same concrete mix, data should be obtained to assess their interaction and to ensure their compatibility.

4.6 Reinforcement

The reinforcement shall be any of the following conforming to the accepted standards [6-5A(12)]:

a) Mild steel and medium tensile steel bars.
b) High strength deformed steel bars.
c) Hard-drawn steel wire fabric.
d) Grade A of structural steel.

4.6.1 All reinforcement shall be free from loose mill scales, loose rust and coats of paints, oil, mud or any other substances which may destroy or reduce bond. Sand blasting or other treatment is recommended to clean reinforcement.

4.6.2 Special precautions like coating of reinforcement may be required for reinforced concrete elements in exceptional cases and for rehabilitation of structures. Specialist literature may be referred to in such cases.

4.6.3 The modulus of elasticity of steel shall be taken as 200 kN/mm^2. The characteristic yield strength of different steel shall be assumed as the minimum yield stress/0.2 percent proof stress specified in the relevant Indian Standard.

4.7 Storage of Materials

Storage of materials shall be as described in good practice [6-5A(13)].

5 CONCRETE

5.1 Grades

The concrete shall be in grades designated as per Table 2.

Table 2 Grades of Concrete

(*Clauses* 5.1, 8.2.2, 14.1.1 *and* 35.1)

Group	Grade Designation	Specified Characteristic Compressive Strength of 150 mm Cube at 28 days in N/mm^2
(1)	(2)	(3)
Ordinary Concrete	M 10	10
	M 15	15
	M 20	20
Standard Concrete	M 25	25
	M 30	30
	M 35	35
	M 40	40
	M 45	45
	M 50	50
	M 55	55
High Strength Concrete	M 60	60
	M 65	65
	M 70	70
	M 75	75
	M 80	80

NOTES

1 In the designation of concrete mix M refers to the mix and the number of the specified compressive strength of 150 mm size cube at 28 days, expressed in N/mm^2.

2 For concrete of compressive strength greater than M 55, design parameters given in the standard may not be applicable and the values may be obtained from specialized literatures and experimental results.

5.1.1 The characteristic strength is defined as the strength of material below which not more than 5 percent of the test results are expected to fall.

5.1.2 The minimum grade of concrete for plain and reinforced concrete shall be as per Table 5.

5.1.3 Concrete of grades lower than those given in Table 5 may be used for plain concrete constructions, lean concrete, simple foundations, foundation for masonry walls and other simple or temporary reinforced concrete construction.

5.2 Properties of Concrete

5.2.1 *Increase of Strength with Age*

There is normally a gain of strength beyond 28 days. The quantum of increase depends upon the grade and type of cement, curing and environmental conditions, etc. The design should be based on 28 days characteristic strength of concrete unless there is an evidence to justify a higher strength for a particular structure due to age.

5.2.1.1 For concrete of grade M 30 and above, the rate of increase of compressive strength with age shall be based on actual investigations.

5.2.1.2 Where members are subjected to lower direct load during construction, they should be checked for stresses resulting from combination of direct load and bending during construction.

5.2.2 *Tensile Strength of Concrete*

The flexural and splitting tensile strengths shall be obtained in accordance with good practice [6-5A(14)]. When the designer wishes to use an estimate of the tensile strength from the compressive strength, the following formula may be used:

$$\text{Flexural strength, } f_{cr} = 0.7\sqrt{f_{ck}}\ \text{N/mm}^2$$

where f_{ck} is the characteristic cube compressive strength of concrete in N/mm^2.

5.2.3 *Elastic Deformation*

The modulus of elasticity is primarily influenced by the elastic properties of the aggregate and to a lesser extent by the conditions of curing and age of the concrete, the mix proportions and the type of cement. The modulus of elasticity is normally related to the compressive strength of concrete.

5.2.3.1 The modulus of elasticity of concrete can be assumed as follows:

$$E_c = 5\,000\sqrt{f_{ck}}$$

where E_c is the short-term static modulus of elasticity in N/mm^2.

Actual measured values may differ by ±20 percent from the values obtained from the above expression.

5.2.4 *Shrinkage*

The total shrinkage of concrete depends upon the constituents of concrete, size of the member and environmental conditions. For a given humidity and temperature, the total shrinkage of concrete is most influenced by the total amount of water present in the concrete at the time of mixing and, to a lesser extent, by the cement content.

5.2.4.1 In the absence of test data, the approximate value of the total shrinkage strain for design may be taken as 0.000 3 (for more information, *see* accepted standard [6-5A(15)]).

5.2.5 *Creep of Concrete*

Creep of concrete depends, in addition to the factors listed in **5.2.4**, on the stress in the concrete, age at loading and the duration of loading. As long as the stress in concrete does not exceed one-third of its characteristic compressive strength, creep may be assumed to be proportional to the stress.

5.2.5.1 In the absence of experimental data and detailed information on the effect of the variables, the ultimate creep strain may be estimated from the following values of creep coefficient (that is, ultimate creep strain/elastic strain at the age of loading); for long span structure, it is advisable to determine actual creep strain, likely to take place:

Age at Loading	*Creep Coefficient*
7 days	2.2
28 days	1.6
1 year	1.1

NOTE — The ultimate creep strain, estimated as described above does not include the elastic strain.

5.2.6 *Thermal Expansion*

The coefficient of thermal expansion depends on nature of cement, the aggregate, the cement content, the relative humidity and the size of sections.

The value of coefficient of thermal expansion for concrete with different aggregates may be taken as below:

Type of Aggregate	*Coefficient of Thermal Expansion for Concrete/°C*
Quartzite	1.2 to 1.3 to 10^{-5}
Sandstone	0.9 to 1.2 to 10^{-5}
Granite	0.7 to 0.95 to 10^{-5}
Basalt	0.8 to 0.95 to 10^{-5}
Limestone	0.6 to 0.9 to 10^{-5}

6 WORKABILITY OF CONCRETE

6.1 The concrete mix proportions chosen should be such that the concrete is of adequate workability for the placing conditions of the concrete and can properly be compacted with the means available. Suggested ranges of workability of concrete measured in accordance with good practice [6-5A(16)] are given below:

Placing Conditions	*Degree of Workability*	*Slump* mm
(1)	(2)	(3)
Blinding concrete; Shallow sections; Pavements using pavers	Very low	*See* **6.1.1**
Mass concrete; Lightly reinforced sections in slabs, beams, walls, columns; Floors; Hand placed pavements; Canal lining; Strip footings	Low	25-75
Heavily reinforced sections in slabs, beams, walls, columns	Medium	50-100
Slipform work; Pumped concrete	Medium	75-100
Trench fill; *in-situ* piling	High	100-150
Tremie concrete	Very high	*See* **6.1.2**

NOTE — For most of the placing conditions, internal vibrators (needle vibrators) are suitable. The diameter of the needle shall be determined based on the density and spacing of reinforcement bars an thickness of sections. For tremie concrete, vibrators are not required to be used (*see also* **13.3**).

6.1.1 In the 'very low' category of workability where strict control is necessary, for example, pavement quality concrete, measurement of workability by determination of compacting factor will be more appropriate than slump (*see* accepted standard [6-5A(16)]) and a value of compacting factor of 0.75 to 0.80 is suggested.

6.1.2 In the 'very high' category of workability, measurement of workability by determination of flow will be appropriate (*see* accepted standard [6-5A(17)]).

7 DURABILITY OF CONCRETE

7.1 General

A durable concrete is one that performs satisfactorily in the working environment during its anticipated exposure conditions during service. The materials and mix proportions specified and used should be such as to maintain its integrity and, if applicable, to protect embedded metal from corrosion.

7.1.1 One of the main characteristics influencing the durability of concrete is its permeability to the ingress of water, oxygen, carbon dioxide, chloride, sulphate and other potentially deleterious substances. Impermeability is governed by the constituents and workmanship used in making the concrete. With normal-weight aggregates a suitably low permeability is achieved by having an adequate cement content, sufficiently low free water/cement ratio, by ensuring complete compaction of the concrete, and by adequate curing.

The factors influencing durability include:

a) the environment;
b) the cover to embedded steel;
c) the type and quality of constituent materials;
d) the cement content and water/cement ratio of the concrete;

e) workmanship, to obtain full compaction and efficient curing; and
f) the shape and size of the member.

The degree of exposure anticipated for the concrete during its service life together with other relevant factors relating to mix composition, workmanship, design and detailing should be considered. The concrete mix to provide adequate durability under these conditions should be chosen taking account of the accuracy of current testing regimes for control and compliance as described in this Section.

7.2 Requirements for Durability

7.2.1 *Shape and Size of Member*

The shape or design details of exposed structures should be such as to promote good drainage of water and to avoid standing pools and rundown of water. Care should also be taken to minimize any cracks that may collect or transmit water. Adequate curing is essential to avoid the harmful effects of early loss of moisture (*see* **12.5**). Member profiles and their intersections with other members shall be designed and detailed in a way to ensure easy flow of concrete and proper compaction during concreting.

Concrete is more vulnerable to deterioration due to chemical or climatic attack when it is in thin sections, in sections under hydrostatic pressure from one side only, in partially immersed sections and at corners and edges of elements. The life of the structure can be lengthened by providing extra cover to steel, by chamfering the corners or by using circular cross-sections or by using surface coatings which prevent or reduce the ingress of water, carbon dioxide or aggressive chemicals.

7.2.2 *Exposure Conditions*

7.2.2.1 *General environment*

The general environment to which the concrete will be exposed during its working life is classified into five levels of severity, that is, mild, moderate, severe, very severe and extreme as described in Table 3.

7.2.2.2 *Abrasive*

Specialist literatures may be referred to for durability requirements of concrete surfaces exposed to abrasive action, for example, in case of machinery and metal tyres.

7.2.2.3 *Freezing and thawing*

Where freezing and thawing actions under wet conditions exist, enhanced durability can be obtained by the use of suitable air entraining admixtures. When concrete lower than grade M 50 is used under these conditions, the mean total air content by volume of the fresh concrete at the time of delivery into the construction should be:

Nominal Maximum Size Aggregate (mm)	*Entrained Air Percentage*
20	5 ± 1
40	4 ± 1

Since air entrainment reduces the strength, suitable adjustments may be made in the mix design for achieving required strength.

7.2.2.4 *Exposure to sulphate attack*

Table 4 gives recommendations for the type of cement, maximum free water/cement ratio and minimum cement content, which are required at different sulphate

Table 3 Environmental Exposure Conditions

(*Clauses* 7.2.2.1 *and* 34.3.2)

Sl No.	Environment	Exposure Conditions
(1)	(2)	(3)
i)	Mild	Concrete surfaces protected against weather or aggressive conditions, except those situated in coastal area.
ii)	Moderate	Concrete surfaces sheltered from severe rain or freezing whilst wet. Concrete exposed to condensation and rain. Concrete continuously under water. Concrete in contact or buried under non-aggressive soil/ground water. Concrete surfaces sheltered from saturated salt air in coastal area.
iii)	Severe	Concrete surfaces exposed to severe rain, alternate wetting and drying or occasional freezing whilst wet or severe condensation. Concrete completely immersed in sea water. Concrete exposed to coastal environment.
iv)	Very Severe	Concrete surfaces exposed to sea water spray, corrosive fumes or severe freezing conditions whilst wet. Concrete in contact with or buried under aggressive sub-soil/ground water.
v)	Extreme	Surface of members in tidal zone. Members in direct contact with liquid/solid aggressive chemicals.

Table 4 Requirements for Concrete Exposed to Sulphate Attack

(*Clauses* 7.2.2.4 *and* 8.1.2)

Sl No.	Class	Concentration of Sulphates, Expressed as SO_3			Type of Cement	Dense, Fully Compacted Concrete. Made with 20 mm Nominal Maximum Size Aggregates in Accordance with Accepted Standard [6-5A(18)]	
		In Soil		In Ground Water			
		Total SO_3	SO_3 in 2:1 Water: Soil Extract			Minimum Cement Content kg/m^3	Maximum Free Water-Cement Ratio
		Percent	g/l	g/l			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	1	Traces (<0.2)	Less than 1.0	Less than 0.3	Ordinary Portland cement or Portland slag cement or Portland pozzolana cement	280	0.55
ii)	2	0.2 to 0.5	1.0 to 1.9	0.3 to 1.2	Ordinary Portland cement or Portland slag cement or Portland pozzolana cement	330	0.50
					Supersulphated cement or sulphate resisting Portland cement	310	0.50
iii)	3	0.5 to 1.0	1.9 to 3.1	1.2 to 2.5	Supersulphated cement or sulphate resisting Portland cement	330	0.50
					Portland pozzolana cement or Portland slag cement	350	0.45
iv)	4	1.0 to 2.0	3.1 to 5.0	2.5 to 5.0	Supersulphated or sulphate resisting Portland cement	370	0.45
v)	5	More than 2.0	More than 5.0	More than 5.0	Sulphate resisting Portland cement or supersulphated cement with protective coatings	400	0.40

NOTES

1 Cement content given in this table is irrespective of grades of cement.

2 Use of supersulphated cement is generally restricted where the prevailing temperature is above 40°C.

3 Supersulphated cement gives an acceptable life provided that the concrete is dense and prepared with a water-cement ratio of 0.4 or less, in mineral acids, down to *p*H 3.5.

4 The cement contents given in col 6 of this table are the minimum recommended. For SO_3 contents near the upper limit of any class, cement contents above these minimum are advised.

5 For severe conditions, such as thin sections under hydrostatic pressure on one side only and sections partly immersed, considerations should be given to a further reduction of water-cement ratio.

6 Portland slag cement conforming to accepted standard [6-5A(3)] with slag content more than 50 percent exhibits better sulphate resisting properties.

7 Where chloride is encountered along with sulphates in soil or ground water, ordinary Portland cement with C_3A content from 5 to 8 percent shall be desirable to be used in concrete, instead of sulphate resisting cement. Alternatively, Portland slag cement conforming to accepted standard [6-5A(3)] having more than 50 percent slag or a blend of ordinary Portland cement and slag may be used provided sufficient information is available on performance of such blended cements in these conditions.

concentrations in near-neutral ground water having *p*H of 6 to 9.

For the very high sulphate concentrations in Class 5 conditions, some form of lining such as polyethylene or polychloroprene sheet; or surface coating based on asphalt, chlorinated rubber, epoxy; or polyurethane materials should also be used to prevent access by the sulphate solution.

7.2.3 *Requirement of Concrete Cover*

7.2.3.1 The protection of the steel in concrete against corrosion depends upon an adequate thickness of good quality concrete.

7.2.3.2 The nominal cover to the reinforcement shall be provided as per **25.4**.

7.2.4 *Concrete Mix Proportions*

7.2.4.1 *General*

The free water-cement ratio is an important factor in governing the durability of concrete and should always be the lowest value. Appropriate values for minimum cement content and the maximum free water-cement ratio are given in Table 5 for different exposure conditions. The minimum cement content and maximum water-cement ratio apply to 20 mm nominal

Table 5 Minimum Cement Content, Maximum Water-Cement Ratio and Minimum Grade of Concrete for Different Exposures with Normal Weight Aggregates of 20 mm Nominal Maximum Size

(*Clauses* 5.1.2, 5.1.3, 7.2.4.1 *and* 8.1.2)

Sl No.	Exposure	Plain Concrete			Reinforced Concrete		
		Minimum Cement Content kg/m^3	Maximum Free Water-Cement Ratio	Minimum Grade of Concrete	Minimum Cement Content kg/m^3	Maximum Free Water-Cement Ratio	Minimum Grade of Concrete
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Mild	220	0.60	—	300	0.55	M 20
ii)	Moderate	240	0.60	M 15	300	0.50	M 25
iii)	Severe	250	0.50	M 20	320	0.45	M 30
iv)	Very Severe	260	0.45	M 20	340	0.45	M 35
v)	Extreme	280	0.40	M 25	360	0.40	M 40

NOTES

1 Cement content prescribed in this table is irrespective of the grades of cement and it is inclusive of additions mentioned in **4.2**. The additions such as fly ash or ground granulated blast furnace slag may be taken into account in the concrete composition with respect to the cement content and water-cement ratio if the suitability is established and as long as the maximum amounts taken into account do not exceed the limit of pozzolana and slag specified in accordance with accepted standard [6-5A(19)].

2 Minimum grade for plain concrete under mild exposure condition is not specified.

maximum size aggregate. For other sizes of aggregate they should be changed as given in Table 6.

Table 6 Adjustments to Minimum Cement Contents for Aggregates Other Than 20 mm Nominal Maximum Size

(*Clause* 7.2.4.1)

Sl No.	Nominal Maximum Aggregate Size mm	Adjustments to Minimum Cement Contents Given in Table 5 kg/m^3
(1)	(2)	(3)
i)	10	+ 40
ii)	20	0
iii)	40	–30

7.2.4.2 *Maximum cement content*

Cement content not including fly ash and ground granulated blast furnace slag in excess of 450 kg/m^3 should not be used unless special consideration has been given in design to the increased risk of cracking due to drying shrinkage in thin sections, or to early thermal cracking and to the increased risk of damage due to alkali silica reactions.

7.2.5 *Mix Constituents*

7.2.5.1 *General*

For concrete to be durable, careful selection of the mix and materials is necessary, so that deleterious constituents do not exceed the limits.

7.2.5.2 *Chlorides in concrete*

Whenever there is chloride in concrete there is an increased risk of corrosion of embedded metal. The higher the chloride content, or if subsequently exposed to warm moist conditions, the greater the risk of corrosion. All constituents may contain chlorides and concrete may be contaminated by chlorides from the external environment. To minimize the chances of deterioration of concrete from harmful chemical salts, the levels of such harmful salts in concrete coming from concrete materials, that is, cement, aggregates water and admixtures, as well as by diffusion from the environment should be limited. The total amount of chloride content (as Cl) in the concrete at the time of placing shall be as given in Table 7.

The total acid soluble chloride content should be calculated from the mix proportions and the measured chloride contents of each of the constituents. Wherever possible, the total chloride content of the concrete should be determined.

Table 7 Limits of Chloride Content of Concrete

(*Clause* 7.2.5.2)

Sl No.	Type or Use of Concrete	Maximum Total Acid Soluble Chloride Content Expressed as kg/m^3 of Concrete
(1)	(2)	(3)
i)	Concrete containing metal and steam cured at elevated temperature and prestressed concrete	0.4
ii)	Reinforced concrete or plain concrete containing embedded metal	0.6
iii)	Concrete not containing embedded metal or any material requiring protection from chloride	3.0

7.2.5.3 *Sulphates in concrete*

Sulphates are present in most cements and in some aggregates; excessive amounts of water-soluble sulphate from these or other mix constituents can cause expansion and disruption of concrete. To prevent this, the total water-soluble sulphate content of the concrete mix, expressed as SO_3, should not exceed 4 percent by mass of the cement in the mix. The sulphate content should be calculated as the total from the various constituents of the mix.

The 4 percent limit does not apply to concrete made with supersulphated cement complying with accepted standard [6-5A(20)].

7.2.5.4 *Alkali-aggregate reaction*

Some aggregates containing particular varieties of silica may be susceptible to attack by alkalis (Na_2O and K_2O) originating from cement or other sources, producing an expansive reaction which can cause cracking and disruption of concrete. Damage to concrete from this reaction will normally only occur when all the following are present together:

a) A high moisture level, within the concrete;
b) A cement with high alkali content, or another source of alkali; and
c) Aggregate containing an alkali reactive constituent.

Where the service records of particular cement/aggregate combination are well established, and do not include any instances of cracking due to alkali-aggregate reaction, no further precautions should be necessary. When the materials are unfamiliar, precautions should take one or more of the following form:

a) Use of non-reactive aggregate from alternate sources.
b) Use of low alkali ordinary Portland cement having total alkali content not more than 0.6 percent (as Na_2O equivalent).

   Further advantage can be obtained by use of fly ash (Grade 1) conforming to accepted standard [6-5A(5)] or granulated blastfurnace slag conforming to accepted standard [6-5A(5)] as part replacement of ordinary Portland cement (having total alkali content as Na_2O equivalent not more than 0.6 percent), provided fly ash content is at least 20 percent or slag content is at least 50 percent.
c) Measures to reduce the degree of saturation of the concrete during service, such as use of impermeable membranes.
d) Limiting the cement content in the concrete mix and thereby limiting total alkali content in the concrete mix. For more guidance specialist literatures may be referred.

7.2.6 *Concrete in Aggressive Soils and Water*

7.2.6.1 *General*

The destructive action of aggressive waters on concrete is progressive. The rate of deterioration decreases as the concrete is made stronger and more impermeable, and increases as the salt content of the water increases. Where structures are only partially immersed or are in contact with aggressive soils or waters on one side only, evaporation may cause serious concentrations of salts with subsequent deterioration, even where the original salt content of the soil or water is not high.

NOTE — Guidance regarding requirements for concrete exposed to sulphate attack is given in **7.2.2.4**.

7.2.6.2 *Drainage*

At sites where alkali concentrations are high or may become very high, the ground water should be lowered by drainage so that it will not come into direct contact with the concrete.

Additional protection may be obtained by the use of chemically resistant stone facing or a layer of plaster of Paris covered with suitable fabric, such as jute thoroughly impregnated with bituminous material.

7.2.7 *Compaction, Finishing and Curing*

Adequate compaction without segregation should be ensured by providing suitable workability and by employing appropriate placing and compacting equipment and procedures. Full compaction is particularly important in the vicinity of construction and movement joints and of embedded water bars and reinforcement.

Good finishing practices are essential for durable concrete.

Overworking the surface and the addition of water/cement to aid in finishing should be avoided; the resulting laitance will have impaired strength and durability and will be particularly vulnerable to freezing and thawing under wet conditions.

It is essential to use proper and adequate curing techniques to reduce the permeability of the concrete and enhance its durability by extending the hydration of the cement, particularly in its surface zone (*see* **12.5**).

7.2.8 *Concrete in Sea-water*

Concrete in sea-water or exposed directly along the sea-coast shall be at least M 20 Grade in the case of plain concrete and M 30 in case of reinforced concrete.

The use of slag or pozzolana cement is advantageous under such conditions.

7.2.8.1 Special attention shall be given to the design of the mix to obtain the densest possible concrete; slag, broken brick, soft limestone, soft sandstone, or other porous or weak aggregates shall not be used.

7.2.8.2 As far as possible, preference shall be given to precast members unreinforced, well-cured and hardened, without sharp corners, and having trowel-smooth finished surfaces free from crazing, cracks or other defects; plastering should be avoided.

7.2.8.3 No construction joints shall be allowed within 600 mm below low water-level or within 600 mm of the upper and lower planes of wave action. Where unusually severe conditions or abrasion are anticipated, such parts of the work shall be protected by bituminous or silico-flouride coatings or stone facing bedded with bitumen.

7.2.8.4 In reinforced concrete structures, care shall be taken to protect the reinforcement from exposure to saline atmosphere during storage, fabrication and use. It may be achieved by treating the surface of reinforcement with cement wash or by suitable methods.

8 CONCRETE MIX PROPORTIONING

8.1 Mix Proportion

The mix proportion shall be selected to ensure the workability of the fresh concrete and when concrete is hardened, it shall have the required strength, durability and surface finish.

8.1.1 The determination of the proportions of cement, aggregates and water to attain the required strengths shall be made as follows:

a) By designing the concrete mix; such concrete shall be called 'Design mix concrete', or
b) By adopting nominal concrete mix; such concrete shall be called 'Nominal mix concrete'.

Design mix concrete is preferred to nominal mix. If design mix concrete cannot be used for any reason on the work for grades of M 20 or lower, nominal mixes may be used with the permission of engineer-in-charge, which, however, is likely to involve a higher cement content.

8.1.2 *Information Required*

In specifying a particular grade of concrete, the following information shall be included:

a) Type of mix, that is, design mix concrete or nominal mix concrete;
b) Grade designation;
c) Type of cement;
d) Maximum nominal size of aggregate;
e) Minimum cement content (for design mix concrete);
f) Maximum water-cement ratio;
g) Workability;
h) Mix proportion (for nominal mix concrete);
j) Exposure conditions as per Tables 4 and 5;
k) Maximum temperature of concrete at the time of placing;
m) Method of placing; and
n) Degree of supervision.

8.1.2.1 In appropriate circumstances, the following additional information may be specified;

a) Type of aggregate,
b) Maximum cement content, and
c) Whether an admixture shall or shall not be used and the type of admixture and the condition of use.

8.2 Design Mix Concrete

8.2.1 As the guarantor of quality of concrete used in the construction, the constructor shall carry out the mix design and the mix so designed (not the method of design) shall be approved by the employer within the limitations of parameters and other stipulations laid down by this standard.

8.2.2 The mix shall be designed to produce the grade of concrete having the required workability and a characteristic strength not less than appropriate values given in Table 2. The target mean strength of concrete mix should be equal to the characteristic strength plus 1.65 times the standard deviation.

8.2.3 Mix design done earlier not prior to one year may be considered adequate for later work provided there is no change in source and the quality of the materials.

8.2.4 *Standard Deviation*

The standard deviation for each grade of concrete shall be calculated, separately.

8.2.4.1 Standard deviation based on test strength of sample

a) *Number of test results of samples* — The total number of test strength of samples required to constitute an acceptable record for calculation of standard deviation shall be not less than 30. Attempts should be made to obtain the 30 samples, as early as possible, when a mix is used for the first time.

b) *In case of significant changes in concrete* — When significant changes are made in the production of concrete batches (for example changes in the materials used, mix design, equipment or technical control), the standard deviation value shall be separately calculated for such batches of concrete.

c) *Standard deviation to be brought up to date* — The calculation of the standard deviation shall be brought up to date after every change of mix design.

8.2.4.2 *Assumed standard deviation*

Where sufficient test results for a particular grade of concrete are not available, the value of standard deviation given in Table 8 may be assumed for design of mix in the first instance. As soon as the results of samples are available, actual calculated standard deviation shall be used and the mix designed properly. However, when adequate past records for a similar grade exist and justify to the designer a value of standard deviation different from that shown in Table 8, it shall be permissible to use that value.

Table 8 Assumed Standard Deviation
(*Clause* 8.2.4.2 and *Table* 11)

Grade of Concrete	Assumed Standard Deviation N/mm^2
(1)	(2)
M 10, M 15	3.5
M 20, M 25	4.0
M 30, M 35, M 40, M 45, M 50	5.0

NOTE — The above values correspond to the site control having proper storage of cement; weigh batching of all materials; controlled addition of water; regular checking of all materials, aggregate gradings and moisture content; and periodical checking of workability and strength. Where there is deviation from the above the values given in the above table shall be increased by 1 N/mm^2.

8.3 Nominal Mix Concrete

Nominal mix concrete may be used for concrete of M 20 or lower. The proportions of materials for nominal mix concrete shall be in accordance with Table 9.

8.3.1 The cement content of the mix specified in Table 9 for any nominal mix shall be proportionately increased if the quantity of water in a mix has to be increased to overcome the difficulties of placement and compaction, so that the water-cement ratio as specified is not exceeded.

9 PRODUCTION OF CONCRETE

9.1 Quality Assurance Measures

9.1.1 In order that the properties of the completed structure be consistent with the requirements and the assumptions made during the planning and the design, adequate quality assurance measures shall be taken. The construction should result in satisfactory strength, serviceability and long-term durability so as to lower the overall life-cycle cost. Quality assurance in construction activity relates to proper design, use of adequate materials and components to be supplied by the producers, proper workmanship in the execution of works by the contractor and ultimately proper care during the use of structure including timely maintenance and repair by the owner.

9.1.2 Quality assurance measures are both technical and organizational. Some common cases should be specified in a general Quality Assurance Plan which shall identify the key elements necessary to provide fitness of the structure and the means by which they are to be provided and measured with the overall purpose to provide confidence that the realized project will work satisfactorily in service fulfilling intended needs. The job of quality control and quality assurance would involve quality audit of both the inputs as well as the outputs. Inputs are in the form of materials for concrete; workmanship in all stages of batching, mixing, transportation, placing, compaction and curing; and the related plant, machinery and equipments; resulting in the output in the form of concrete in place. To ensure proper performance, it is necessary that each step in concreting which will be covered by the next step is inspected as the work proceeds (*see also* **16**).

9.1.3 Each party involved in the realization of a project should establish and implement a Quality Assurance Plan, for its participation in the project. Supplier's and subcontractor's activities shall be covered in the plan. The individual Quality Assurance Plans shall fit into the general Quality Assurance Plan. A Quality Assurance Plan shall define the tasks and responsibilities of all persons involved, adequate control and checking procedures, and the organization and maintaining adequate documentation of the building process and its results. Such documentation should generally include:

a) test reports and manufacturer's certificate for materials, concrete mix design details;

b) pour cards for site organization and clearance for concrete placement;

c) record of site inspection of workmanship, field tests;

d) non-conformance reports, change orders;

Table 9 Proportions for Nominal Mix Concrete

(*Clauses* 8.3 *and* 8.3.1)

Grade of Concrete	Total Quantity of Dry Aggregates by Mass per 50 kg of Cement, to be Taken as the Sum of the Individual Masses of Fine and Coarse Aggregates, kg, *Max*	Proportion of Fine Aggregate to Coarse Aggregate (by Mass)	Quantity of Water per 50 kg of Cement, *Max*
(1)	(2)	(3)	(4)
M 5	800	Generally 1:2 but subject to an upper limit of 1:1½ and a lower limit of 1:2½	60
M 7.5	625		45
M 10	480		34
M 15	330		32
M 20	250		30

NOTE — The proportion of the fine to coarse aggregates should be adjusted from upper limit to lower limit progressively as the grading of fine aggregates becomes finer and the maximum size of coarse aggregate becomes larger. Graded coarse aggregate shall be used.

Example

For an average grading of fine aggregate (that is, Zone II of Table 4 of IS 383), the proportions shall be 1:1½, 1:2 and 1:2½ for maximum size of aggregates 10 mm, 20 mm and 40 mm respectively.

e) quality control charts; and
f) statistical analysis.

NOTE — Quality control charts are recommended wherever the concrete is in continuous production over considerable period.

9.2 Batching

To avoid confusion and error in batching, consideration should be given to using the smallest practical number of different concrete mixes on any site or in any one plant. In batching concrete, the quantity of both cement and aggregate shall be determined by mass; admixture, if solid, by mass; liquid admixture may, however, be measured in volume or mass; water shall be weighed or measured by volume in a calibrated tank (*see also* accepted standard [6-5A(21)]).

Ready-mixed concrete supplied by ready-mixed concrete plant shall be preferred. For large and medium project sites the concrete shall be sourced from ready-mixed concrete plants or from on site or off site batching and mixing plants (*see also* accepted standard [6-5A(21)]).

9.2.1 Except where it can be shown to the satisfaction of the engineer-in-charge that supply of properly graded aggregate of uniform quality can be maintained over a period of work, the grading of aggregate should be controlled by obtaining the coarse aggregate in different sizes and blending them in the right proportions when required, the different sizes being stocked in separate stock-piles. The material should be stock-piled for several hours preferably a day before use. The grading of coarse and fine aggregate should be checked as frequently as possible, the frequency for a given job being determined by the engineer-in-charge to ensure that the specified grading is maintained.

9.2.2 The accuracy of the measuring equipment shall be within ± 2 percent of the quantity of cement being measured and within ± 3 percent of the quantity of aggregate, admixtures and water being measured.

9.2.3 Proportion/Type and grading of aggregates shall be made by trial in such a way so as to obtain densest possible concrete. All ingredients of the concrete should be used by mass only.

9.2.4 Volume batching may be allowed only where weigh-batching is not practical and provided accurate bulk densities of materials to be actually used in concrete have earlier been established. Allowance for bulking shall be made in accordance with accepted standard [6-5A(23)]. The mass volume relationship should be checked as frequently as necessary, the frequency for the given job being determined by engineer-in-charge to ensure that the specified grading is maintained.

9.2.5 It is important to maintain the water-cement ratio constant at its correct value. To this end, determination of moisture contents in both fine and coarse aggregates shall be made as frequently as possible, the frequency for a given job being determined by the engineer-in-charge according to weather conditions. The amount of the added water shall be adjusted to compensate for any observed variations in the moisture contents. For the determination of moisture content in the aggregates, accepted standard [6-5A(23)] may be referred to. To allow for the variation in mass of aggregate due to variation in their moisture content, suitable adjustments in the masses of aggregates shall also be made. In the absence of exact data, only in the case of nominal mixes, the amount of surface water may be estimated from the values given in Table 10.

Table 10 Surface Water Carried by Aggregate
(*Clause* 9.2.5)

Sl No.	Aggregate	Approximate Quantity of Surface Water	
		Percent by Mass	l/m[2]
(1)	(2)	(3)	(4)
i)	Very wet sand	7.5	120
ii)	Moderately wet sand	5.0	80
iii)	Moist sand	2.5	40
iv)	Moist gravel or crushed rock[1)]	1.25 – 2.5	20 – 40

[1)] Coarser the aggregate, less the water it will carry.

9.2.6 No substitutions in materials used on the work or alterations in the established proportions, except as permitted in **9.2.4** and **9.2.5** shall be made without additional tests to show that the quality and strength of concrete are satisfactory.

9.3 Mixing

Concrete shall be mixed in a mechanical mixer. The mixer should comply with accepted standard [6-5A(24)]. The mixers shall be fitted with water measuring (metering) devices. The mixing shall be continued until there is a uniform distribution of the materials and the mass is uniform in colour and consistency. If there is segregation after unloading from the mixer, the concrete should be re-mixed.

9.3.1 For guidance, the mixing time shall be at least 2 min. For other types of more efficient mixers, manufacturers recommendations shall be followed; for hydrophobic cement it may be decided by the engineer-in-charge.

9.3.2 Workability should be checked at frequent intervals (*see* accepted standard [6-5A(16)]).

9.3.3 Dosages of retarders, plasticizers and superplasticizers shall be restricted to 0.5, 1.0 and 2.0 percent respectively by weight of cementitious materials, unless a higher value is agreed upon between the manufacturer and the constructor based on performance test.

10 FORMWORK

10.1 General

The formwork shall be designed and constructed so as to remain sufficiently rigid during placing and compaction of concrete and shall be such as to prevent loss of slurry from the concrete. For further details regarding design, detailing, etc, reference may be made to good practice [6-5A(25)]. The tolerances on the shapes, lines and dimensions shown in the drawing shall be within the limits given below:

a) Deviation from specified dimensions of cross-section of columns and beams — $^{+12}_{-6}$ mm
b) Deviation from dimensions of footings
   1) Dimensions in plan — $^{+50}_{-12}$ mm
   2) Eccentricity — 0.02 times the width of the footing in the direction of deviation but not more than 50 mm
   3) Thickness — ± 0.05 times the specified thickness

These tolerances apply to concrete dimensions only, and not to positioning of vertical reinforcing steel or dowels.

10.2 Cleaning and Treatment of Formwork

All rubbish, particularly, chippings, shavings and sawdust shall be removed from the interior of the forms before the concrete is placed. The face of formwork in contact with the concrete shall be cleaned and treated with form release agent. Release agents should be applied so as to provide a thin uniform coating to the forms without coating the reinforcement.

10.3 Stripping Time

Forms shall not be released until the concrete has achieved a strength of at least twice the stress to which the concrete may be subjected at the time of removal of formwork. The strength referred to shall be that of concrete using the same cement and aggregates and admixture, if any, with the same proportions and cured under conditions of temperature and moisture similar to those existing on the work.

10.3.1 While the above criteria of strength shall be the guiding factor for removal of formwork, in normal circumstances where ambient temperature does not fall below 15°C and where ordinary Portland cement is used and adequate curing is done, following striking period may deem to satisfy the guideline given in **10.3**:

	Type of Formwork	*Minimum Period Before Striking Formwork*
a)	Vertical formwork to columns, walls, beams	16-24 h
b)	Soffit formwork to slabs (Props to be refixed immediately after removal of formwork)	3 days
c)	Soffit formwork to beams (Props to be refixed immediately after removal of formwork)	7 days
d)	Props to slabs:	
	1) Spanning up to 4.5 m	7 days
	2) Spanning over 4.5 m	14 days
e)	Props to beams and arches:	
	1) Spanning up to 6 m	14 days
	2) Spanning over 6 m	21 days

For other cements and lower temperature, the stripping time recommended above may be suitably modified.

10.3.2 The number of props left under, their sizes and disposition shall be such as to be able to safely carry the full dead load of the slab, beam or arch as the case may be together with any live load likely to occur during curing or further construction.

10.3.3 Where the shape of the element is such that the formwork has re-entrant angles, the formwork shall be removed as soon as possible after the concrete has set, to avoid shrinkage cracking occurring due to the restraint imposed.

11 ASSEMBLY OF REINFORCEMENT

11.1 Reinforcement shall be bent and fixed in accordance with procedure specified in good practice [6-5A(26)]. The high strength deformed steel bars should not be re-bent or straightened without the approval of engineer-in-charge.

Bar bending schedules shall be prepared for all reinforcement work.

11.2 All reinforcement shall be placed and maintained in the position shown in the drawings by providing proper cover blocks, spacers, supporting bars, etc.

11.2.1 Crossing bars should not be tack-welded for assembly of reinforcement unless permitted by engineer-in-charge.

11.3 Placing of Reinforcement

Rough handling, shock loading (prior to embedment) and the dropping of reinforcement from a height should be avoided. Reinforcement should be secured against displacement outside the specified limits.

11.3.1 *Tolerances on Placing of Reinforcement*

Unless otherwise specified by engineer-in-charge, the reinforcement shall be placed within the following tolerances:

a) for effective depth 200 mm or less ± 10 mm
b) for effective depth more than 200 mm ± 15 mm

12.3.2 *Tolerance for Cover*

Unless specified otherwise, actual concrete cover should not deviate from the required nominal cover by $^{+10}_{-\ 0}$ mm.

Nominal cover as given in **25.4.1** should be specified to all steel reinforcement including links. Spacers between the links (or the bars where no links exist) and the formwork should be of the same nominal size as the nominal cover.

Spacers, chairs and other supports detailed on drawings, together with such other supports as may be necessary, should be used to maintain the specified nominal cover to the steel reinforcement. Spacers or chairs should be placed at a maximum spacing of 1 m and closer spacing may sometimes be necessary.

Spacers, cover blocks should be of concrete of same strength or PVC.

11.4 Welded Joints or Mechanical Connections

Welded joints or mechanical connections in reinforcement may be used but in all cases of important connections, test shall be made to prove that the joints are of the full strength of bars connected. Welding of reinforcements shall be done in accordance with good practice [6-5A(27)].

11.5 Where reinforcement bars up to 12 mm for high strength deformed steel bars and up to 16 mm for mild steel bars are bent aside at construction joints and afterwards bent back into their original positions, care should be taken to ensure that at no time is the radius of the bend less than 4 bar diameters for plain mild steel or 6 bar diameters for deformed bars. Care shall also be taken when bending back bars, to ensure that the concrete around the bar is not damaged beyond the band.

11.6 Reinforcement should be placed and tied in such a way that concrete placement be possible without segregation of the mix. Reinforcement placing should allow compaction by immersion vibrator. Within the concrete mass, different types of metal in contact should be avoided to ensure that bimetal corrosion does not take place.

12 TRANSPORTING, PLACING, COMPACTION AND CURING

12.1 Transporting and Handling

After mixing, concrete shall be transported to the formwork as rapidly as possible by methods which will prevent the segregation or loss of any of the ingredients or ingress of foreign matter or water and maintaining the required workability.

12.1.1 During hot or cold weather, concrete shall be transported in deep containers. Other suitable methods to reduce the loss of water by evaporation in hot weather and heat loss in cold weather may also be adopted.

12.2 Placing

The concrete shall be deposited as nearly as practicable in its final position to avoid rehandling. The concrete shall be placed and compacted before initial setting of concrete commences and should not be subsequently disturbed. Methods of placing should be such as to preclude segregation. Care should be taken to avoid displacement of reinforcement or movement of formwork. As a general guidance, the maximum permissible free fall of concrete may be taken as 1.5 m.

12.3 Compaction

Concrete should be thoroughly compacted and fully worked around the reinforcement, around embedded fixtures and into corners of the formwork.

12.3.1 Concrete shall be compacted using mechanical vibrators complying with accepted standard [6-5A(28)]. Over vibration and under vibration of concrete are harmful and should be avoided. Vibration of very wet mixes should also be avoided.

Whenever vibration has to be applied externally, the design of formwork and the disposition of vibrators should receive special consideration to ensure efficient compaction and to avoid surface blemishes.

12.4 Construction Joints and Cold Joints

Joints are a common source of weakness and, therefore, it is desirable to avoid them. If this is not possible, their number shall be minimized. Concreting shall be carried out continuously up to construction joints, the position and arrangement of which shall be indicated by the designer. Construction joints should comply with accepted standard [6-5A(29)].

Construction joints shall be placed at accessible locations to permit cleaning out of laitance, cement slurry and unsound concrete, in order to create rough/uneven surface. It is recommended to clean out laitance and cement slurry by using wire brush on the surface of joint immediately after initial setting of concrete and to clean out the same immediately thereafter. The prepared surface should be in a clean saturated surface dry condition when fresh concrete is placed, against it.

In the case of construction joints at locations where the previous pour has been cast against shuttering the recommended method of obtaining a rough surface for the previously poured concrete is to expose the aggregate with a high pressure water jet or any other appropriate means.

Fresh concrete should be thoroughly vibrated near construction joints so that mortar from the new concrete flows between large aggregates and develop proper bond with old concrete.

Where high shear resistance is required at the construction joints, shear keys may be provided.

Sprayed curing membranes and release agents should be thoroughly removed from joint surfaces.

12.5 Curing

Curing is the process of preventing the loss of moisture from the concrete whilst maintaining a satisfactory temperature regime. The prevention of moisture loss from the concrete is particularly important if the water-cement ratio is low, if the cement has a high rate of strength development, if the concrete contains granulated blast furnace slag or pulverized fuel ash. The curing regime should also prevent the development of high temperature gradients within the concrete.

The rate of strength development at early ages of concrete made with supersulphated cement is significantly reduced at lower temperatures. Supersulphated cement concrete is seriously affected by inadequate curing and the surface has to be kept moist for at least seven days.

12.5.1 *Moist Curing*

Exposed surfaces of concrete shall be kept continuously in a damp or wet condition by ponding or by covering with a layer of sacking, canvas, hessian or similar materials and kept constantly wet for at least seven days from the date of placing concrete in case of ordinary Portland Cement and at least 10 days where mineral admixtures or blended cements are used. The period of curing shall not be less than 10 days for concrete exposed to dry and hot weather conditions. In the case of concrete where mineral admixtures or blended cements are used, it is recommended that above minimum periods may be extended to 14 days.

12.5.2 *Membrane Curing*

Approved curing compounds may be used in lieu of moist curing with the permission of the engineer-in-charge. Such compounds shall be applied to all exposed surfaces of the concrete as soon as possible after the concrete has set. Impermeable membranes such as polyethylene sheeting covering closely the concrete surface may also be used to provide effective barrier against evaporation.

12.6 Supervision

It is exceedingly difficult and costly to alter concrete once placed. Hence, constant and strict supervision of all the items of the construction is necessary during the progress of the work, including the proportioning and mixing of the concrete. Supervision is also of extreme importance to check the reinforcement and its placing before being covered.

12.6.1 Before any important operation, such as concreting or stripping of the formwork is started, adequate notice shall be given to the construction supervisor.

13 CONCRETING UNDER SPECIAL CONDITIONS

13.1 Work in Extreme Weather Conditions

During hot or cold weather, the concreting should be done as per good practice [6-5A(30)].

13.2 Under-Water Concreting

13.2.1 When it is necessary to deposit concrete under water, the methods, equipment, materials and proportions of the mix to be used shall be submitted to and approved by the engineer-in-charge before the work is started.

13.2.2 Under-water concrete should have a slump recommended in **6.1**. The water-cement ratio shall not exceed 0.6 and may need to be smaller, depending on the grade of concrete or the type of chemical attack. For aggregates of 40 mm maximum particle size, the cement content shall be at least 350 kg/m^3 of concrete.

13.2.3 Coffer-dams or forms shall be sufficiently tight to ensure still water if practicable, and in any case to reduce the flow of water to less than 3 m/min through the space into which concrete is to be deposited. Coffer-dams or forms in still water shall be sufficiently tight to prevent loss of mortar through the walls. De-watering by pumping shall not be done while concrete is being placed or until 24 h thereafter.

13.2.4 Concrete cast under water should not fall freely through the water. Otherwise it may be leached and become segregated. Concrete shall be deposited continuously until it is brought to the required height. While depositing, the top surface shall be kept as nearly level as possible and the formation of seams avoided. The methods to be used for depositing concrete under water shall be one of the following:

a) *Tremie* — The concrete is placed through vertical pipes the lower end of which is always inserted sufficiently deep into the concrete which has been placed previously but has not set. The concrete emerging from the pipe pushes the material that has already been placed to the side and upwards and thus does not come into direct contact with water.

   When concrete is to be deposited under-water by means of tremie, the top section of the tremie shall be a hopper large enough to hold one entire batch of the mix or the entire contents the transporting bucket, if any. The tremie pipe shall be not less than 200 mm in diameter and shall be large enough to allow a free flow of concrete and strong enough to withstand the external pressure of the water in which it is suspended, even if a partial vacuum develops inside the pipe. Preferably, flanged steel pipe of adequate strength for the job should be used. A separate lifting device shall be provided for each tremie pipe with its hopper at the upper end. Unless the lower end of the pipe is equipped with an approved automatic check valve, the upper end of the pipe shall be plugged with a wadding of the gunny sacking or other approved material before delivering the concrete to the tremie pipe through the hopper, so that when the concrete is forced down from the hopper to the pipe, it will force the plug (and along with it any water in the pipe) down the pipe and out of the bottom end, thus establishing a continuous stream of concrete. It will be necessary to raise slowly the tremie in order to cause a uniform flow of the concrete, but the tremie shall not be emptied so that water enters the pipe. At all times after the placing of concrete is started and until all the concrete is placed, the lower end of the tremie pipe shall be below the top surface of the plastic concrete. This will cause the concrete to build up from below instead of flowing out over the surface, and thus avoid formation of laitance layers. If the charge in the tremie is lost while depositing, the tremie shall be raised above the concrete surface, and unless sealed by a check valve, it shall be re-plugged at the top end, as at the beginning, before refilling for depositing concrete.

b) *Direct placement with pumps* — As in the case of the tremie method, the vertical end piece of the pipe line is always inserted sufficiently deep into the previously cast concrete and should not move to the side during pumping.

c) *Drop bottom bucket* — The top of the bucket shall be covered with a canvas flap. The bottom doors shall open freely downward and outward when tripped. The bucket shall be filled completely and lowered slowly to avoid backwash. The bottom doors shall not be opened until the bucket rests on the surface upon which the concrete is to be deposited and when discharged, shall be withdrawn slowly until well above the concrete.

d) *Bags* — Bags of at least 0.028 m^3 capacity of jute or other coarse cloth shall be filled about two-thirds full of concrete, the spare end turned under so that bag is square ended and securely tied. They shall be placed carefully in header and stretcher courses so that the whole mass is interlocked. Bags used for this purpose shall be free from deleterious materials.

e) *Grouting* — A series of round cages made from 50 mm mesh of 6 mm steel and extending over the full height to be concreted shall be prepared and laid vertically over the area to be concreted so that the distance between centres of the cages and also to the faces of the concrete shall not exceed 1 m. Stone aggregate of not less than 50 mm nor more than 200 mm size shall be deposited outside the steel cages over the full area and height to be concreted with due care to prevent displacement of the cages.

   A stable 1:2 cement-sand grout with a water-cement ratio of not less than 0.6 and not more than 0.8 shall be prepared in a mechanical mixer and sent down under pressure (about 0.2 N/mm^2) through 38 to 50 mm diameter pipes terminating into steel cages, about 50 mm above the bottom of the concrete. As the grouting proceeds, the pipe shall be raised gradually up to a height of not more than 6 000 mm above its starting level after which it may be withdrawn and placed into the next cage for further grouting by the same procedure.

   After grouting the whole area for a height of about 600 mm, the same operation shall be repeated, if necessary, for the next layer of 600 mm and so on.

   The amount of grout to be sent down shall be sufficient to fill all the voids which may be either ascertained or assumed as 55 percent of the volume to be concreted.

13.2.5 To minimize the formulation of laitance, great care shall be exercised not to disturb the concrete as far as possible while it is being deposited.

14 SAMPLING AND STRENGTH OF DESIGNED CONCRETE MIX

14.1 General

Samples from fresh concrete shall be taken as per accepted standard [6-5A(16)] and cubes shall be made, cured and tested at 28 days in accordance with accepted standard [6-5A(9)].

14.1.1 In order to get a relatively quicker idea of the quality of concrete, optional tests on beams for modulus of rupture at 72 ± 2 h or at 7 days, or compressive strength tests at 7 days may be carried out in addition to 28 days compressive strength test. For this purpose the values should be arrive at based on actual testing. In all cases, the 28 days compressive strength specified in Table 2 shall alone be the criterion for acceptance or rejection of the concrete.

14.2 Frequency of Sampling

14.2.1 *Sampling Procedure*

A random sampling procedure shall be adopted to ensure that each concrete batch shall have a reasonable chance of being tested that is, the sampling should be spread over the entire period of concreting and cover all mixing units.

14.2.2 *Frequency*

The minimum frequency of sampling of concrete of each grade shall be in accordance with the following:

Quantity of Concrete in the Work, m^3	*Number of Samples*
1-5	1
6-15	2
16-30	3
31-50	4
51 and above	4 plus one additional sample for each additional 50 m^3 or part thereof

NOTE — At least one sample shall be taken from each shift. Where concrete is produced at continuous production unit, such as ready-mixed concrete plant, frequency of sampling may be agreed upon mutually by suppliers and purchasers.

14.3 Test Specimen

Three test specimens shall be made for each sample for testing at 28 days. Additional specimens may be required for various purposes such as to determine the strength of concrete at 7 days or at the time of striking the formwork, or to determine the duration of curing, or to check the testing error. Additional specimens may also be required for testing specimens cured by accelerated methods as described in accepted standard [6-5A(31)]. The specimen shall be tested as described in accepted standard [6-5A(9)].

14.4 Test Results of Sample

The test results of the sample shall be the average of the strength of three specimens. The individual variation should not be more than ± 15 percent of the average. If more, the test results of the sample are invalid.

15 ACCEPTANCE CRITERIA

15.1 Compressive Strength

The concrete shall be deemed to comply with the strength requirements when both the following conditions are met:

a) The mean strength determined from any group of four non-overlapping consecutive test results complies with the appropriate limits in col 2 of Table 11.
b) Any individual test result complies with the appropriate limits in col 3 of Table 11.

15.2 Flexural Strength

When both the following conditions are met, the concrete complies with the specified flexural strength:

a) The mean strength determined from any group of four consecutive test results exceeds the specified characteristic strength by at least 0.3 N/mm^2.
b) The strength determined from any test result is not less than the specified characteristic strength less 0.3 N/mm^2.

15.3 Quantity of Concrete Represented by Strength Test Results

The quantity of concrete represented by a group of four consecutive test results shall include the batches from which the first and last samples were taken together with all intervening batches.

For the individual test result requirements given in col 3 of Table 11 or in **15.2** (b), only the particular batch from which the sample was taken shall be at risk.

Where the mean rate of sampling is not specified the maximum quantity of concrete that four consecutive test results represent shall be limited to 60 m^3.

15.4 If the concrete is deemed not to comply pursuant to **15.1** or **15.2** as the case may be, the structural adequacy of the parts affected shall be investigated (*see* **16**) and any consequential action as needed shall be taken.

15.5 Concrete of each grade shall be assessed separately.

15.6 Concrete is liable to be rejected if it is porous or honey-combed, its placing has been interrupted without providing a proper construction joint, the reinforcement has been displaced beyond the tolerances specified, or construction tolerances have not been met. However, the hardened concrete may be accepted after carrying out suitable remedial measures to the satisfaction of the engineer-in-charge.

16 INSPECTION AND TESTING OF STRUCTURES

16.1 Inspection

To ensure that the construction complies with the design an inspection procedure should be set up covering materials, records, workmanship and construction.

16.1.1 Tests should be made on reinforcement and the constituent materials of concrete in accordance with the relevant standards. Where applicable, use should be made of suitable quality assurance schemes.

Table 11 Characteristic Compressive Strength Compliance Requirement
(*Clauses* 15.1 *and* 15.3)

Specified Grade (1)	Mean of the Group of 4 Non-Overlapping Consecutive Test Results in N/mm^2 (2)	Individual Test Results in N/mm^2 (3)
M 15	$\geq f_{ck}$ + 0.825 × established standard deviation (rounded off to nearest 0.5 N/mm^2) or f_{ck} + 3 N/mm^2, whichever is greater	$\geq f_{ck}$ – 3 N/mm^2
M 20 or above	$\geq f_{ck}$ + 0.825 × established standard deviation (rounded off to nearest 0.5 N/mm^2) or f_{ck} + 4 N/mm^2, whichever is greater	$\geq f_{ck}$ – 4 N/mm^2

NOTE — In the absence of established value of standard deviation, the values given in Table 8 may be assumed, and attempt should be made to obtain results of 30 samples as early as possible to establish the value of standard deviation.

16.1.2 Care should be taken to see that:

a) design and detail are capable of being executed to a suitable standard, with due allowance for dimensional tolerances;
b) there are clear instructions on inspection standards;
c) there are clear instructions on permissible deviations;
d) elements critical to workmanship, structural performance, durability and appearance are identified; and
e) there is a system to verify that the quality is satisfactory in individual parts of the structure, especially the critical ones.

16.2 Immediately after stripping the formwork, all concrete shall be carefully inspected and any defective work or small defects either removed or made good before concrete has thoroughly hardened.

16.3 Testing

In case of doubt regarding the grade of concrete used, either due to poor workmanship or based on results of cube strength tests, compressive strength tests of concrete on the basis of **16.4** and/or load test (*see* **16.6**) may be carried out.

16.4 Core Test

16.4.1 The points from which cores are to be taken and the number of cores required shall be at the discretion of the engineer-in-charge and shall be representative of the whole of concrete concerned. In no case, however, shall fewer than three cores be tested.

16.4.2 Cores shall be prepared and tested as described in accepted standard [6-5A(9)].

16.4.3 Concrete in the member represented by a core test shall be considered acceptable if the average equivalent cube strength of the cores is equal to at least 85 percent of the cube strength of the grade of concrete specified for the corresponding age and no individual core has a strength less than 75 percent.

16.5 In case the core test results do not satisfy the requirements of **16.4.3** or where such tests have not been done, load test (**16.6**) may be restored to.

16.6 Load Tests for Flexural Member

16.6.1 Load tests should be carried out as soon as possible after expiry of 28 days from the time of placing of concrete.

16.6.2 The structure should be subjected to a load equal to full dead load of the structure plus 1.25 times the imposed load for a period of 24 h and then the imposed load shall be removed.

NOTE — Dead load includes self weight of the structural members plus weight of finishes and walls or partitions, if any, as considered in the design.

16.6.3 The deflection due to imposed load only shall be recorded. If within 24 h of removal of the imposed load, the structure does not recover at least 75 percent of the deflection under superimposed load, the test may be repeated after a lapse of 72 h. If the recovery is less than 80 percent, the structure shall be deemed to be unacceptable.

16.6.3.1 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, the overall depth of the section in mm, it is not necessary for the recovery to be measured and the recovery provisions of **16.6.3** shall not apply.

16.7 Members Other than Flexural Members

Members other than flexural members should be preferably investigated by analysis.

16.8 Non-destructive Tests

Non-destructive tests are used to obtain estimation of the properties of concrete in the structure. The methods adopted include ultrasonic pulse velocity (*see* accepted standard [6-5A(32)]), probe penetration, pullout and maturity. Non-destructive tests provide alternatives to core tests for estimating the strength of concrete in a structure, or can supplement the data obtained from a limited number of cores. These methods are based on measuring a concrete property that bears some relationship to strength. The accuracy of these methods, in part, is determined by the degree of correlation between strength and the physical quality measured by the non-destructive tests.

Any of these methods may be adopted, in which case the acceptance criteria shall be agreed upon prior to testing.

SECTION 5A (c) GENERAL DESIGN CONSIDERATION

17 BASES FOR DESIGN

17.1 Aim of Design

The aim of design is the achievement of an acceptable probability that structures being designed will perform satisfactorily during their intended life. With an appropriate degree of safety, they should sustain all the loads and deformations of normal construction and use and have adequate durability and adequate resistance to the effects of misuse and fire.

17.2 Methods of Design

17.2.1 Structure and structural elements shall normally be designed by Limit State Method. Account should be taken of accepted theories, experiment and experience and the need to design for durability. Calculations alone do not produce safe, serviceable and durable structures. Suitable materials, quality control, adequate detailing and good supervision are equally important.

17.2.2 Where the Limit State Method can not be conveniently adopted, Working Stress Method may be used (*see* Annex B).

17.2.3 *Design Based on Experimental Basis*

Designs based on experimental investigations on models or full size structure or element may be accepted if they satisfy the primary requirements of **17.1** and subject to experimental details and the analysis connected therewith being approved by the engineer-in-charge.

17.2.3.1 Where the design is based on experimental investigation on full size structure or element, load tests shall be carried out to ensure the following:

a) The structure shall satisfy the requirements for deflection (*see* **22.2**) and cracking (*see* **34.3.2**) when subjected to a load for 24 h equal to the characteristic load multiplied by 1.33 γ_f where γ_f shall be taken from Table 18, for the limit state of serviceability. If within 24 h of the removal of the load, the structure does not show a recovery of at least 75 percent of the maximum deflection shown during the 24 h under the load, the test loading should be repeated after a lapse of 72 h. The recovery after the second test should be at least 75 percent of the maximum deflection shown during the second test.

   NOTE — 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.

b) The structure shall have adequate strength to sustain for 24 h, a total load equal to the characteristic load multiplied by 1.33 γ_f where γ_f shall be taken from Table 18 for the limit state of collapse.

17.3 Durability, Workmanship and Materials

It is assumed that the quality of concrete, steel and other materials and of the workmanship, as verified by inspections, is adequate for safety, serviceability and durability.

17.4 Design Process

Design, including design for durability, construction and use in service should be considered as a whole. The realization of design objectives requires compliance with clearly defined standards for materials, production, workmanship and also maintenance and use of structure in service.

18 LOADS AND FORCES

18.1 General

In structural design, account shall be taken of the dead, imposed and wind loads and forces such as those caused by earthquake, and effects due to shrinkage, creep, temperature, etc, where applicable.

18.2 Dead Loads

Dead loads shall be calculated on the basis of unit weights which shall be established taking into consideration the materials specified for construction.

18.2.1 Alternatively, the dead loads may be calculated on the basis of unit weights of materials given in good practice [6-5A(33)]. Unless more accurate calculations are warranted, the unit weights of plain concrete and reinforced concrete made with sand and gravel or crushed natural stone aggregate may be taken as 24 kN/m^3 and 25 kN/m^3 respectively.

18.3 Imposed Loads, Wind Loads and Snow Loads

Imposed loads, wind loads and snow loads shall be assumed in accordance with good practice [6-5A(33)].

18.4 Earthquake Forces

The earthquake forces shall be calculated in accordance with accepted standard [6-5A(34)].

18.5 Shrinkage, Creep and Temperature Effects

If the effects of shrinkage, creep and temperature are liable to affect materially the safety and serviceability of the structure, these shall be taken into account in the calculations (*see* **5.2.4**, **5.2.5** and **5.2.6**) and good practice [6-5A(33)].

18.5.1 In ordinary buildings, such as low rise dwellings whose lateral dimension do not exceed 45 m, the effects due to temperature fluctuations and shrinkage and creep can be ignored in design calculations.

18.6 Other Forces and Effects

In addition, account shall be taken of the following forces and effects if they are liable to affect materially the safety and serviceability of the structure:

a) Foundation movement (*see* good practice [6-5A(35)]),
b) Elastic axial shortening,

c) Soil and fluid pressures (*see* good practice [6-5A(33)]),
d) Vibration,
e) Fatigue,
f) Impact (*see* good practice [6-5A(33)]),
g) Erection loads (*see* good practice [6-5A(33)]), and
h) Stress concentration effect due to point load and the like.

18.7 Combination of Loads

The combination of loads shall be as given in good practice [6-5A(33)].

18.8 Dead Load Counteracting Other Loads and Forces

When dead load counteracts the effects due to other loads and forces in structural member or joint, special care shall be exercised by the designer to ensure adequate safety for possible stress reversal.

18.9 Design Load

Design load is the load to be taken for use in the appropriate method of design; it is the characteristic load in case of working stress method and characteristic load with appropriate partial safety factors for limit state design.

19 STABILITY OF THE STRUCTURE

19.1 Overturning

The stability of a structure as a whole against overturning shall be ensured so that the restoring moment shall be not 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 shall be considered. Restoring moment due to imposed loads shall be ignored.

19.1.1 The anchorages or counterweights provided for overhanging members (during construction and service) should be such that static equilibrium should remain, even when overturning moment is doubled.

19.2 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 only 0.9 times the characteristic dead load shall be taken into account.

19.3 Probable Variation in Dead Load

To ensure stability at all times, account shall be taken of probable variations in dead load during construction, repair or other temporary measures. Wind and seismic loading shall be treated as imposed loading.

19.4 Moment Connection

In designing the framework of a building provisions shall be made by adequate moment connections or by a system of bracings to effectively transmit all the horizontal forces to the foundations.

19.5 Lateral Sway

Under transient wind load the lateral sway at the top should not exceed $H/500$, where H is the total height of the building. For seismic loading, reference should be made to good practice [6-5A(34)].

20 FIRE RESISTANCE

20.1 A structure or structural element required to have fire resistance should be designed to possess an appropriate degree of resistance to flame penetration; heat transmission and failure. The fire resistance of a structural element is expressed in terms of time in hours in accordance with good practice [6-5A(36)]. Fire resistance of concrete elements depends upon details of member size, cover to steel reinforcement detailing and type of aggregate (normal weight or light weight) used in concrete. General requirements for fire protection are given in good practice [6-5A(37)]

20.2 Minimum requirements of concrete cover and member dimensions for normal weight aggregate concrete members so as to have the required fire resistance shall be in accordance with **25.4.3** and Fig. 1 respectively.

20.3 The reinforcement detailing should reflect the changing pattern of the structural action and ensure that both individual elements and the structure as a whole contain adequate support, ties, bonds and anchorages for the required fire resistance.

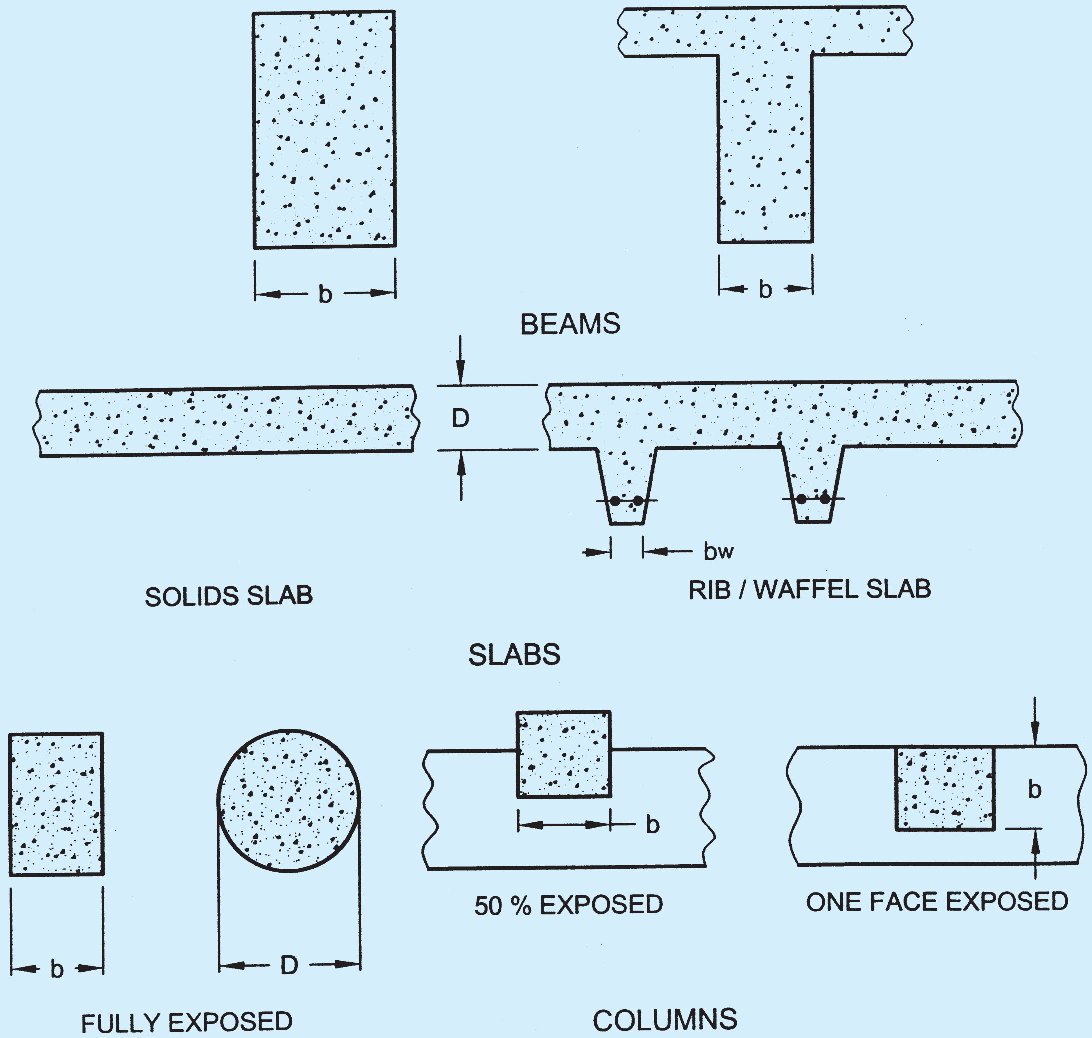
20.3.1 Additional measures such as application of fire resistant finishes, provision of fire resistant false ceilings and sacrificial steel in tensile zone, should be adopted in case the nominal cover required exceeds 40 mm for beams and 35 mm for slabs, to give protection against spalling.

20.4 Specialist literature may be referred to for determining fire resistance of the structures which have not been covered in Fig. 1 or Table 16A.

21 ANALYSIS

21.1 General

All structures may be analyzed by the linear elastic theory to calculate internal actions produced by design



Fire Resistance h	*Maximum Beam Width* *b* mm	*Rib Width of Slabs* b_w mm	*Minimum Thickness of Floors* *D* mm	*Column Dimension (b or D)* Fully Exposed mm	50% Exposed mm	One Face Exposed mm	*Minimum Wall Thickness* $p < 0.4\%$ mm	$0.4\% \leq p \leq 1\%$ mm	$p > 1\%$ mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
0.5	200	125	75	150	125	100	150	100	100
1	200	125	95	200	160	120	150	120	100
1.5	200	125	110	250	200	140	175	140	100
2	200	125	125	300	200	160	—	160	100
3	240	150	150	400	300	200	—	200	150
4	280	175	170	450	350	240	—	240	180

NOTES

1 These minimum dimensions relate specifically to the covers given in Table 16A.

2 *p* is the percentage of steel reinforcement.

FIG. 1 MINIMUM DIMENSIONS OF REINFORCED CONCRETE MEMBERS FOR FIRE RESISTANCE loads. In lieu of rigorous elastic analysis, a simplified analysis as given in **21.4** for frames and as given in **21.5** for continuous beams may be adopted.

21.2 Effective Span

Unless otherwise specified, the effective span of a member shall be as follows:

a) *Simply Supported Beam or Slab* — The effective span of a member that is not built integrally with it supports shall be taken as clear span plus the effective depth of slab or beam or centre to centre of supports, whichever is less.

b) *Continuous Beam or Slab* — In the case of continuous beam or slab, if the width of the support is less than 1/12 of the clear span, the effective span shall be as given in **21.2** (a). If the supports are wider than 1/12 of the clear span or 600 mm whichever is less, the effective span shall be taken as under:

   1) For end span with one end fixed and the other continuous or for intermediate spans, the effective span shall be the clear span between supports;

   2) For end span with one end free and the other continuous, the effective span shall be equal to the clear span plus half the effective depth of the beam or slab or the clear span plus half the width of the discontinuous support, whichever is less;

   3) In the case of spans with roller or rocket bearings, the effective span shall always be the distance between the centres of bearings.

c) *Cantilever* — The effective length of a cantilever shall be taken as its length to the face of the support plus half the effective depth except where it forms the end of a continuous beam where the length to the centre of support shall be taken.

d) *Frames* — In the analysis of a continuous frame, centre-to-centre distance shall be used.

21.3 Stiffness

21.3.1 *Relative Stiffness*

The relative stiffness of the members may be based on the moment of inertia of the section determined on the basis of any one of the following definitions:

a) *Gross section* — The cross-section of the member ignoring reinforcement;

b) *Transformed section* — The concrete cross-section plus the area of reinforcement transformed on the basis of modular ratio (*see* **B-1.3**); or

c) *Cracked section* — The area of concrete in compression plus the area of reinforcement transformed on the basis of modular ratio.

The assumptions made shall be consistent for all the members of the structure throughout any analysis.

21.3.2 For deflection calculations, appropriate values of moment of inertia as specified in Annex C should be used.

21.4 Structural Frames

The simplifying assumptions as given in **21.4.1** to **21.4.3** may be used in the analysis of frames.

21.4.1 *Arrangement of Imposed Load*

a) Consideration may be limited to combinations of:

   1) Design dead load on all spans with full design imposed load on two adjacent spans; and

   2) Design dead load on all spans with full design imposed load on alternate spans.

b) When design imposed load does not exceed three-fourths of the design dead load, the load arrangement may be design dead load and design imposed load on all the spans.

   NOTE — For beams and slabs continuous over support **21.4.1** (a) may be assumed.

21.4.2 *Substitute Frame*

For determining the moments and shears at any floor or roof level due to gravity loads, the beams at that level together with columns above and below with their far ends fixed may be considered to constitute the frame.

21.4.2.1 Where side sway consideration becomes critical due to unsymmetry in geometry or loading, rigorous analysis may be required.

21.4.3 For lateral loads, simplified methods may be used to obtain the moments and shears for structures that are symmetrical. For unsymmetrical or very tall structures, more rigorous methods should be used.

21.5 Moment and Shear Coefficients for Continuous Beams

21.5.1 Unless more exact estimates are made, for beams of uniform cross-section which support substantially uniformly distributed loads over three or more spans which do not differ by more than 15 percent of the longest, the bending moments and shear forces used in design may be obtained using the coefficients given in Table 12 and Table 13 respectively.

Table 12 Bending Moment Coefficients
(*Clause* 21.5.1)

Type of Load	Span Moments		Support Moments	
	Near Middle of End Span	At Middle of Interior Span	At Support Next to the End Support	At Other Interior Supports
(1)	(2)	(3)	(4)	(5)
Dead load and imposed load (fixed)	$+\frac{1}{12}$	$+\frac{1}{16}$	$-\frac{1}{10}$	$-\frac{1}{12}$
Imposed load (not fixed)	$+\frac{1}{10}$	$+\frac{1}{12}$	$-\frac{1}{9}$	$-\frac{1}{9}$

NOTE — For obtaining the bending moment, the coefficient shall be multiplied by the total design load and effective span.

Table 13 Shear for Coefficients
(*Clauses* 21.5.1 *and* 21.5.2)

Type of Load	At End Support	At Support Next to the End Support		At All Other Interior Supports
		Outer Side	Inner Side	
(1)	(2)	(3)	(4)	(5)
Dead load and imposed load (fixed)	0.4	0.6	0.55	0.5
Imposed load (not fixed)	0.45	0.6	0.6	0.6

NOTE — For obtaining the shear force, the coefficient shall be multiplied by the total design load.

For moments at supports where two unequal spans meet or in case where the spans are not equally loaded, the average of the two values for the negative moment at the support may be taken for design.

Where coefficients given in Table 12 are used for calculation of bending moments, redistribution referred to in **21.7** shall not be permitted.

21.5.2 *Beams and Slabs Over Free End Supports*

Where a member is built into a masonry wall which develops only partial restraint, the member shall be designed to resist a negative moment at the face of the support of $Wl/24$ where W is the total design load and l is the effective span, or such other restraining moment as may be shown to be applicable. For such a condition shear coefficient given in Table 13 at the end support may be increased by 0.05.

21.6 Critical Sections for Moment and Shear

21.6.1 For monolithic construction, the moments computed at the face of the supports shall be used in the design of the members at those section. For non-monolithic construction the design of the member shall be done keeping in view **21.2**.

21.6.2 *Critical Section for Shear*

The shears computed at the face of the support shall be used in the design of the member at that section except as in **21.6.2.1**.

21.6.2.1 When the reaction in the direction of the applied shear introduces compression into the end region of the member, sections located at a distance less than d from the face of the support may be designed for the same shear as that computed at distance d (*see* Fig. 2).

NOTE — The above clauses are applicable for beams generally carrying uniformly distributed load or where the principal load is located farther than $2d$ from the face of the support.

21.7 Redistribution of Moments

Redistribution of moments may be done in accordance with **36.1.1** for Limit State Method and in accordance with **B-1.2** for Working Stress Method. However, where simplified analysis using coefficients is adopted, redistribution of moments shall not be done.

22 BEAMS

22.0 Effective Depth

Effective depth of a beam is the distance between the centroid of the area of tension reinforcement and the maximum compression fibre, excluding the thickness of finishing material not placed monolithically with the member and the thickness of any concrete provided to allow for wear. This will not apply to deep beams.

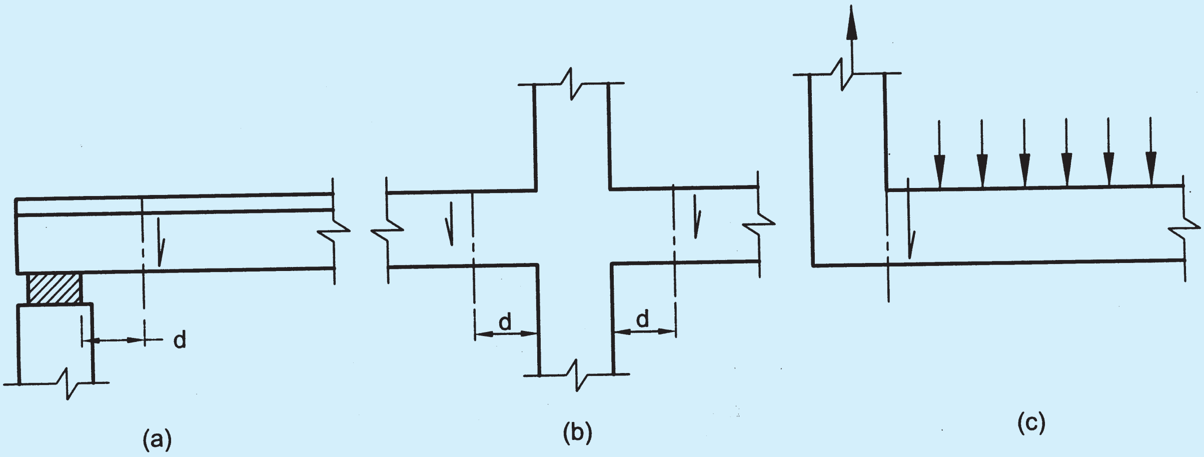


FIG. 2 TYPICAL SPORT CONDITIONS FOR LOCATING FACTORED SHEAR FORCE

22.1 T-Beams and L-Beams

22.1.1 *General*

A slab which is assumed to act as a compression flange of a T-beam or L-beam shall satisfy the following:

a) The slab shall be cast integrally with the web, or the web and the slab shall be effectively bonded together in any other manner; and

b) If the main reinforcement of the slab is parallel to the beam, transverse reinforcement shall be provided as in Fig. 3; such reinforcement shall not be less than 60 percent of the main reinforcement at mid span of the slab.

22.1.2 *Effective Width of Flange*

In the absence of more accurate determination, the effective width of flange may be taken as the following but in no case greater than the breadth of the web plus half the sum of the clear distances to the adjacent beams on either side.

a) For T-beams, $b_f = \frac{l_0}{6} + b_w + 6\,D_f$

b) For L-beams, $b_f = \frac{l_0}{12} + b_w + 3\,D_f$

c) For isolated beams, the effective flange width shall be obtained as below but in no case greater than the actual width:

$$\text{T-beams, } b_f = \frac{l_0}{\left(\frac{l_0}{b}\right) + 4} + b_w$$

$$\text{L-beams, } b_f = \frac{0.5\,l_0}{\left(\frac{l_0}{b}\right) + 4} + b_w$$

where

b_f = Effective width of flange,

l_0 = Distance between points of zero moments in the beam,

b_w = Breadth of the web,

D_f = Thickness of flange, and

b = Actual width of the flange.

NOTE — For continuous beams and frames, ; 0 may be assumed as 0.7 times the effective span.

22.2 Control of Deflection

The deflection of a structure or part thereof shall not adversely affect the appearance or efficiency of the structure or finishes or partitions. The deflection shall generally be limited to the following:

a) The final deflection due to all loads including the effects of temperature, creep and shrinkage and measured from the as-cast level of the supports of floors, roofs and all other horizontal members, should not normally exceed span/250.

b) The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.

22.2.1 The vertical deflection limits may generally be assumed to be satisfied provided that the span to depth ratios are not greater than the values obtained as below:

a) Basic values of span to effective depth ratios for spans up to 10 m:

Cantilever	7
Simply supported	20
Continuous	26

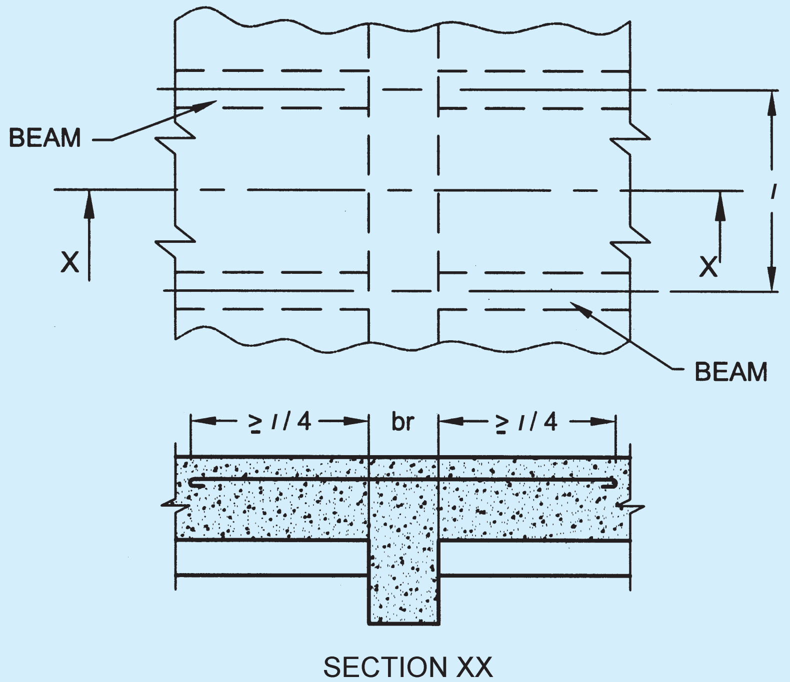


FIG. 3 TRANSVERSE REINFORCEMENT IN FLANGE OF T-BEAM WHEN MAIN REINFORCEMENT OF SLAB IS PARALLEL TO THE BEAM

b) For spans above 10 m, the values in (a) may be multiplied by 10/span in metres, except for cantilever in which case deflection calculations should be made.

c) Depending on the area and the stress of steel for tension reinforcement, the values in (a) or (b) shall be modified by multiplying with the modification factor obtained as per Fig. 4.

d) Depending on the area of compression reinforcement, the value of span to depth ratio be further modified by multiplying with the modification factor obtained as per Fig. 5.

e) For flanged beams, the values of (a) or (b) be modified as per Fig. 6 and the reinforcement percentage for use in Fig. 4 and 5 should be based on area of section equal to $b_f d$.

NOTE — When deflections are required to be calculated, the method given in Annex C may be used.

22.3 Slenderness Limits for Beams to Ensure Lateral Stability

A simply supported or continuous beam shall be so proportioned that the clear distance between the lateral restraints does not exceed 60 b or $\frac{250\,b^2}{d}$ whichever is less, where d is the effective depth of the beam and b the breadth of the compression face midway between the lateral restraints.

For a cantilever, the clear distance from the free end of the cantilever to the lateral restraint shall not exceed 25 b or $\frac{100\,b^2}{d}$ whichever is less.

23 SOLID SLABS

23.1 General

The provisions of **22.2** for beams apply to slabs also.

NOTES

1 For slabs spanning in two directions, the shorter of the two spans should be used for calculating the span to effective depth ratios.

2 For two-way slabs of shorter spans (up to 3.5 m) with mild steel reinforcement, the span to overall depth ratios given below may generally be assumed to satisfy vertical deflection limits for loading class up to 3 kN/m^2.

Simply supported slabs	35
Continuous slabs	40

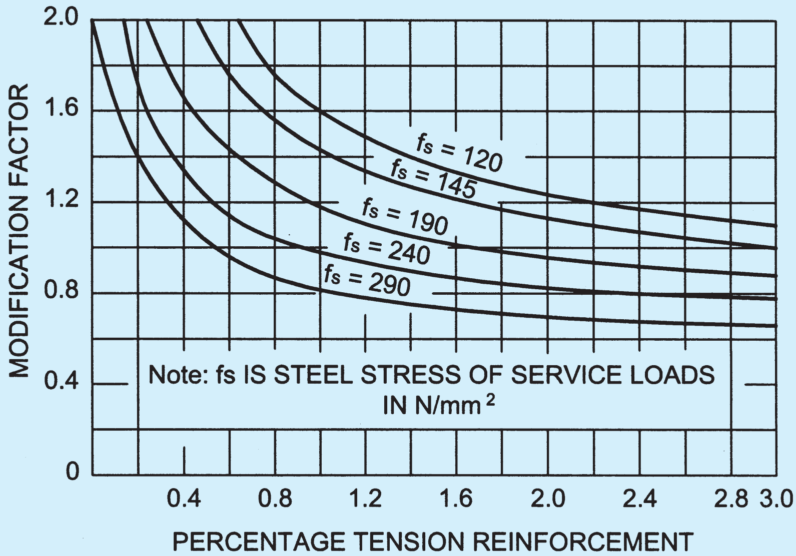
For high strength deformed bars of grade Fe 415, the values given above should be multiplied by 0.8.

23.2 Slabs Continuous Over Supports

Slabs spanning in one direction and continuous over supports shall be designed according to the provisions applicable to continuous beams.

23.3 Slabs Monolithic with Supports

Bending moments in slabs (except flat slabs)



$$f_s = 0.58\, f_y \frac{\text{Area of cross - section of steel required}}{\text{Area of cross - section of steel provided}}$$

FIG. 4 MODIFICATION FACTOR FOR TENSION REINFORCEMENT

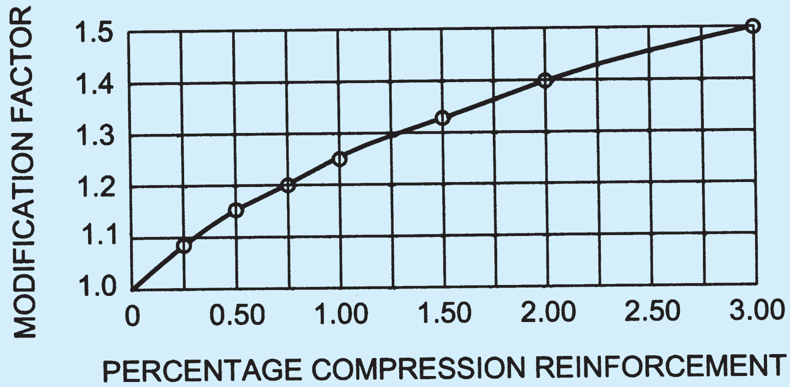


FIG. 5 MODIFICATION FACTOR FOR COMPRESSION REINFORCEMENT

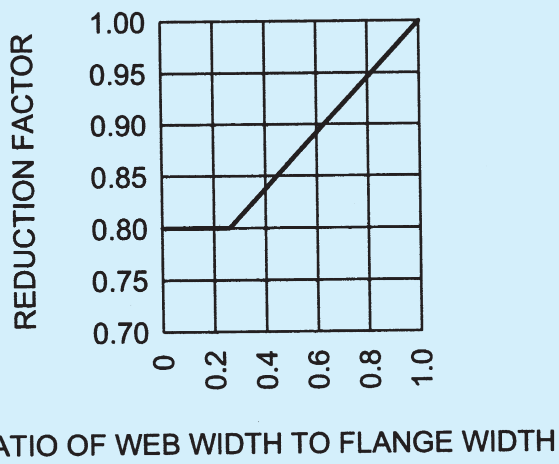


FIG. 6 REDUCTION FACTORS FOR RATIOS OF SPAN TO EFFECTIVE DEPTH FOR FLANGED BEAMS

constructed monolithically with the supports shall be calculated by taking such slabs either as continuous over supports and capable of free rotation, or as members of a continuous framework with the supports, taking into account the stiffness of such supports. If such supports are formed due to beams which justify fixity at the support of slabs, then the effects on the supporting beam, such as, the bending of the web in the transverse direction of the beam and the torsion in the longitudinal direction of the beam, wherever applicable, shall also be considered in the design of the beam.

23.3.1 For the purpose of calculation of moments in slabs in a monolithic structure, it will generally be sufficiently accurate to assume that members connected to the ends of such slabs are fixed in position and direction at the ends remote from their connections with the slabs.

23.3.2 *Slabs Carrying Concentrated Load*

23.3.2.1 If a solid slab supported on two opposite edges, carries concentrated loads the maximum bending moment caused by the concentrated loads shall be assumed to be resisted by an effective width of slab (measured parallel to the supporting edges) as follows:

a) For a single concentrated load, the effective width shall be calculated in accordance with the following equation provided that it shall not exceed the actual width of the slab:

$$b_{ef} = kx\left(1 - \frac{x}{l_{ef}}\right) + a$$

where

b_{ef} = Effective width of slab,

k = Constant having the values given in Table 14 depending upon the ratio of the width of the slab (l') to the effective span l_{ef},

x = Distance of the centroid of the concentrated load from nearer support,

l_{ef} = Effective span, and

a = Width of the contact area of the concentrated load from nearer support measured parallel to the supported edge.

And provided further that in case of a load near the unsupported edge of a slab, the effective width shall not exceed the above value nor half the above value plus the distance of the load from the unsupported edge.

Table 14 Values of *k* for Simply Supported and Continuous Slabs

(*Clause* 24.3.2.1)

l/l_{ef} (1)	*k* for Simply Supported Slabs (2)	*k* for Continuous Slabs (3)
0.1	0.4	0.4
0.2	0.8	0.8
0.3	1.16	1.16
0.4	1.48	1.44
0.5	1.72	1.68
0.6	1.96	1.84
0.7	2.12	1.96
0.8	2.24	2.08
0.9	2.36	2.16
1.0 and above	2.48	2.24

b) For two or more concentrated loads placed in a line in the direction of the span, the bending moment per metre width of slab shall be calculated separately for each load according to its appropriate effective width of slab calculated as in (a) above and added together for design calculations.

c) For two or more loads not in a line in the direction of the span, if the effective width of slab for one load does not overlap the effective width of slab for another load, both calculated as in (a) above, then the slab for each load can be designed separately. If the effective width of slab for one load overlaps the effective width of slab for an adjacent load, the overlapping portion of the slab shall be designed for the combined effect of the two loads.

d) For cantilever solid slabs, the effective width shall be calculated in accordance with the following equation:

$$b_{ef} = 1.2a_1 + a$$

where

b_{ef} = Effective width,

a_1 = Distance of the concentrated load from the face of the cantilever support, and

a = Width of contact area of the concentrated load measured parallel to the supporting edge.

Provided that the effective width of the cantilever slab shall not exceed one-third the length of the cantilever slab measured parallel to the fixed edge.

And provided further that when the concentrated load is placed near the extreme ends of the length of cantilever slab in the direction parallel to the fixed edge, the effective width shall not exceed the above value, nor shall it exceed half the above value plus the distance of the concentrated load from the extreme end measured in the direction parallel to the fixed edge.

23.3.2.2 For slabs other than solid slabs, the effective width shall depend on the ratio of the transverse and longitudinal flexural rigidities of the slab. Where this ratio is one, that is, where the transverse and longitudinal flexural rigidities are approximately equal, the value of effective width as found for solid slabs may be used. But as the ratio decreases, proportionately smaller value shall be taken.

23.3.2.3 Any other recognized method of analysis for cases of slabs covered by **23.3.2.1** and **23.3.2.2** and for all other cases of slabs may be used with the approval of the engineer-in-charge.

23.3.2.4 The critical section for checking shear shall be as given in **33.2.4.1**.

23.4 Slabs Spanning in Two Directions at Right Angles

The slabs spanning in two directions at right angles and carrying uniformly distributed load may be designed by any acceptable theory or by using coefficients given in Annex D. For determining bending moments in slabs spanning in two directions at right angles and carrying concentrated load, any accepted method approved by the engineer-in-charge may be adopted.

NOTE — The most commonly used elastic methods are based on Pigeaud's or Wester guard's theory and the most commonly used limit state of collapse method is based on Johansen's yield-line theory.

23.4.1 *Restrained Slab with Unequal Conditions at Adjacent Panels*

In some cases the support moments calculated from Table 26 for adjacent panels may differ significantly. The following procedure may be adopted to adjust them:

a) Calculate the sum of moments at midspan and supports (neglecting signs).
b) Treat the values from Table 26 as fixed end moments.
c) According to the relative stiffness of adjacent spans, distribute the fixed end moments across the supports, giving new support moments.
d) Adjust midspan moment such that, when added to the support moments from (c) (neglecting signs), the total should be equal to that from (a).

If the resulting support moments are significantly greater than the value from Table 26, the tension steel over the supports will need to be extended further. The procedure should be as follows:

1) Take the span moment as parabolic between supports: its maximum value is as found from (d).
2) Determine the points of contraflexure of the new support moments [from (c)] with the span moment [from (1)].
3) Extend half the support tension steel at each end to at least an effective depth or 12 bar diameters beyond the nearest point of contraflexure.
4) Extend the full area of the support tension steel at each end to half the distance from (3).

23.5 Loads on Supporting Beams

The loads on beams supporting solid slabs spanning in two directions at right angles and supporting uniformly distributed loads, may be assumed to be in accordance with Fig. 7.

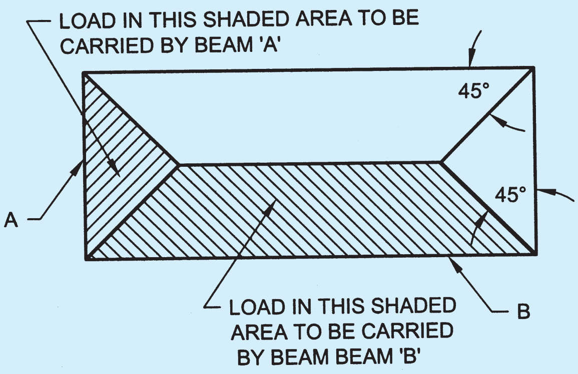


FIG. 7 LOAD CARRIED BY SUPPORTED BEAMS

24 COMPRESSION MEMBERS

24.1 Definitions

24.1.1 Column or strut is a compression member, the effective length of which exceeds three times the least lateral dimension.

24.1.2 *Short and Slender Compression Members*

A compression member may be considered as short when both the slenderness ratios $\frac{l_{ex}}{D}$ and $\frac{l_{ey}}{b}$ are less than 12:

where

l_{ex} = Effective length in respect of the major axis,
D = Depth in respect of the major axis,
l_{ey} = Effective length in respect of the minor axis, and
b = Width of the member.

It shall otherwise be considered as a slender compression member.

24.1.3 *Unsupported Length*

The unsupported length, l, of a compression member shall be taken as the clear distance between end restraints except that:

a) in flat slab construction, it shall be clear distance between the floor and the lower extremity of the capital, the drop panel or slab whichever is the least.

b) in beam and slab construction, it shall be the clear distance between the floor and the underside of the shallower beam framing into the columns in each direction at the next higher floor level.

c) in columns restrained laterally by struts, it shall be the clear distance between consecutive struts in each vertical plane, provided that to be an adequate support, two such struts shall meet the columns at approximately the same level and the angle between vertical planes through the struts shall not vary more than 30° from a right angle. Such struts shall be of adequate dimensions and shall have sufficient anchorage to restrain the member against lateral deflection.

d) in columns restrained laterally by struts or beams, with brackets used at the junction, it shall be the clear distance between the floor and the lower edge of the bracket, provided that the bracket width equals that of the beam strut and is at least half that of the column.

24.2 Effective Length of Compression Members

In the absence of more exact analysis, the effective length l_{ef} of columns may be obtained as described in Annex E.

24.3 Slenderness Limits for Columns

24.3.1 The unsupported length between end restraints shall not exceed 60 times the least lateral dimension of a column.

24.3.2 If, in any given plane, one end of a column is unrestrained, its unsupported length, l, shall not exceed $\dfrac{100\,b^2}{D}$.

where

b = Width of that cross-section, and

D = Depth of the cross-section measured in the plane under consideration.

24.4 Minimum Eccentricity

All columns shall be designed for minimum eccentricity, equal to the unsupported length of column/500 plus lateral dimensions/30, subject to a minimum of 20 mm. Where bi-axial bending is considered, it is sufficient to ensure that eccentricity exceeds the minimum about one axis at a time.

25 REQUIREMENTS GOVERNING REINFORCEMENT AND DETAILING

25.1 General

Reinforcing steel of same type and grade shall be used as main reinforcement in a structural member. However, simultaneous use of two different types or grades of steel for main and secondary reinforcement respectively is permissible.

25.1.1 Bars may be arranged singly, or in pairs in contact, or in groups of three or four bars bundled in contact. Bundled bars shall be enclosed within stirrups or ties. Bundled bars shall be tied together to ensure the bars remaining together. Bars larger than 32 mm diameter shall not be bundled, except in columns.

25.1.2 The recommendations for detailing for earthquake-resistant construction given in good practice [6-5A(38)] should be taken into consideration, where applicable (*see also* good practice [6-5A(38)]).

25.2 Development of Stress in Reinforcement

The calculated tension or compression in any bar at any section shall be developed on each side of the section by an appropriate development length or end anchorage or by a combination thereof.

25.2.1 *Development Length of Bars*

The development length L_d is given by

$$L_d = \frac{\phi \sigma_s}{4\tau_{bd}}$$

where

ϕ = Nominal diameter of the bar,

σ_s = Stress in bar at the section considered at design load, and

τ_{bd} = Design bond stress given in **25.2.1.1**.

NOTES

1 The development length includes anchorage values of hooks in tension reinforcement.

2 For bars of sections other than circular, the development length should be sufficient to develop the stress in the bar by bond.

25.2.1.1 Design bond stress in limit state method for plain bars in tension shall be as below:

Grade of concrete	M 20	M 25	M 30	M 35	M 40 and above
Design bond stress, τ_{bd}, N/mm^2	1.2	1.4	1.5	1.7	1.9

For deformed bars conforming to accepted standard [6-5A(40)] these values shall be increased by 60 percent.

For bars in compression, the values of bond stress for bars in tension shall be increased by 25 percent.

The values of bond stress in working stress design, are given in **B-2.1**.

25.2.1.2 *Bars bundled in contact*

The development length of each bar of bundled bars shall be that for the individual bar, increased by 10 percent for two bars in contact, 20 percent for three bars in contact and 33 percent for four bars in contact.

25.2.2 *Anchoring Reinforcing Bars*

25.2.2.1 *Anchoring bars in tension*

a) Deformed bars may be used without end anchorages provided development length requirement is satisfied. Hooks should normally be provide for plain bars in tension.
b) *Bends and hooks* — Bends and hooks shall conform to good practice [6-5A(26)]:
    1) *Bends* — 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) *Hooks* — The anchorage value of a standard U-type hook shall be equal to 16 times the diameter of the bar.

25.2.2.2 *Anchoring bars in compression*

The anchorage length of straight bar in compression shall be equal to the development length of bars in compression as specified in **25.2.1**. The projected length of hooks, bends and straight lengths beyond bends if provided for a bar in compression, shall only be considered for development length.

25.2.2.3 *Mechanical devices for anchorage*

Any mechanical or other device capable of developing the strength of the bar without damage to concrete may be used as anchorage with the approval of the engineer-in-charge.

25.2.2.4 *Anchoring shear reinforcement*

a) *Inclined bars* — The development length shall be as for bars in tension; this length shall be measured as under:
    1) In tension zone, from the end of the sloping or inclined portion of the bar, and
    2) In the compression zone, from the mild depth of the beam.
b) *Stirrups* — Notwithstanding any of the provisions of this standard, incase of secondary reinforcement, such as stirrups and transverse ties, complete development lengths and anchorage shall be deemed to have been provided when the bar is bent through an angle of at least 90° round a bar of at least its own diameter and is continued beyond the end of the curve for a length of at least eight diameters, or when the bar is bent through an angle of 135° and is continued beyond the end of the curve for a length of at least six bar diameters or when the bar is bent through an angle of 180° and is continued beyond the end of the curve for a length of at least four bar diameters.

25.2.2.5 *Bearing stresses at bends*

The bearing stress in concrete for bends and hooks described in good practice [6-5A(26)] need not be checked. The bearing stress inside a bend in any other bend shall be calculated as given below:

$$\text{Bearing stress} = \frac{F_{bt}}{r\phi}$$

where

F_{bt} = Tensile force due to design loads in a bar or group of bars,

r = Internal radius of the bend, and

ϕ = Size of the bar or, in bundle, the size of bar of equivalent area.

For limit state method of design, this stress shall not exceed $\dfrac{1.5\, f_{ck}}{1+2\phi/a}$ where f_{ck} is the characteristic cube strength of concrete and a, for a particular bar or group of bars in contact shall be taken as the centre to centre distance between bars or groups of bars perpendicular to the plane of the bend; for a bar or group of bars adjacent to the face of the member a shall be taken as the cover plus size of bar (ϕ). For working stress method of design, the bearing stress shall not exceed $\dfrac{f_{ck}}{1+2\phi/a}$.

25.2.2.6 If a change in direction of tension or compression reinforcement induces a resultant force acting outward tending to split the concrete, such force should be taken up by additional links or stirrups. Bent tension bar at a re-entrant angle should be avoided.

25.2.3 *Curtailment of Tension Reinforcement in Flexural Members*

25.2.3.1 For curtailment, reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member or 12 times the bar diameter, whichever is greater except at simple support or end of cantilever. In addition **25.2.3.2** to **25.2.3.5** shall also be satisfied.

NOTE — A point at which reinforcement is no longer required to resist flexure is where the resistance moment of the section, considering only the continuing bars, is equal to the design moment.

25.2.3.2 Flexural reinforcement shall not be terminated in a tension zone unless any one of the following conditions is satisfied:

a) The shear at the cut-off point does not exceed two-thirds that permitted, including the shear strength of web reinforcement provided.

b) Stirrup area in excess of that required for shear and torsion is provided along each terminated bar over a distance from the cut-off point equal to three-fourths the effective depth of the member. The excess stirrup area shall be not less than $0.4\ bs/f_y$, where b is the breadth of beam, s is the spacing and f_y is the characteristic strength of reinforcement in N/mm^2. The resulting spacing shall not exceed $d/8\ \beta_b$ where β_b is the ratio of the area of bars cut-off to the total area of bars at the section, and d is the effective depth.

c) For 36 mm and smaller bars, the continuing bars provide double the area required for flexure at the cut-off point and the shear does not exceed three-fourths that permitted.

25.2.3.3 *Positive moment reinforcement*

a) At least one-third the positive moment reinforcement in simple members and one-fourth the positive moment reinforcement in continuous members shall extend along the same face of the member into the support, to a length equal to $L_d/3$.

b) When a flexural member is part of the primary lateral load resisting system, the positive reinforcement required to be extended into the support as described in (a) shall be anchored to develop its design stress in tension at the face of the support.

c) At simple supports and at points of inflection, positive moment tension reinforcement shall be limited to a diameter such that L_d computed for f_d by **25.2.1** does not exceed

$$\frac{M_1}{V} + L_0$$

where

M_1 = Moment of resistance of the section assuming all reinforcement at the section to be stressed to f_d;

f_d = $0.87 f_y$ in the case of limit state design and the permissible stress σ_{st} in the case of working stress design;

V = Shear force at the section due to design loads;

L_0 = Sum of the anchorage beyond the centre of the support and the equivalent anchorage value of any hook or mechanical anchorage at simple support; and at a point of inflection, L_0 is limited to the effective depth of the members or 12ϕ, whichever is greater; and

ϕ = Diameter of bar.

The value of M_1/V in the above expression may be increased by 30 percent when the ends of the reinforcement are confined by a compressive reaction.

25.2.3.4 *Negative moment reinforcement*

At least one-third of the total reinforcement provided for negative moment at the support shall extend beyond the point of inflection for a distance not less than the effective depth of the member of $12\ \phi$ or one-sixteenth of the clear span whichever is greater.

25.2.3.5 *Curtailment of bundled bars*

Bars in a bundle shall terminate at different points spaced apart by not less than 40 times the bar diameter except for bundles stopping at a support.

25.2.4 *Special Members*

Adequate end anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as sloped, stepped, or tapered footings; brackets; deep beams; and members in which the tension reinforcement is not parallel to the compression face.

25.2.5 *Reinforcement Splicing*

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 should 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 spirals or closely-spaced stirrups around the length of the splice.

25.2.5.1 *Lap splices*

a) Lap splices shall not be used for bars larger than 36 mm; for larger diameters, bars may be welded (*see* **11.4**); in cases where welding is not practicable, lapping of bars larger than 36 mm may be permitted, in which case additional spirals should be provided around the lapped bars.

b) Lap splices shall be considered as staggered if the centre to centre distance of the splices is not less than 1.3 times the lap length calculated as described in (c).

c) Lap length including anchorage value of hooks for bars in flexural tension shall be L_d (*see* **25.2.1**) or 30 ϕ whichever is greater. The straight length of the lap shall not be less than 15 ϕ or 200 mm. The following provisions shall also apply:

   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.

   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.

      Where both conditions (1) and (2) apply, the lap length should be increased by a factor of 2.0.

      NOTE: Splices in tension members shall be enclosed in spirals made of bars not less than 6 mm diameter with pitch not more than 100 mm.

d) The lap length in compression shall be equal to the development length in compression, calculated as described in **25.2.1**, but not less than 24 ϕ.

e) When bars of two different diameters are to be spliced, the lap length shall be calculated on the basis of diameter of the smaller bar.

f) When splicing of welded wire fabric is to be carried out, lap splices of wires shall be made so that overlap measured between the extreme cross wires shall be not less than the spacing of cross wires plus 100 mm.

g) In case of bundled bars, lapped splices of bundled bars shall be made by splicing one bar at a time; such individual splices within a bundle shall be staggered.

25.2.5.2 *Strength of welds*

The following values may be used where the strength of the weld has been proved by tests to be at least as great as that of the parent bar.

a) *Splices in compression* — For welded splices and mechanical connection, 100 percent of the design strength of joined bars.

b) *Splices in tension*

   1) 80 percent of the design strength of welded bars (100 percent if welding is strictly supervised and if at any cross-section of the member not more than 20 percent of the tensile reinforcement is welded).

   2) 100 percent of design strength of mechanical connection.

25.2.5.3 *End-bearing splices*

End-bearing splices shall be used only for bars in compression. The ends of the bars shall be square cut and concentric bearing ensured by suitable devices.

25.3 Spacing of Reinforcement

25.3.1 For the purpose of this clause, the diameter of a round bar shall be its nominal diameter, and in the case of bars which are not round or in the case of deformed bars or crimped bars, the diameter shall be taken as the diameter of a circle giving an equivalent effective area. Where spacing limitations and minimum concrete cover (*see* **25.4**) are based on bar diameter, a group of bars bundled in contact shall be treated as a single bar of diameter derived from the total equivalent area.

25.3.2 *Minimum Distance Between Individual Bars*

The following shall apply for spacing of bars:

a) The horizontal distance between two parallel main reinforcing bars shall usually be not less than the greatest of the following:

   1) The diameter of the bar if the diameters are equal,

   2) The diameter of the larger bar if the diameters are unequal, and

   3) 5 mm more than the nominal maximum size of coarse aggregate.

      NOTE — This does not preclude the use of larger size of aggregates beyond the congested reinforcement in the same member; the size of aggregates may be reduced around congested reinforcement to comply with this provision.

b) Greater horizontal distance than the minimum specified in (a) should be provided wherever possible. However, when needle vibrators are used the horizontal distance between bars of a group may be reduced to two-thirds the nominal maximum size of the coarse aggregate, provided that sufficient space is left between groups of bars to enable the vibrator to be immersed.

c) Where there are two or more rows of bars, the bars shall be vertically in line and the minimum vertical distance between the bars shall be 15 mm, two-thirds the nominal maximum size of aggregate or the maximum size of bars, whichever is greater.

25.3.3 *Maximum Distance Between Bars in Tension*

Unless the calculation of crack widths shows that a greater spacing is acceptable, the following rules shall be applied to flexural members in normal internal or external conditions of exposure.

a) *Beams* — The horizontal distance between parallel reinforcement bars, or groups, near the tension face of a beam shall not be greater than the value given in Table 15 depending on the amount of re-distribution carried out in analysis and the characteristic strength of the reinforcement.

Table 15 Clear Distance Between Bars
(*Clause* 25.3.3)

f_y	Percentage Re-distribution to or from Section Considered				
	– 30	– 15	0	+15	+ 30
	Clear Distance Between Bars				
N/mm^2	mm	mm	mm	mm	mm
(1)	(2)	(3)	(4)	(5)	(6)
250	215	260	300	300	300
415	125	155	180	210	235
500	105	130	150	175	195

NOTE — The spacings given in table are not applicable to members subjected to particularly aggressive environments unless in the calculation of the moment of resistance f_y has been limited to 300 N/mm^2 in limit state design and σ_{st} limited to 165 N/mm^2 in working stress design.

b) *Slabs*

1) The horizontal distance between parallel main reinforcement bars shall not be more than three times the effective depth of solid slab or 300 mm whichever is smaller.

2) The horizontal distance between parallel reinforcement bars provided against shrinkage and temperature shall not be more than five times the effective depth of a solid slab or 450 mm whichever is smaller.

25.4 Nominal Cover to Reinforcement

25.4.1 *Nominal Cover*

Nominal cover is the design depth of concrete cover to all steel reinforcements, including links. It is the dimension used in design and indicated in the drawings. It shall be not less than the diameter of the bar.

25.4.2 *Nominal Cover to Meet Durability Requirement*

Minimum values for the nominal cover of normal-weight aggregate concrete which should be provided to all reinforcement, including links depending on the condition of exposure described in **7.2.2** shall be as given in Table 16.

25.4.2.1 However for a longitudinal reinforcing bar in a column nominal cover shall in any case not be less than 40 mm, or less than the diameter of such bar. In the case of columns of minimum dimension of 200 mm or under, whose reinforcing bars do not exceed 12 mm, a nominal cover of 25 mm may be used.

25.4.2.2 For footings minimum cover shall be 50 mm.

25.4.3 *Nominal Cover to Meet Specified Period of Fire Resistance*

Minimum values of nominal cover of normal-weight aggregate concrete to be provided to all reinforcement including links to meet specified period of fire resistance shall be given in Table 16 A.

Table 16 Nominal Cover to Meet Durability Requirements
(*Clause* 25.4.2)

Exposure	Nominal Concrete Cover in mm not Less Than
(1)	(2)
Mild	20
Moderate	30
Severe	45
Very Severe	50
Extreme	75

NOTES

1 For main reinforcement up to 12 mm diameter bar for mild exposure the nominal cover may be reduced by 5 mm.

2 Unless specified otherwise, actual concrete cover should not deviate from the required nominal cover by $\begin{smallmatrix}+10\\-\ 0\end{smallmatrix}$ mm.

3 For exposure condition 'severe' and 'very severe', reduction of 5 mm may be made, where concrete grade is M 35 and above.

Table 16A Nominal Cover to Meet Specified Period of Fire Resistance
(*Clauses* 20.4 *and* 25.4.3 and *Fig.* 1)

Fire Resistance	Nominal Cover						
	Beams		Slabs		Ribs		Columns
	Simply supported	Continuous	Simply supported	Continuous	Simply supported	Continuous	
h	mm	mm	mm	mm	mm	mm	mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0.5	20	20	20	20	20	20	40
1	20	20	20	20	20	20	40
1.5	20	20	25	20	35	20	40
2	40	30	35	25	45	35	40
3	60	40	45	35	55	45	40
4	70	50	55	45	65	55	40

NOTES

1 The nominal covers given relate specifically to the minimum member dimensions given in Fig. 1.

2 Cases that lie below the bold line require attention to the additional measures necessary to reduce the risks of spalling (*see* **20.3.1**).

25.5 Requirements of Reinforcement for Structural Members

25.5.1 *Beams*

25.5.1.1 *Tension reinforcement*

a) *Minimum reinforcement* — The minimum area of tension reinforcement shall be not less than that given by the following:

$$\frac{A_s}{bd} = \frac{0.85}{f_y}$$

where

A_s = Minimum area of tension reinforcement,

b = Breadth of beam of the breadth of the web of T-beam,

d = Effective depth, and

f_y = Characteristic strength of reinforcement in N/mm^2.

b) *Maximum reinforcement* — The maximum area of tension reinforcement shall not exceed 0.04 *bD*.

25.5.1.2 *Compression reinforcement*

The maximum area of compression reinforcement shall not exceed 0.04 *bD*. Compression reinforcement in beams shall be enclosed by stirrups for effective lateral restraint. The arrangement of stirrups shall be as specified in **25.5.3.2**.

25.5.1.3 *Side face reinforcement*

Where the depth of the web in a beam exceeds 750 mm, side face reinforcement shall be provided along the two faces. The total area of such reinforcement shall be not less than 0.1 percent of the web area and shall be distributed equally on two faces at a spacing not exceeding 300 mm or web thickness whichever is less.

25.5.1.4 *Transverse reinforcement in beams for shear and torsion*

The transverse reinforcement in beams shall be taken around the outer-most tension and compression bars. In T-beams and I-beams, such reinforcement shall pass around longitudinal bars located close to the outer face of the flange.

25.5.1.5 *Maximum spacing of shear reinforcement*

The maximum spacing of shear reinforcement measured along the axis of the member shall not exceed 0.75 *d* for vertical stirrups and *d* for inclined stirrups at 45°, where *d* is the effective depth of the section under consideration. In no case shall the spacing exceed 300 mm.

25.5.1.6 *Minimum shear reinforcement*

Minimum shear reinforcement in the form of stirrups shall be provided such that:

$$\frac{A_{sv}}{bs_v} \geq \frac{0.4}{0.87 f_y}$$

where

A_{sv} = Total cross-sectional area of stirrups legs effective in shear,

s_v = Stirrup spacing along the length of the member,

b = Breadth of the beam or breadth of the web of flanged beam, and

f_y = Characteristic strength of the stirrup reinforcement in N/mm^2 which shall not be taken greater than 415 N/mm^2.

Where the maximum shear stress calculated is less than half the permissible value and in members of minor structural importance such as lintels, this provision need not be complied with.

25.5.1.7 *Distribution of torsion reinforcement*

When a member is designed for torsion (*see* **40** or **B-6**) torsion reinforcement shall be provided as below:

a) The transverse reinforcement for torsion shall be rectangular closed stirrups placed perpendicular to the axis of the member. The spacing of the stirrups shall not exceed the least of x_1, $\frac{x_1 + y_1}{4}$ and 300 mm, where x_1 and y_1 are respectively the short and long dimensions of the stirrup.
b) Longitudinal reinforcement shall be placed as close as is practicable to the corners of the cross-section and in all cases, there shall be at least one longitudinal bar in each corner of the ties. When the cross-sectional dimension of the member exceeds 450 mm, additional longitudinal bars shall be provided to satisfy the requirements of minimum reinforcement and spacing given in **25.5.1.3**.

25.5.1.8 Reinforcement in flanges of T-beams and L-beams shall satisfy the requirements in **22.1.1**(b). Where flanges are in tension, a part of the main tension reinforcement shall be distributed over the effective flange width or a width equal to one-tenth of the span, whichever is smaller. If the effective flange width exceeds one-tenth of the span, nominal longitudinal reinforcement shall be provided in the outer portions of the flange.

25.5.2 *Slabs*

The rules given in **25.5.2.1** and **25.5.2.2** shall apply to slabs in addition to those given in the appropriate clauses.

25.5.2.1 *Minimum reinforcement*

The mild steel reinforcement in either direction in slabs shall not be less than 0.15 percent of the total cross-sectional area. However, this value can be reduced to 0.12 percent when high strength deformed bars or welded wire fabric are used.

25.5.2.2 *Maximum diameter*

The diameter of reinforcing bars shall not exceed one-eight of the total thickness of the slab.

25.5.3 *Columns*

25.5.3.1 *Longitudinal reinforcement*

a) The cross-sectional area of longitudinal reinforcement, shall be not less than 0.8 percent nor more than 6 percent of the gross cross-sectional area of the column.

   NOTE — The use of 6 percent reinforcement may involve practical difficulties in placing and compacting of concrete; hence lower percentage is recommended. Where bars from the columns below have to be lapped with those in the column under consideration, the percentage of steel shall usually not exceed 4 percent.
b) In any column that has a larger cross-sectional area than that required to support the load, the minimum percentage of steel shall be based upon the area of concrete required to resist the direct stress and not upon the actual area.
c) The minimum number of longitudinal bars provided in a column shall be four in rectangular columns and six in circular columns.
d) The bars shall not be less than 12 mm in diameter.
e) A reinforced concrete column having helical reinforcement shall have at least six bars of longitudinal reinforcement within the helical reinforcement.
f) In a helically reinforced column, the longitudinal bars shall be in contact with the helical reinforcement and equidistant around its inner circumference.
g) Spacing of longitudinal bars measured along the periphery of the column shall not exceed 300 mm.
h) In case of pedestals in which the longitudinal reinforcement is not taken in account in strength calculations, nominal longitudinal reinforcement not less than 0.15 percent of the cross-sectional area shall be provided.

   NOTE — Pedestal is a compression member, the effective length of which does not exceed three times the least lateral dimension.

25.5.3.2 *Transverse reinforcement*

a) *General* — A reinforced concrete compression member shall have transverse or helical reinforcement so disposed that every longitudinal bar nearest to the compression face has effective lateral support against buckling subject to provisions in (b). The effective lateral support is given by transverse reinforcement either in the form of circular rings capable of taking up circumferential tension or by polygonal links (lateral ties) with internal angles not exceeding 135°. The ends of the transverse reinforcement shall be properly anchored [*see* **25.2.2.4** (b)].

b) *Arrangement of transverse reinforcement*

1) If the longitudinal bars are not spaced more than 75 mm on either side, transverse reinforcement need only to go round corner and alternate bars for the purpose of providing effective lateral supports (*see* Fig. 8).

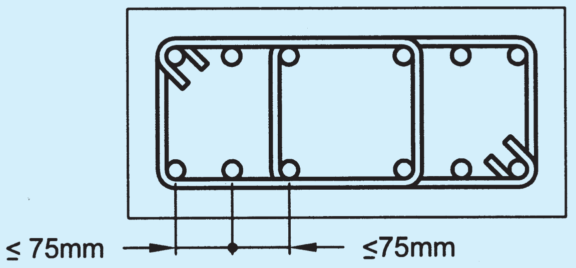


FIG. 8

2) If the longitudinal bars spaced at a distance of not exceeding 48 times the diameter of the tie are effectively tied in two directions, additional longitudinal bars in between these bars need to be tied in one direction by open ties (*see* Fig. 9).

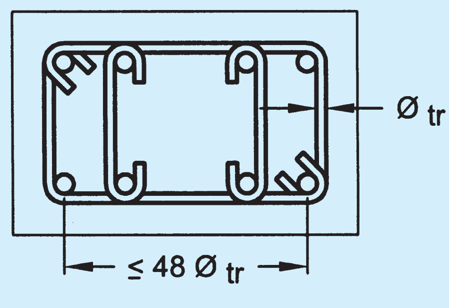


FIG. 9

3) Where the longitudinal reinforcing bars in a compression member are placed in more than one row, effective lateral support to the longitudinal bars in the inner rows may be assumed to have been provided if:

i) transverse reinforcement is provided for the outer-most row in accordance with **25.5.3.2**, and

ii) no bar of the inner row is closer to the nearest compression face than three times the diameter of the largest bar in the inner row (*see* Fig. 10).

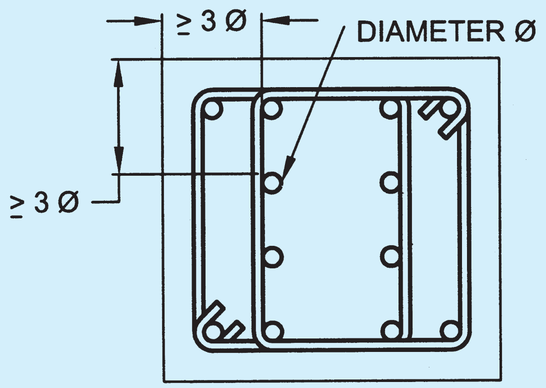


FIG. 10

4) Where the longitudinal bars in a compression member are grouped (not in contact) and each group adequately tied with transverse reinforcement in accordance with **25.5.3.2**, the transverse reinforcement for the compression member as a whole may be provided on the assumption that each group is a single longitudinal bar for purpose of determining the pitch and diameter of the transverse reinforcement in accordance with **25.5.3.2**. The diameter of such transverse reinforcement need not, however, exceed 20 mm (*see* Fig. 11).

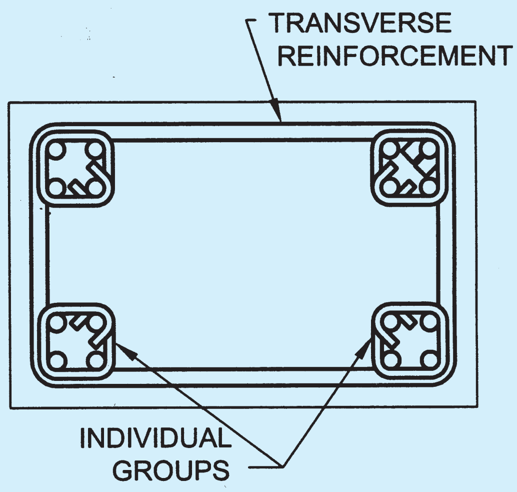


FIG. 11

c) *Pitch and diameter of lateral ties*

1) *Pitch* — The pitch of transverse reinforcement shall be not more than the least of the following distances:

i) The least lateral dimension of the compression members;

ii) Sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied; and

iii) 300 mm.

2) *Diameter* — The diameter of the polygonal links or lateral ties shall be not less than one-fourth of the diameter of the largest longitudinal bar, and in no case less than 6 mm.

d) *Helical reinforcement*

1) *Pitch* — Helical reinforcement shall be of regular formation with the turns of the helix spaced evenly and its ends shall be anchored properly by providing one and a half extra turns of the spiral bar. Where an increased load on the column on the strength of the helical reinforcement is allowed for, the pitch of helical turns shall be not more than 75 mm, nor more than one-sixth of the core diameter of the column, nor less than 25 mm, nor less than three times the diameter of the steel bar forming the helix. In other cases, the requirements of **25.5.3.2** shall be complied with.

2) The diameter of the helical reinforcement shall be in accordance with **25.5.3.2** (c) (2).

25.5.3.3 In columns where longitudinal bars are offset at a splice, the slope of the inclined portion of the bar with the axis of the column shall not exceed 1 in 6, and the portions of the bar above and below the offset shall be parallel to the axis of the column. Adequate horizontal support at the offset bends shall be treated as a matter of design, and shall be provided by metal ties, spirals, or parts of the floor construction. Metal ties or spirals so designed shall be placed near (not more than eight-bar diameters from) the point of bend. The horizontal thrust to be resisted shall be assumed as one and half times the horizontal components of the nominal stress in the inclined portion of the bar. Offset bars shall be bent before they are placed in the forms. Where column faces are offset 75 mm or more, splices of vertical bars adjacent to the offset face shall be made by separate dowels overlapped as specified in **25.2.5.1**.

26 EXPANSION JOINTS

26.1 Structures in which marked changes in plan dimensions take place abruptly shall be provided with expansion on joints at the section where such changes occur. Expansion joints shall be so provided that the necessary movement occurs with a minimum resistance at the joint. The structures adjacent to the joint should preferably be supported on separate columns or walls but not necessarily on separate foundations. Reinforcement shall not extend across an expansion joint and the break between the sections shall be complete.

26.2 The details as to the length of a structure where expansion joints have to be provided can be determined after taking into consideration various factors, such as temperature, exposure to weather, the time and season of the laying of the concrete, etc. Normally structures exceeding 45 m in length are designed with one or more expansion joints. However in view of the large number of factors involved in deciding the location, spacing and nature of expansion joints, the provision of expansion joint in reinforced cement concrete structures should be left to the discretion of the designer. Good practice [6-5A(41)] gives the design considerations, which need to be examined and provided for.

SECTION 5A (d) SPECIAL DESIGN REQUIREMENTS FOR STRUCTURAL MEMBERS AND SYSTEMS

27 CONCRETE CORBELS

27.1 General

A corbel is a short cantilever projection which supports a load bearing member and where:

a) the distance a_v between the line of the reaction to the supported load and the root of the corbel is less than d (the effective depth of the root of the corbel); and

b) the depth at the outer edge of the contact area of the supported load is not less than one-half of the depth at the root of the corbel.

The depth of the corbel at the face of the support is determined in accordance with **39.5.1**.

27.2 Design

27.2.1 *Simplifying Assumptions*

The concrete and reinforcement may be assumed to act as elements of a simple strut-and-tie system, with the following guidelines:

a) The magnitude of the resistance provided to horizontal force should be not less than one-half of the design vertical load on the corbel (*see also* **27.2.4**).
b) Compatibility of strains between the strut-and-tie at the corbel root should be ensured.

It should be noted that the horizontal link requirement described in **27.2.3** will ensure satisfactory serviceability performance.

27.2.2 *Reinforcement Anchorage*

At the front face of the corbel, the reinforcement should be anchored either by:

a) *welding to a transverse bar of equal strength* — In this case the bearing area of the load should stop short of the face of the support by a distance equal to the cover of the tie reinforcement, or
b) *bending back the bars to form a loop* — In this case the bearing area of the load should not project beyond the straight portion of the bars forming the main tension reinforcement.

27.2.3 *Shear Reinforcement*

Shear reinforcement should be provided in the form of horizontal links distributed in the upper two-third of the effective depth of root of the corbel; this reinforcement should be not less than one-half of the area of the main tension reinforcement and should be adequately anchored.

27.2.4 *Resistance to Applied Horizontal Force*

Additional reinforcement connected to the supported member should be provided to transmit this force in its entirety.

28 DEEP BEAMS

28.1 General

a) A beam shall be deemed to be a deep beam when the ratio of effective span to overall depth, $\frac{l}{D}$ is less than:
   1) 2.0 for a simply supported beam; and
   2) 2.5 for a continuous beam.
b) A deep beam complying with the requirements of **28.2** and **28.3** shall be deemed to satisfy the provisions for shear.

28.2 Lever Arm

The lever arm z for a deep beam shall be determined as below:

a) *For simply supported beams*:

$$z = 0.2\,(l + 2D) \qquad \text{when } 1 \leq \frac{l}{D} \leq 2$$

or

$$z = 0.6 \qquad \text{when } \frac{l}{D} < 1$$

b) *For continuous beams*:

$$z = 0.2\,(l + 1.5D) \qquad \text{when } 1 \leq \frac{l}{D} \leq 2.5$$

or

$$z = 0.5\,l \qquad \text{when } \frac{l}{D} < 1$$

where l is the effective span taken as centre to centre distance between supports or 1.15 times the clear span, whichever is smaller, and D is the overall depth.

28.3 Reinforcement

28.3.1 *Positive Reinforcement*

The tensile reinforcement required to resist positive bending moment in any span of a deep beam shall:

a) extend without curtailment between supports;
b) be embedded beyond the face of each support, so that at the face of the support it shall have a development length not less than 0.8 L_d; where L_d is the development length (*see* **25.2.1**), for the design stress in the reinforcement; and
c) be placed within a zone of depth equal to 0.25 D – 0.05 l adjacent to the tension face of the beam where D is the overall depth and l is the effective span.

28.3.2 *Negative Reinforcement*

a) *Termination of reinforcement* — For tensile reinforcement required to resist negative bending moment over a support of a deep beam:
   1) It shall be permissible to terminate not more than half of the reinforcement at a distance of 0.5 D from the face of the support where D is as defined in **29.2**; and
   2) The remainder shall extend over the full span.
b) *Distribution* — When ratio of clear span to overall depth is in the range 1.0 to 2.5, tensile reinforcement over a support of a deep beam shall be placed in two zones comprising:

1) a zone of depth 0.2 D, adjacent to the tension face, which shall contain a proportion of the tension steel given by

$$0.5\left(\frac{l}{D}-0.5\right)$$

where

l = Clear span, and

D = Overall depth.

2) A zone measuring 0.3 D on either side of the mid-depth of the beam, which shall contain the remainder of the tension steel, evenly distributed.

For span to depth ratios less than unity, the steel shall be evenly distributed over a depth of 0.8 D measured from the tension face.

28.3.3 *Vertical Reinforcement*

If forces are applied to a deep beam in such a way that hanging action is required, bars or suspension stirrups shall be provided to carry all the forces concerned.

28.3.4 *Side Face Reinforcement*

Side face reinforcement shall comply with requirements of minimum reinforcement of wall (*see* **31.4**).

29 RIBBED, HOLLOW BLOCK OR VOIDED SLAB

29.1 General

This covers the slabs constructed in one of the ways described below:

a) As a series of concrete ribs with topping cast on forms which may be removed after the concrete has set;

b) As a series of concrete ribs between precast blocks which remain part of the completed structure; the top of the ribs may be connected by a topping of concrete of the same strength as that used in the ribs; and

c) With a continuous top and bottom face but containing voids of rectangular, oval or other shape.

29.2 Analysis of Structure

The moments and forces due to design loads on continuous slabs may be obtained by the methods given in Section 5A (c) for solid slabs. Alternatively, the slabs may be designed as a series of simply supported spans provided they are not exposed to weather or corrosive conditions; wide cracks may develop at the supports and the engineer shall satisfy himself that these will not impair finishes or lead to corrosion of the reinforcement.

29.3 Shear

Where hollow blocks are used, for the purpose of calculating shear stress, the rib width may be increased to take account of the wall thickness of the block on one side of the rib; with narrow precast units, the width of the jointing mortar or concrete may be included.

29.4 Deflection

The recommendations for deflection in respect of solid slabs may be applied to ribbed, hollow block or voided construction. The span to effective depth ratios given in **22.2** for a flanged beam are applicable but when calculating the final reduction factor for web width, the rib width for hollow block slabs may be assumed to include the walls of the blocks on both sides of the rib. For voided slabs and slabs constructed of box or I-section units, an effective rib width shall be calculated assuming all material below the upper flange of the unit to be concentrated in a rectangular rib having the same cross-sectional area and depth.

29.5 Size and Position of Ribs

In-situ ribs shall be not less than 65 mm wide. They shall be spaced at centres not greater than 1.5 m apart and their depth, excluding any topping, shall be not more than four times their width. Generally ribs shall be formed along each edge parallel to the span of one way slabs. When the edge is built into a wall or rests on a beam, a rib at least as wide as the bearing shall be formed along the edge.

29.6 Hollow Blocks and Formers

Blocks and formers may be of any suitable material. Hollow clay tiles for the filler type shall conform to accepted standard [6-5A(42)]. When required to contribute to the structural strength of a slab they shall:

a) be made of concrete or burnt clay; and

b) have a crushing strength of at least 14 N/mm^2 measured on the net section when axially loaded in the direction of compressive stress in the slab.

29.7 Arrangement of Reinforcement

The recommendations given in **25.3** regarding maximum distance between bars apply to areas of solid concrete in this form of construction. The curtailment, anchorage and cover to reinforcement shall be as described below:

a) At least 50 percent of the total main reinforcement shall be carried through at the bottom on to the bearing and anchored in accordance with **25.2.3.3**.
b) Where a slab, which is continuous over supports, has been designed as simply supported, reinforcement shall be provided over the support to control cracking. This reinforcement shall have a cross-sectional area of not less than one-quarter that required in the middle of the adjoining spans and shall extend at least one-tenth of the clear span into adjoining spans.
c) In slabs with permanent blocks, the side cover to the reinforcement shall not be less than 10 mm. In all other cases, cover shall be provided according to **25.4**.

29.8 Precasts Joists and Hollow Filler Blocks

The construction with precast joists and hollow concrete filler blocks shall conform to good practice [6-5A(43)] and precast joist and hollow clay filler blocks shall conform to good practice [6-5A(44)].

30 FLAT SLABS

30.1 General

The term flat slab means a reinforced concrete slab with or without drops, supported generally without beams, by columns with or without flared column heads (*see* Fig. 12). A flat slab may be solid slab or may have recesses formed on the soffit so that the soffit comprises a series of ribs in two directions. The recesses may be formed by removable or permanent filler blocks.

30.1.1 For the purpose of this clause, the following definitions shall apply:

a) *Column strip* — Column strip means a design strip having a width of 0.25 l_2, but not greater than 0.25 l_1 on each side of the column centre-line, where l_1 is the span in the direction moments are being determined, measured centre-to-centre of supports and l_2 is the span transverse to l_1, measured centre-to-centre of supports.
b) *Middle strip* — Middle strip means a design strip bounded on each of its opposite sides by the column strip.
c) *Panel* — Panel means that part of a slab bounded on each of its four sides by the centre-line of a column or centre-lines of adjacent spans.

30.2 Proportioning

30.2.1 *Thickness of Flat Slab*

The thickness of the flat slab shall be generally controlled by considerations of span to effective depth ratios given in **22.2**.

For slabs with drops conforming to **30.2.2**, span to effective depth ratios given in **22.2** shall be applied directly; otherwise the span to effective depth ratios obtained in accordance with provisions in **22.2** shall be multiplied by 0.9. For this purpose, the longer span shall be considered. The minimum thickness of slab shall be 125 mm.

30.2.2 *Drop*

The drops when provided shall be rectangular in plan, and have a length in each direction not less than one-third of the panel length in that direction. For exterior panels, the width of drops at right angles to the non-continuous edge and measured from the centre-line of the columns shall be equal to one-half the width of drop for interior panels.

30.2.3 *Column Heads*

Where column heads are provided, that portion of a column head which lies within the largest right circular cone or pyramid that has a vertex angle of 90° and can be included entirely within the outlines of the column and the column head, shall be considered for design purposes (*see* Fig. 12).

30.3 Determination of Bending Moment

30.3.1 *Methods of Analysis and Design*

It shall be permissible to design the slab system by one of the following methods:

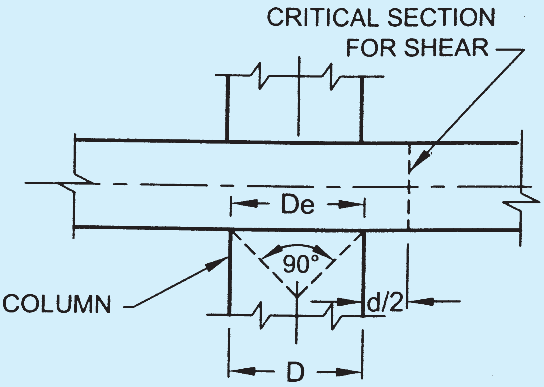
a) The direct design method as specified in **30.4**, and
b) The equivalent frame method as specified in **30.5**.

In each case the applicable limitations given in **30.4** and **30.5** shall be met.

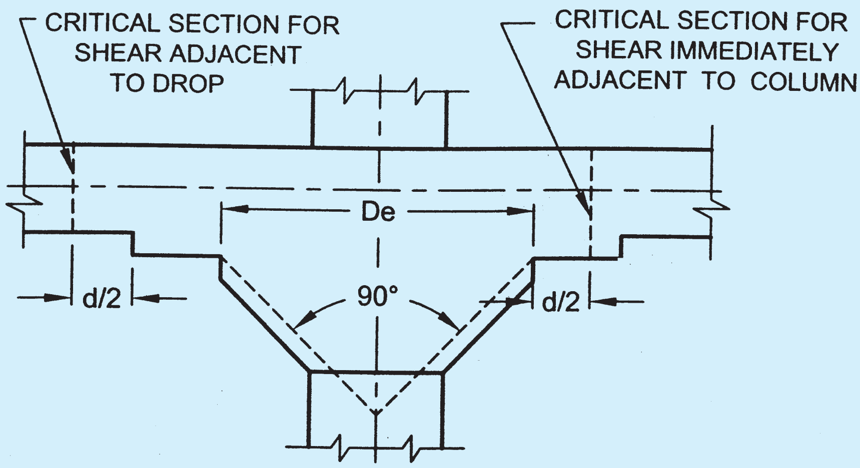
30.3.2 *Bending Moments in Panels with Marginal Beams or Walls*

Where the slab is supported by a marginal beam with a depth greater than 1.5 times the thickness of the slab, or by a wall, then:

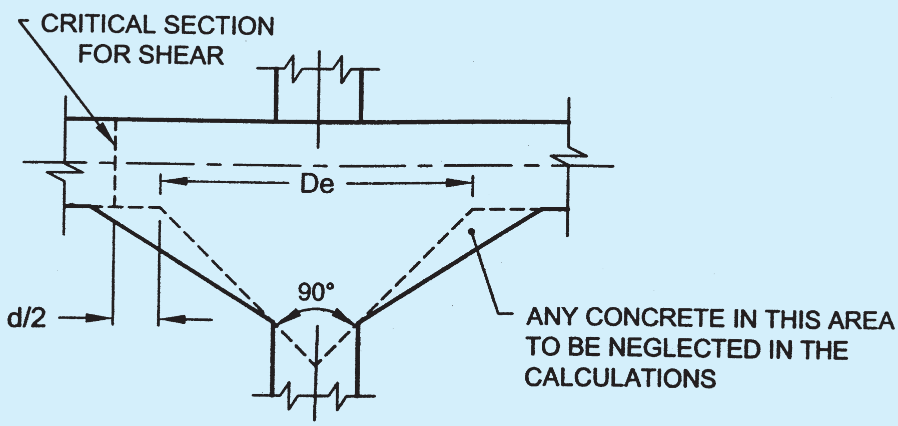
a) the total load to be carried by the beam or wall shall comprise those loads directly on the wall or beam plus a uniformly distributed load equal to one-quarter of the total load on the slab, and



12 A SLAB WITHOUT DROP & COLUMN WITHOUT COLUMN HEAD



12 B SLAB WITH DROP & COLUMN WITH COLUMN HEAD



12 C SLAB WITHOUT DROP & COLUMN WITH COLUMN HEAD

NOTE — D_e is the diameter of column or column head to be considered for design and d is effective depth of slab or drop as appropriate.

FIG. 12 CRITICAL SECTIONS FOR SHEAR IN FLAT SLABS

b) the bending moments on the half-column strip adjacent to the beam or wall shall be one-quarter of the bending moments for the first interior column strip.

30.3.3 *Transfer of Bending Moments to Columns*

When unbalanced gravity load, wind, earthquake, or other lateral loads cause transfer of bending moment between slab and column, the flexural stresses shall be investigated using a fraction, α of the moment given by:

$$\alpha = \frac{1}{1 + \frac{2}{3}\sqrt{a_1 / a_2}}$$

where

a_1 = Overall dimension of the critical section for shear in the direction in which moment acts, and

a_2 = Overall dimension of the critical section for shear transverse to the direction in which moment acts.

A slab width between lines that are one and one-half slab or drop panel thickness; 1.5 D, on each side of the column or capital may be considered effective, D being the size of the column.

Concentration of reinforcement over column head by closer spacing or additional reinforcement may be used to resist the moment on this section.

30.4 Direct Design Method

30.4.1 *Limitations*

Slab system designed by the direct design method shall fulfil the following conditions:

a) There shall be minimum of three continuous spans in each direction,
b) The panels shall be rectangular, and the ratio of the longer span to the shorter span within a panel shall not be greater than 2.0,
c) It shall be permissible to offset columns to a maximum of 10 percent of the span in the direction of the offset notwithstanding the provision in (b),
d) The successive span lengths in each direction shall not differ by more than one-third of the longer span. The end spans may be shorter but not longer than the interior spans, and
e) The design live load shall not exceed three times the design dead load.

30.4.2 *Total Design Moment for a Span*

30.4.2.1 In the direct design method, the total design movement for a span shall be determined for a strip bounded laterally by the centre-line of the panel on each side of the centre-line of the supports.

30.4.2.2 The absolute sum of the positive and average negative bending movements in each direction shall be taken as:

$$M_o = \frac{W\, l_n}{8}$$

where

M_o = Total movement;
W = Design load on an area $l_2\, l_n$;
l_n = Clear span extending from face-to-face of columns, capitals, brackets or walls, but not less than 0.65 l_1;
l_1 = Length of span in the direction of M_o; and
l_2 = Length of span transverse to l_1.

30.4.2.3 Circular supports shall be treated as square supports having the same area.

30.4.2.4 When the transverse span of the panels on either side of the centre-line of supports varies l_2 shall be taken as the average of the transverse spans.

30.4.2.5 When the span adjacent and parallel to an edge is being considered, the distance from the edge to the centre-line of the panel shall be substituted for l_2 in **30.4.2.2**.

30.4.3 *Negative and Positive Design Moments*

30.4.3.1 The negative design moment shall be located at the face of rectangular supports, circular supports being treated as square supports having the same area.

30.4.3.2 In an interior span, the total design moment M_o shall be distributed in the following proportions:

Negative design moment	0.65
Positive design moment	0.35

30.4.3.3 In an end span, the total design moment M_o shall be distributed in the following proportions:

Interior negative design moment:

$$0.75 - \frac{0.10}{1 + \frac{1}{\alpha_c}}$$

Positive design moment:

$$0.63 - \frac{0.28}{1 + \frac{1}{\alpha_c}}$$

Exterior negative design moment:

$$\frac{0.65}{1 + \frac{1}{\alpha_c}}$$

α_c is the ratio of flexural stiffness of the exterior columns to the flexural stiffness of the slab at a joint taken in the direction moments are being determined and is given by

$$\alpha_c = \frac{\Sigma K_c}{K_s}$$

where

K_c = Sum of the flexural stiffness of the columns meeting at the joint; and

K_s = Flexural stiffness of the slab, expressed as moment per unit rotation.

30.4.3.4 It shall be permissible to modify these design moments by up to 10 percent, so long as the total design moment, M_o for the panel in the direction considered is not less than that required by **30.4.2.2**.

30.4.3.5 The negative moment section shall be designed to resist the larger of the two interior negative design moments determined for the spans framing into a common support unless an analysis is made to distribute the unbalanced moment in accordance with the stiffness of the adjoining parts.

30.4.4 *Distribution of Bending Moments Across the Panel Width*

Bending moments at critical cross-section shall be distributed to the column strips and middle strips as specified in **30.5.5** as applicable.

30.4.5 *Moments in Columns*

30.4.5.1 Columns built integrally with the slab system shall be designed to resist moments arising from loads on the slab system.

30.4.5.2 At an interior support, the supporting members above and below the slab shall be designed to resist the moment *M* given by the following equation, in direct proportion to their stiffnesses unless a general analysis is made:

$$M = 0.08 \frac{(w_d + 0.5\, w_l)\, l_2 l_n^2 - w'_d l'_2 l'^2_n}{1 + \frac{1}{\alpha_c}}$$

where

w_d, w_l = Design dead and live loads respectively, per unit area;

l_2 = Length of span transverse to the direction of *M*;

l_n = Length of the clear span in the direction of *M*, measured face to face of supports;

α_c = $\frac{\Sigma K_c}{\Sigma K_s}$ where K_c and K_s are as defined in **30.4.3.3**; and

w'_d, l'_2 and l'_n, refer to the shorter span.

30.4.6 *Effects of Pattern Loading*

In the direct design method, when the ratio of live load to dead load exceeds 0.5;

a) the sum of the flexural stiffness of the columns above and below the slab, ΣK_c, shall be such that α_c is not less than the appropriate minimum value $\alpha_{c\,min}$ specified in Table 17, or

b) if the sum of the flexural stiffnesses of the columns, ΣK_c, does not satisfy (a), the positive design moments for the panel shall be multiplied by the coefficient β_s given by the following equation:

$$\beta_s = 1 + \left[\frac{2 - \frac{w_d}{w_l}}{4 + \frac{w_d}{w_l}}\right]\left(1 - \frac{\alpha_c}{\alpha_{c\,min}}\right)$$

α_c is the ratio of flexural stiffness of the columns above and below the slab to the flexural stiffness of the slabs at a joint taken in the direction moments are being determined and is given by:

$$\alpha_c = \frac{\Sigma K_c}{\Sigma K_s}$$

where K_c and K_s are flexural stiffnesses of column and slab respectively.

Table 17 Minimum Permissible Values of α_c

(*Clause* 30.4.6)

Imposed Load/ Dead Load	Ratio $\frac{l_2}{l_1}$	Value of $\alpha_{c\,min}$
(1)	(2)	(3)
0.5	0.5 to 2.0	0
1.0	0.5	0.6
1.0	0.8	0.7
1.0	1.0	0.7
1.0	1.25	0.8
1.0	2.0	1.2
2.0	0.5	1.3
2.0	0.8	1.5
2.0	1.0	1.6
2.0	1.25	1.9
2.0	2.0	4.9
3.0	0.5	1.8
3.0	0.8	2.0
3.0	1.0	2.3
3.0	1.25	2.8
3.0	2.0	13.0

30.5 Equivalent Frame Method

30.5.1 *Assumptions*

The bending moments and shear forces may be determined by an analysis of the structure as a continuous frame and the following assumptions may be made:

a) The structure shall be considered to be made up of equivalent frames on column lines taken longitudinally and transversely through the building. Each frame consists of a row of equivalent columns or supports, bounded laterally by the centre-line of the panel on each side of the centre-line of the columns or supports. Frames adjacent and parallel to an edge shall be bounded by the edge and the centre-line of the adjacent panel.

b) Each such frame may be analyzed in its entirety, or, for vertical loading, each floor thereof and the roof may be analyzed separately with its columns being assumed fixed at their remote ends. Where slabs are thus analyzed separately, it may be assumed in determining the bending moment at a given support that the slab is fixed at any support two panels distant therefrom provided the slab continuous beyond the point.

c) For the purpose of determining relative stiffness of members, the moment of inertia of any slab or column may be assumed to be that of the gross cross-section of the concrete alone.

d) Variations of moment of inertia along the axis of the slab on account of provision of drops shall be taken into account. In the case of recessed or coffered slab which is made solid in the region of the columns, the stiffening effect may be ignored provided the solid part of the slab does not extend more than 0.15 l_{ef}, into the span measured from the centre-line of the columns. The stiffening effect of flared column heads may be ignored.

30.5.2 *Loading Pattern*

30.5.2.1 When the loading pattern is known, the structure shall be analyzed for the load concerned.

30.5.2.2 When the live load is variable but does not exceed three-quarters of the dead load, or the nature of the live load is such that all panels will be loaded simultaneously, the maximum moments may be assumed to occur at all sections when full design live load is on the entire slab system.

30.5.2.3 For other conditions of live load/dead load ratio and when all panels are not loaded simultaneously:

a) maximum positive moment near midspan of a panel may be assumed to occur when three-quarters of the full design live load is on the panel and on alternate panels; and

b) maximum negative moment in the slab at a support may be assumed to occur when three-quarters of the full design live load is on the adjacent panels only.

30.5.2.4 In no case shall design moments be taken to be less than those occurring with full design live load on all panels.

30.5.3 *Negative Design Moment*

30.5.3.1 At interior supports, the critical section for negative moment, in both the column strip and middle strip, shall be taken at the face of rectilinear supports, but in no case at a distance greater than 0.175 l_1 from the centre of the column where l_1 is the length of the span in the direction moments are being determined, measured centre-to-centre of supports.

30.5.3.2 At exterior supports provided with brackets or capitals, the critical section for negative moment in the direction perpendicular to the edge shall be taken at a distance from the face of the supporting element not greater than one-half the projection of the bracket or capital beyond the face of the supporting element.

30.5.3.3 Circular or regular polygon shaped supports shall be treated as square supports having the same area.

30.5.4 *Modification of Maximum Moment*

Moments determined by means of the equivalent frame method, for slabs which fulfil the limitations of **30.4** may be reduced in such proportion that the numerical sum of the positive and average negative moments is not less than the value of total design moment M_o specified in **30.4.2.2**.

30.5.5 *Distribution of Bending Moment Across the Panel Width*

30.5.5.1 *Column strip: Negative moment at an interior support*

At an interior support, the column strip shall be designed to resist 75 percent of the total negative moment in the panel at that support.

30.5.5.2 *Column strip: Negative moment at a exterior support*

a) At an exterior support, the column strip shall be designed to resist the total negative moment in the panel at that support.

b) Where the exterior support consists of a column or a wall extending for a distance equal to or greater than three-quarters of the value of l_2, the length of span transverse to the direction moments are being determined, the exterior negative moment shall be considered to be uniformly distributed across the length l_2.

30.5.5.3 *Column strip: Positive moment for each span*

For each span, the column strip shall be designed to resist 60 percent of the total positive moment in the panel.

30.5.5.4 *Moments in the middle strip*

The middle strip shall be designed on the following bases:

a) That portion of the design moment not resisted by the column strip shall be assigned to the adjacent middle strips.
b) Each middle strip shall be proportioned to resist the sum of the moments assigned to its two half middle strips.
c) The middle strip adjacent and parallel to an edge supported by a wall shall be proportioned to resist twice the moment assigned to half the middle strip corresponding to the first row of interior columns.

30.6 Shear in Flat Slab

30.6.1 The critical section for shear shall be at a distance $d/2$ from the periphery of the column/capital/ drop panel, perpendicular to the plane of the slab where d is the effective depth of the section (*see* Fig. 12). The shape in plan is geometrically similar to the support immediately below the slab (*see* Fig. 13A and Fig. 13B).

NOTE — For column sections with re-entrant angles, the critical section shall be taken as indicated in Fig. 13C and 13D.

30.6.1.1 In the case of columns near the free edge of a slab, the critical section shall be taken as shown in Fig. 14.

30.6.1.2 When openings in flat slabs are located at a distance less than ten times the thickness of the slab from a concentrated reaction or when the openings are located within the column strips, the critical sections specified in **30.6.1** shall be modified so that the part of the periphery of the critical section which is enclosed by radial projections of the openings to the centroid of the reaction area shall be considered ineffective (*see* Fig. 15), and openings shall not encroach upon column head.

30.6.2 *Calculation of Shear Stress*

The shear stress τ_v shall be the sum of the values calculated according to **30.6.2.1** and **30.6.2.2.**

30.6.2.1 The nominal shear stress in flat slabs shall be taken as $V/b_o d$ where V is the shear force due to design load, b_o is the periphery of the critical section and d is the effective depth.

30.6.2.2 When unbalanced gravity load, wind, earthquake or other forces cause transfer of bending moment between slab and column, a fraction $(1 - \alpha)$ of the moment shall be considered transferred by eccentricity of the shear about the centroid of the critical section. Shear stresses shall be taken as varying linearly about the centroid of the critical section. The value of α shall be obtained from the equation given in **30.3.3**.

30.6.3 *Permissible Shear Stress*

30.6.3.1 When shear reinforcement is not provided, the calculated shear stress at the critical section shall not exceed $k_s \tau_c$,

where

k_s = $(0.5 + \beta_c)$ but not greater than 1, β_c being the ratio of short side to long side of the column/capital; and

τ_c = $0.25\sqrt{f_{ck}}$ in limit state method of design, and $0.16\sqrt{f_{ck}}$ in working stress method of design.

30.6.3.2 When the shear stress at the critical section exceeds the value given in **30.6.3.1**, but less than 1.5 τ_c shear reinforcement shall be provided. If the shear stress 1.5 τ_c, the flat slab shall be redesigned. Shear stresses shall be investigated at successive sections more distant from the support and shear reinforcement shall be provided up to a section where the shear stress does not exceed 0.5 τ_c. While designing the shear reinforcement, the shear stress carried by the concrete shall be assumed to be 0.5 τ_c and reinforcement shall carry the remaining shear.

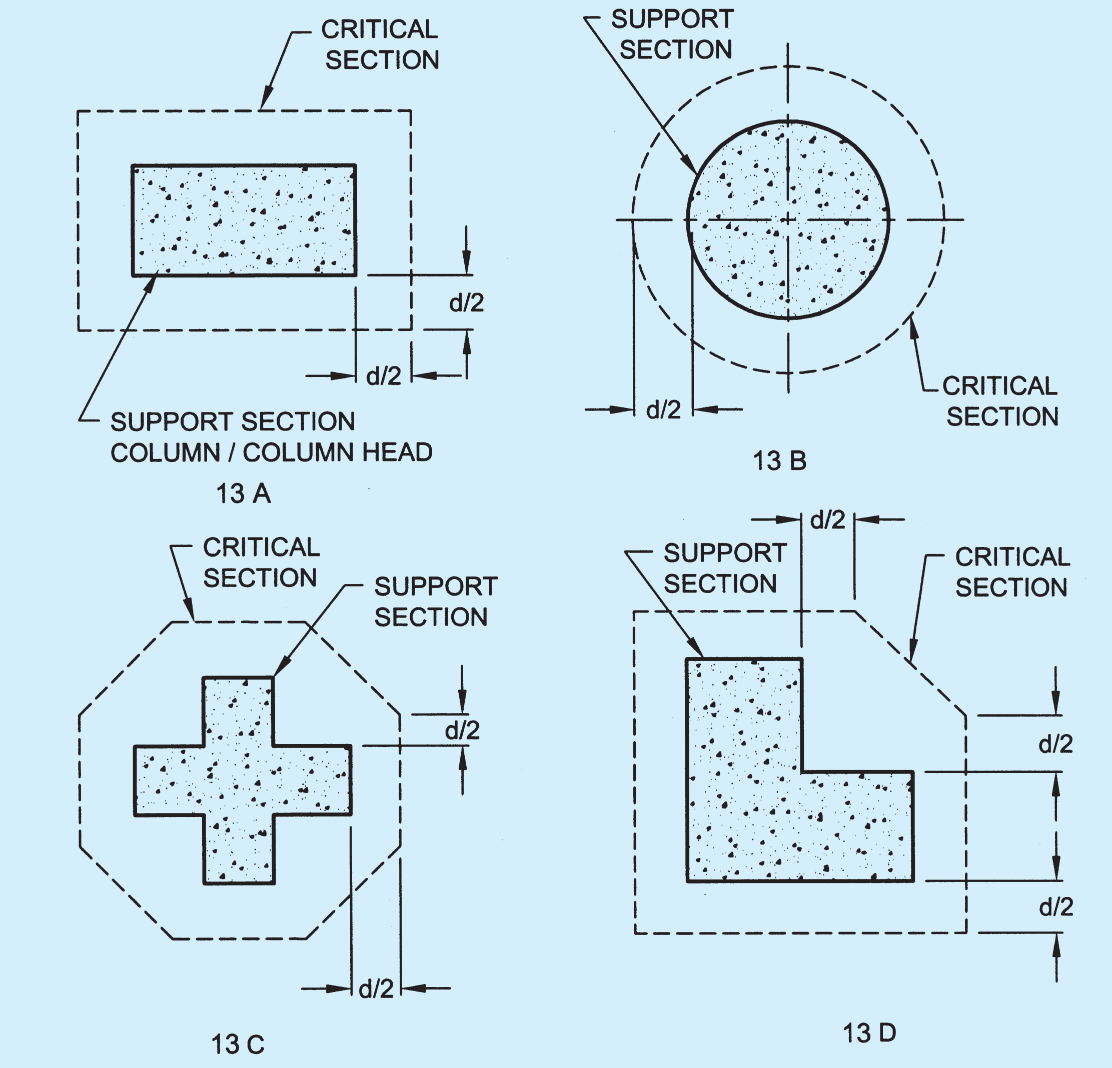
30.7 Slab Reinforcement

30.7.1 *Spacing*

The spacing of bars in a flat slab, shall not exceed 2 times the slab thickness, except where a slab is of cellular or ribbed construction.

30.7.2 *Area of Reinforcement*

When drop panels are used, the thickness of drop panel for determination of area of reinforcement shall be the lesser of the following:



NOTE — *d* is the effective depth of the flat slab/drop.

FIG. 13 CRITICAL SECTIONS IN PLAN FOR SHEAR IN FLAT SLABS

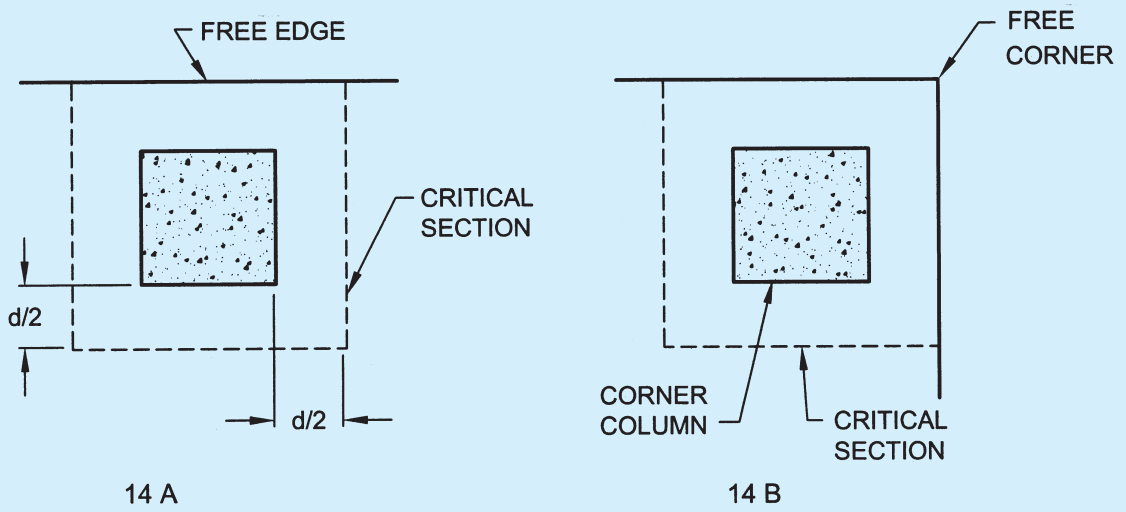


FIG. 14 EFFECT OF FREE EDGES ON CRITICAL SECTION FOR SHEAR

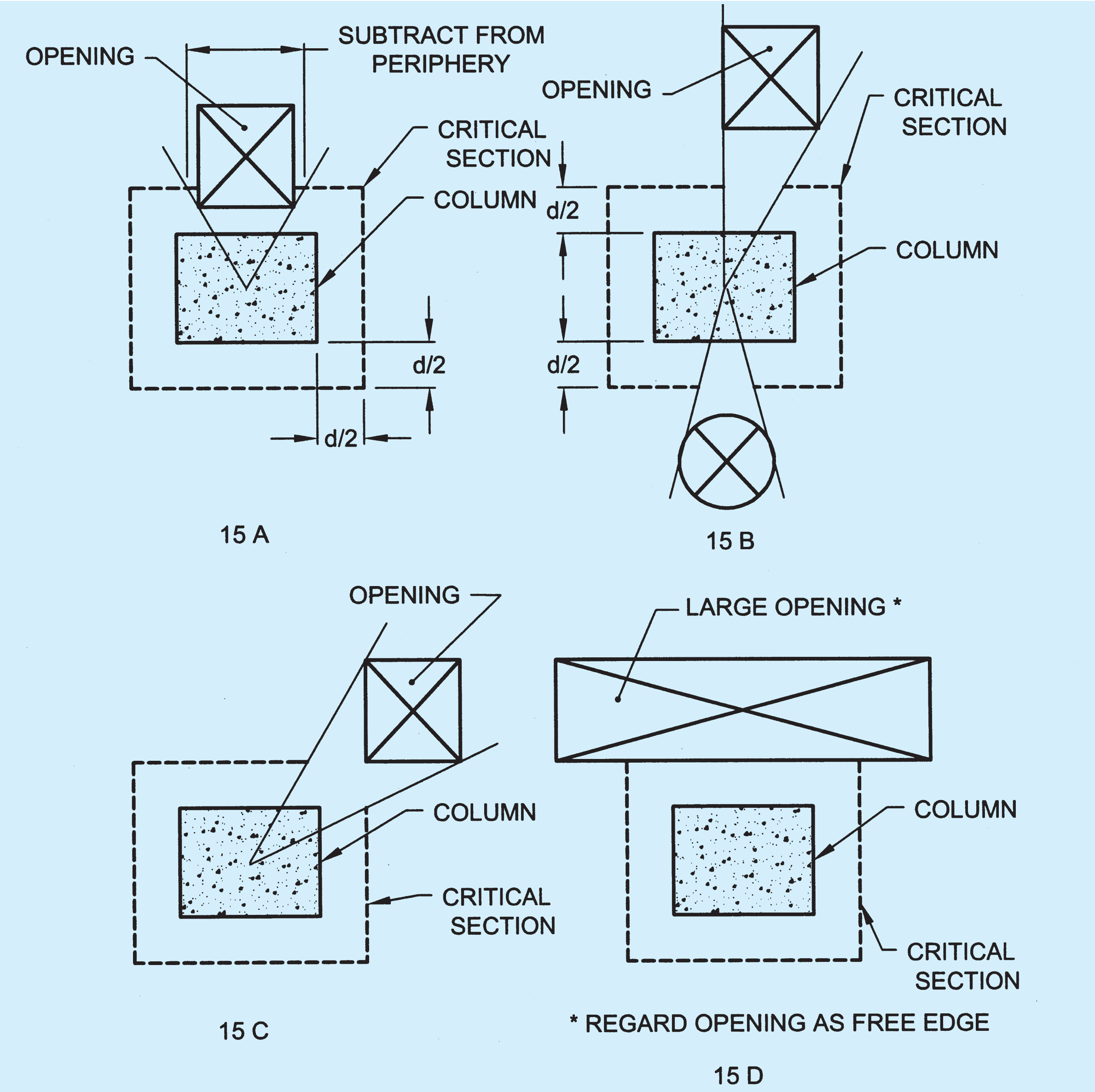


FIG. 15 EFFECT OF OPENINGS ON CRITICAL SECTION FOR SHEAR

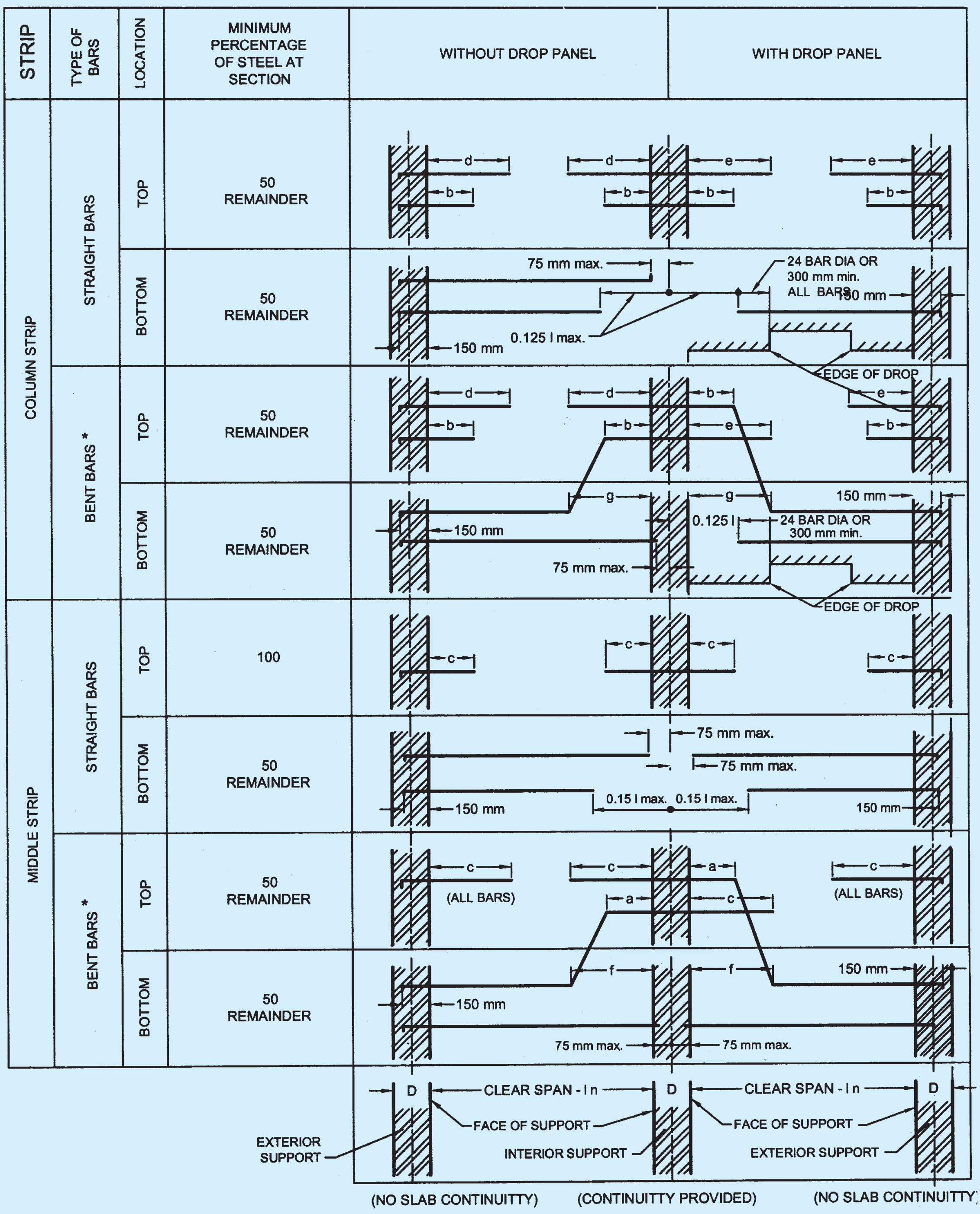
a) Thickness of drop, and
b) Thickness of slab plus one quarter the distance between edge of drop and edge of capital.

30.7.3 *Minimum Length of Reinforcement*

a) Reinforcement in flat slabs shall have the minimum lengths specified in Fig. 16. Larger lengths of reinforcement shall be provided when required by analysis.
b) Where adjacent spans are unequal, the extension of negative reinforcement beyond each face of the common column shall be based on the longer span.
c) The length of reinforcement for slabs in frames not braced against sideways and for slabs resisting lateral loads shall be determined by analysis but shall not be less than those prescribed in Fig. 16.

30.7.4 *Anchoring Reinforcement*

a) All slab reinforcement perpendicular to a discontinuous edge shall have an anchorage (straight, bent or otherwise anchored) past the internal face of the spandrel beam, wall or column, of an amount:
   1) *For positive reinforcement* — not less than 150 mm except that with fabric reinforcement having a fully welded transverse wire directly over the support,



	Bar Length from Face of Support						
	Minimum Length					*Maximum Length*	
Mark	*a*	*b*	*c*	*d*	*e*	*f*	*g*
Length	0.14 l_n	0.20 l_n	0.22 l_n	0.30 l_n	0.33 l_n	0.20 l_n	0.24 l_n

* Bent bars at exterior supports may be used if a general analysis is made.

NOTE — *D* is the diameter of the column and the dimension of the rectangular column in the direction under consideration.

FIG. 16 MINIMUM BEND JOINT LOCATION AND EXTENSION FOR REINFORCEMENT IN FLAT SLABS it shall be permissible to reduce this length to one-half of the width of the support or 50 mm, whichever is greater; and

2) *For negative reinforcement* — such that the design stress is developed at the internal face, in accordance with Section 5A (c).

b) Where the slab is not supported by a spandrel beam or wall, or where the slab cantilevers beyond the support, the anchorage shall be obtained within the slab.

30.8 Openings in Flat Slabs

Openings of any size may be provided in the flat slab if it is shown by analysis that the requirements of strength and serviceability are met. However, for openings conforming to the following, no special analysis is required.

a) Openings of any size may be placed within the middle half of the span in each direction, provided the total amount of reinforcement required for the panel without the opening is maintained.
b) In the area common to two column strips, not more than one-eighth of the width of strip in either span shall be interrupted by the openings. The equivalent of reinforcement interrupted shall be added on all sides of the openings.
c) In the area common to one column strip and one middle strip, not more than one-quarter of the reinforcement in either strip shall be interrupted by the openings. The equivalent of reinforcement interrupted shall be added on all sides of the openings.
d) The shear requirements of **30.6** shall be satisfied.

31 WALLS

31.1 General

Reinforced concrete walls subjected to direct compression or combined flexure and direct compression should be designed in accordance with Section 5 or Annex B provided the vertical reinforcement is provided in each face. Braced walls subjected to only vertical compression may be designed as per empirical procedure given in **31.2**. The minimum thickness of walls shall be 100 mm.

31.1.1 Guidelines or design of walls subjected to horizontal and vertical loads are given in **31.3**.

31.2 Empirical Design Method for Walls Subjected to Inplane Vertical Loads

31.2.1 *Braced Walls*

Walls shall be assumed to be braced if they are laterally supported by a structure in which all the following apply:

a) Walls or vertical braced elements are arranged in two directions so as to provide lateral stability to the structure as a whole.
b) Lateral forces are resisted by shear in the planes of these walls or by braced elements.
c) Floor and roof systems are designed to transfer lateral forces.
d) Connections between the wall and the lateral supports are designed to resist a horizontal force not less than
   1) the simple static reactions to the total applied horizontal forces at the level of lateral support; and
   2) 2.5 percent of the total vertical load that the wall is designed to carry at the level of lateral support.

31.2.2 *Eccentricity of Vertical Load*

The design of a wall shall take account of the actual eccentricity of the vertical force subject to a minimum value of 0.05 t.

The vertical load transmitted to a wall by a discontinuous concrete floor or roof shall be assumed to act at one-third the depth of the bearing area measured from the span face of the wall. Where there is an *in-situ* concrete floor continuous over the wall, the load shall be assumed to act at the centre of the wall.

The resultant eccentricity of the total vertical load on a braced wall at any level between horizontal lateral supports, shall be calculated on the assumption that the resultant eccentricity of all the vertical loads above the upper support is zero.

31.2.3 *Maximum Effective Height to Thickness Ratio*

The ratio of effective height to thickness, H_{we}/t, it shall not exceed 30.

31.2.4 *Effective Height*

The effective height of a braced wall shall be taken as follows:

a) Where restrained against rotation at both ends by

1) floors	$0.75\ H_w$ or
2) intersecting walls or similar members whichever is the lesser.	$0.75\ L_1$

b) Where not restrained against rotation at both ends by
   1) floors — $1.0\ H_w$ or
   2) intersecting walls or similar members whichever is the lesser. — $1.0\ L_1$

where

H_w = Unsupported height of the wall.

L_1 = Horizontal distance between centres of lateral restraint.

31.2.5 *Design Axial Strength of Wall*

The design axial strength P_{uw} per unit length of a braced wall in compression may be calculated from the following equation:

$$P_{uw} = 0.3\ (t - 1.2\ e - 2\ e_a)\ f_{ck}$$

where

t = Thickness of the wall,

e = Eccentricity of load measured at right angles to the plane of the wall determined in accordance with **31.2.2**, and

e_a = Additional eccentricity due to slenderness effect taken as $H^2_{we}/2\ 500\ t$.

31.3 Walls Subjected to Combined Horizontal and Vertical Forces

31.3.1 When horizontal forces are in the plane of the wall, it may be designed for vertical forces in accordance with **31.2** and for horizontal shear in accordance with **31.4**. In plane bending may be neglected in case a horizontal cross-section of the wall is always under compression due to combined effect of horizontal and vertical loads.

31.3.2 Walls subjected to horizontal forces perpendicular to the wall and for which the design axial load does not exceed $0.04\ f_{ck} A_g$, shall be designed as slabs in accordance with the appropriate provisions given in **23**, where A_g is gross area of the section.

31.4 Design for Horizontal Shear

31.4.1 *Critical Section for Shear*

The critical section for maximum shear shall be taken at a distance from the base of $0.5\ L_w$ or $0.5\ H_w$ whichever is less.

31.4.2 *Nominal Shear Stress*

The nominal shear stress τ_{vw} in walls shall be obtained as follows:

$$\tau_{vw} = V_u / t.d$$

where

V_u = Shear force due to design loads,

t = Wall thickness,

d = $0.8 \times L_w$ where L_w is the length of the wall.

31.4.2.1 Under no circumstances shall the nominal shear stress τ_{cw} in walls exceed $0.17\ f_{ck}$ in limit state method and $0.12\ f_{ck}$ in working stress method.

31.4.3 *Design Shear Strength of Concrete*

The design shear strength of concrete in walls, τ_{cw}, without shear reinforcement shall be taken as below:

a) For $H_w / L_w \leq 1$

   $$\tau_{cw} = (3.0 - H_w/L_w)\ K_1 \sqrt{f_{ck}}$$

   where K_1 is 0.2 in limit state method and 0.13 in working stress method.

b) For $H_w / L_w > 1$

   Lesser of the values calculated from (a) above and from

   $$\tau_{cw} = K_2 \sqrt{f_{ck}}\ \frac{(H_w / L_w + 1)}{(H_w / L_w - 1)}$$

   where K_2 is 0.045 in limit state method and 0.03 in working stress method, but τ_{cw} shall be not less than $K_3\ \sqrt{f_{ck}}$ in any case where K_3 is 0.15 in limit state method and 0.10 in working stress method.

31.4.4 *Design of Shear Reinforcement*

Shear reinforcement shall be provided to carry a shear equal to $V_u - \tau_{cw}.t(0.8\ L_w)$. In case of working stress method V_u is replaced by V. The strength of shear reinforcement shall be calculated as per **39.4** or **B-5.4** with A_{av} defined as below:

$$A_{av} = P_w\ (0.8\ L_w)\ t$$

where P_w is determined as follows:

a) For walls where $H_w/L_w \leq 1$, P_w shall be the lesser of the ratios of either the vertical reinforcement area or the horizontal reinforcement area to the cross-sectional area of wall in the respective direction.

b) For walls where $H_w/L_w > 1$, P_w shall be the ratio of the horizontal reinforcement area to the cross-sectional area of wall per vertical metre.

31.5 Minimum Requirements for Reinforcement in Walls

The reinforcement for walls shall be provided as below:

a) the minimum ratio of vertical reinforcement to gross concrete area shall be:
   1) 0.001 2 for deformed bars not larger than 16 mm in diameter and with a

characteristic strength of 415 N/mm^2 or greater.
2) 0.001 5 for other types of bars.
3) 0.001 2 for welded wire fabric not larger than 16 mm in diameter.

b) Vertical reinforcement shall be spaced not farther apart than three times the wall thickness nor 450 mm.

c) The minimum ratio of horizontal reinforcement to gross concrete area shall be:
   1) 0.002 0 for deformed bars not larger than 16 mm in diameter and with a characteristic strength of 415 N/mm^2 or greater.
   2) 0.002 5 for other types of bars.
   3) 0.002 0 for welded wire fabric not larger than 16 mm in diameter.

d) Horizontal reinforcement shall be spaced not farther apart than three times the wall thickness nor 450 mm.

NOTE — The minimum reinforcement may not always be sufficient to provide adequate resistance to the effects of shrinkage and temperature.

31.5.1 For walls having thickness more than 200 mm, the vertical and horizontal reinforcement shall be provided in two grids, one near each face of the wall.

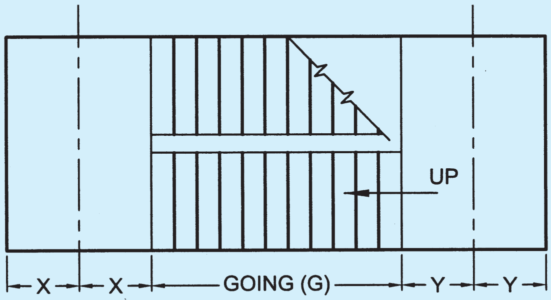
31.5.2 Vertical reinforcement need not be enclosed by transverse reinforcement as given in **25.5.3.2** for column, if the vertical reinforcement is not greater than 0.01 times the gross sectional area or where the vertical reinforcement is not required for compression.

32 STAIRS

32.1 Effective Span of Stairs

The effective span of stairs without stringer beams shall be taken as the following horizontal distances:

a) Where supported at top and bottom risers by beams spanning parallel with the risers, the distance centre-to-centre of beams;
b) Where spanning on to the edge of a landing slab, which spans parallel, with the risers (*see* Fig. 17), a distance equal to the going of the stairs plus at each end either half the width of the landing or one metre, whichever is smaller; and
c) Where the landing slab spans in the same direction as the stairs, they shall be considered as acting together to form a single slab and the span determined as the distance centre-to-centre of the supporting beams or walls, the going being measured horizontally.



X	Y	SPAN IN METRES
< 1 m	< 1 m	G + X + Y
< 1 m	≥ 1 m	G + X + 1
≥ 1 m	< 1 m	G + Y + 1
≥ 1 m	≥ 1 m	G + 1 + 1

FIG. 17 EFFECTIVE SPAN FOR STAIRS SUPPORTED AT EACH END BY LANDINGS SPANNING PARALLEL WITH THE RISERS

32.2 Distribution of Loading on Stairs

In the case of stairs with open wells, where spans partly crossing at right angles occur, the load on areas common to any two such spans may be taken as one-half in each direction as shown in Fig. 18. Where flights or landings are embedded into walls for a length of not less than 110 mm and are designed to span in the direction of the flight, a 150 mm strip may be deducted from the loaded area and the effective breadth of the section increased by 75 mm for purposes of design (*see* Fig. 19).

32.3 Depth of Section

The depth of section shall be taken as the minimum thickness perpendicular to the soffit of the staircase.

33 FOOTINGS

33.1 General

Footings shall be designed to sustain the applied loads, moments and forces and the induced reactions and to ensure that any settlement which may occur shall be as nearly uniform as possible, and the safe bearing capacity of the soil is not exceeded {*see* good practice [6-5A(35)] }.

33.1.1 In sloped or stepped footings the effective cross-section in compression shall be limited by the area above the neutral plane, and the angle of slope or depth and location of steps shall be such that the design

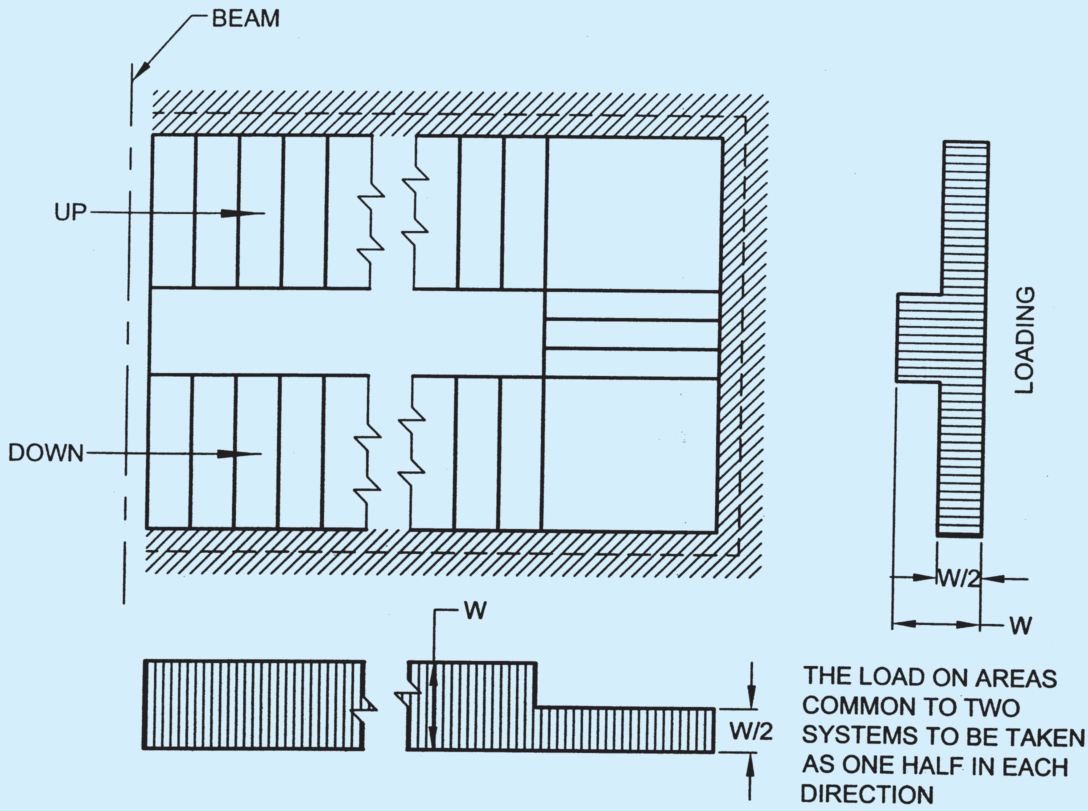


FIG. 18 LOADING ON STAIRS WITH OPEN WELLS

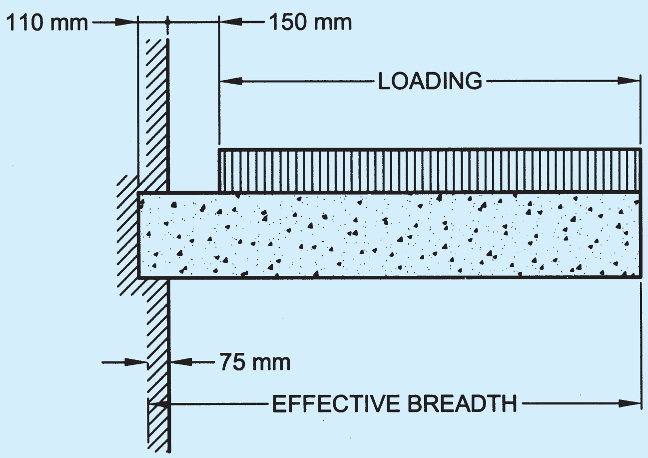


FIG. 19 LOADING ON STAIRS BUILT INTO WALLS

requirements are satisfied at every section. Sloped and stepped footings that are designed as a unit shall be constructed to assure action as a unit.

33.1.2 *Thickness at the Edge of Footing*

In reinforced and plain concrete footings, the thickness at the edge shall be not less than 150 mm for footings on soils, nor less than 300 mm above the tops of piles for footings on piles.

33.1.3 In the case of plain concrete pedestals, the angle between the plane passing through the bottom edge of the pedestal and the corresponding junction edge of the column with pedestal and the horizontal plane (*see* Fig. 20) shall be governed by the expression:

$$\tan \alpha \nleq 0.9 \sqrt{\frac{100\, q_{o}}{f_{ck}} + 1}$$

where

q_o = Calculated maximum bearing pressure at the base of the pedestal in N/mm², and

f_{ck} = Characteristic strength of concrete at 28 days in N/mm²

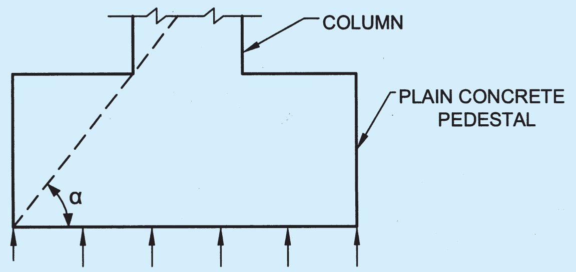


FIG. 20

33.2 Moments and Forces

33.2.1 In the case of footings on piles, computation for moments and shears may be based on the assumption that the reaction from any pile is concentrated at the centre of the pile.

33.2.2 For the purpose of computing stresses in footings which support a round or octagonal concrete column or pedestal, the face of the column or pedestal shall be taken as the side of a square inscribed within the perimeter of the round or octagonal column or pedestal.

33.2.3 *Bending Moment*

33.2.3.1 The bending moment at any section shall be determined by passing through the section a vertical plane which extends completely across the footing, and computing the moment of the forces acting over the entire area of the footing on one side of the said plane.

33.2.3.2 The greatest bending moment to be used in the design of an isolated concrete footing which supports a column, pedestal or wall, shall be the moment computed in the manner prescribed in **33.2.3.1** at sections located as follows:

a) At the face of the column, pedestal or wall, for footings supporting a concrete column, pedestal or wall;
b) Halfway between the centre-line and the edge of the wall, for footings under masonry walls; and
c) Halfway between the face of the column or pedestal and the edge of the gussetted base, for footings under gussetted bases.

33.2.4 *Shear and Bond*

33.2.4.1 The shear strength of footings is governed by the more severe of the following two conditions:

a) the footing acting essentially as a wide beam, with a potential diagonal crack extending in a plane across the entire width; the critical section for this condition shall be assumed as a vertical section located from the face of the column, pedestal or wall at a distance equal to the effective depth of footing in case of footings on soils, and at a distance equal to half the effective depth of footing for footings on piles.
b) Two-way action of the footing, with potential diagonal cracking along the surface of truncated cone or pyramid around the concentrated load; in this case, the footing shall be designed for shear in accordance with appropriate provisions specified in **30.6**.

33.2.4.2 In computing the external shear on any section through a footing supported on piles, the entire reaction from any pile of diameter D_p whose centre is located $D_p/2$ or more outside the section shall be assumed as producing shear on the section; the reaction from any pile whose centre is located $D_p/2$ or more inside the section shall be assumed as producing no shear on the section. For intermediate positions of the pile centre, the portion of the pile reaction to be assumed as producing shear on the section shall be based on straight line interpolation between full value at $D_p/2$ outside the section and zero value at $D_p/2$ inside the section.

33.2.4.3 The critical section for checking the development length in a footing shall be assumed at the same planes as those described for bending moment in **33.2.3** and also at all other vertical planes where abrupt changes of section occur. If reinforcement is curtailed, the anchorage requirements shall be checked in accordance with **25.2.3**.

33.3 Tensile Reinforcement

The total tensile reinforcement at any section shall provide a moment of resistance at least equal to the bending moment on the section calculated in accordance with **33.2.3**.

33.3.1 Total tensile reinforcement shall be distributed across the corresponding resisting section as given below:

a) In one-way reinforced footing, the reinforcement extending in each direction shall be distributed uniformly across the full width of the footing;
b) In two-way reinforced square footing, the reinforcement extending in each direction shall be distributed uniformly across the full width of the footing; and

c) In two-way reinforced rectangular footing, the reinforcement in the long direction shall be distributed uniformly across the full width of the footing. For reinforcement in the short direction, a central band equal to the width of the footing shall be marked along the length of the footing and portion of the reinforcement determined in accordance with the equation given below shall be uniformly distributed across the central band:

$$\frac{\text{Reinforcement in central band width}}{\text{Total reinforcement in short direction}} = \frac{2}{\beta + 1}$$

where β is the ratio of the long side to the short side of the footing. The remainder of the reinforcement shall be uniformly distributed in the outer portions of the footing.

33.4 Transfer of Load at the Base of Column

The compressive stress in concrete at the base of a column or pedestal shall be considered as being transferred by bearing to the top of the supporting pedestal or footing. The bearing pressure on the loaded area shall not exceed the permissible bearing stress in direct compression multiplied by a value equal to

$$\sqrt{\frac{A_1}{A_2}} \text{ but not greater than 2;}$$

where

A_1 = Supporting area for bearing of footing, which in sloped or stepped footing may be taken as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base, the area actually loaded and having side slope of one vertical to two horizontal; and

A_2 = Loaded area at the column base.

For working stress method of design the permissible bearing stress on full area of concrete shall be taken as 0.25 f_{ck}; for limit state method of design the permissible bearing stress shall be 0.45 f_{ck}.

33.4.1 Where the permissible bearing stress on the concrete in the supporting or supported member would be exceeded, reinforcement shall be provided for developing the excess force, either by extending the longitudinal bars into the supporting member, or by dowels (*see* **33.4.3**).

33.4.2 Where transfer of force is accomplished by reinforcement, the development length of the reinforcement shall be sufficient to transfer the compression or tension to the supporting member in accordance with **25.2**.

33.4.3 Extended longitudinal reinforcement or dowels of at least 05 percent of the cross-sectional area of the supported column or pedestal and a minimum of four bars shall be provided. Where dowels are used, their diameter shall not exceed the diameter of the column bars by more than 3 mm.

33.4.4 Column bars of diameters larger than 36 mm, in compression only can be dowelled at the footings with bars of smaller size of the necessary area. The dowel shall extend into the column, a distance equal to the development length of the column bar and into the footing, a distance equal to the development length of the dowel.

33.5 Nominal Reinforcement

33.5.1 Minimum reinforcement and spacing shall be as per the requirements of solid slab.

33.5.2 The nominal reinforcement for concrete sections of thickness greater than 1 m shall be 360 mm^2 per metre length in each direction on each face. This provision does not supersede the requirement of minimum tensile reinforcement based on the depth of the section.

SECTION 5A (e) STRUCTURAL DESIGN (LIMIT STATE METHOD)

34 SAFETY AND SERVICEABILITY REQUIREMENTS

34.1 General

In the method of design based on limit state concept, the structure shall be designed to withstand safely all loads liable to act on it throughout its life; it shall also satisfy the serviceability requirements, such as limitations on deflection and cracking. The acceptable limit for the safety and serviceability requirements before failure occurs is called a 'limit state'. The aim of design is to achieve acceptable probabilities that the structure will not become unfit for the use for which it is intended, that is, that it will not reach a limit state.

34.1.1 All relevant limit states shall be considered in design to ensure an adequate degree of safety and serviceability. In general, the structure shall be designed on the basis of the most critical limit state and shall be checked for other limit states.

34.1.2 For ensuring the above objective, the design should be based on characteristic values for material strengths and applied loads, which take into account the 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 are not available they should be based on experience.

The 'design values' are derived from the characteristic values through the use of partial safety factors, one for material strengths and the other for loads. In the absence of special considerations these factors should have the values given in **35** according to the material, the type of loading and the limit state being considered.

34.2 Limit State of Collapse

The limit state of collapse of the structure or part of the structure could be assessed from rupture of one or more critical sections and from buckling due to elastic or plastic instability (including the effects of sway where appropriate) or overturning. The resistance to bending, shear, torsion and axial loads at every section shall not be less than the appropriate value at that section produced by the probable most unfavourable combination of loads on the structure using the appropriate partial safety factors.

34.3 Limit States of Serviceability

34.3.1 *Deflection*

Limiting values of deflections are given in **22.2**.

34.3.2 *Cracking*

Cracking of concrete should not adversely affect the appearance or durability of the structure; the acceptable limits of cracking would vary with the type of structure and environment. Where specific attention is required to limit the designed crack width to a particular value, crack width calculation may be done using formula given in Annex F.

The practical objective of calculating crack width is merely to give guidance to the designer in making appropriate structural arrangements and in avoiding gross errors in design, which might result in concentration and excessive width of flexural crack.

The surface width of the cracks should not, in general, exceed 0.3 mm in members where cracking is not harmful and does not have any serious adverse effects upon the preservation of reinforcing steel nor upon the durability of the structures. In members where cracking in the tensile zone is harmful either because they are exposed to the effects of the weather or continuously exposed to moisture or in contact soil or ground water, an upper limit of 0.2 mm is suggested for the maximum width of cracks. For particularly aggressive environment, such as the 'sever' category in Table 3, the assessed surface width of cracks should not in general, exceed 0.1 mm.

34.4 Other Limit States

Structures designed for unusual or special functions shall comply with any relevant additional limit state considered appropriate to that structure.

35 CHARACTERISTIC AND DESIGN VALUES AND PARTIAL SAFETY FACTORS

35.1 Characteristic Strength of Materials

The term 'characteristic strength' means that value of the strength of the material below which not more than 5 percent of the test results are expected to fall. The characteristic strength for concrete shall be in accordance with Table 2. Until the relevant Indian Standard Specifications for reinforcing steel are modified to include the concept of characteristic strength, the characteristic value shall be assumed as the minimum yield stress/0.2 percent proof stress specified in the relevant Indian Standard Specifications.

35.2 Characteristic Loads

The term 'characteristic load' means that value of load which has a 95 percent probability of not being exceeded during the life of the structure. Since data are not available to express loads in statistical terms, for the purpose of this Section, dead loads, imposed loads, wind loads, snow load in accordance with the good practice [6-5A(33)] and seismic forces in accordance with the good practice [6-5A(34)] shall be assumed as the characteristic loads.

35.3 Design Values

35.3.1 *Materials*

The design strength of the materials, f_d is given by

$$f_d = \frac{f}{\gamma_m}$$

where

f = Characteristic strength of the material (*see* **35.1**), and

γ_m = Partial safety factor appropriate to the material and the limit state being considered.

35.3.2 *Loads*

The design load, F_d is given by

$$F_d = F\gamma_f$$

where

F = Characteristic load (*see* **35.2**), and

γ_f = Partial safety factor appropriate to the nature of loading and the limit state being considered.

35.3.3 *Consequences of Attaining Limit State*

Where the consequences of a structure attaining a limit state are of a serious nature such as huge loss of life and disruption of the economy, higher values for γ_f and γ_m than those given under **35.4.1** and **35.4.2** may be applied.

35.4 Partial Safety Factors

35.4.1 *Partial Safety Factor* γ_f *for Loads*

The values γ_f given in Table 18 shall normally be used.

Table 18 Values of Partial Safety Factor γ_f for Loads

(*Clauses* 17.2.3.1, 35.4.1 *and* B-4.3)

Load Combination	Limit State of Collapse			Limit States of Serviceability		
	DL	*IL*	*WL*	*DL*	*IL*	*WL*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
DL+ IL	1.5		—	1.0	1.0	—
DL+ WL	1.5 or 0.9[1)]	—	1.5	1.0	—	1.0
DL+ IL+ WL	1.2			1.0	0.8	0.8

NOTES

1 While considering earthquake effects, substitute *EL* for *WL.*

2 For the limit states of serviceability, the values of γ_f given in this table are applicable for short-term effects. While assessing the long-term effects due to creep the dead load and that part of the live load likely to be permanent may only be considered.

[1)] This value is to be considered when stability against overturning or stress reversal is critical.

35.4.2 *Partial Safety Factor* γ_m *for Material Strength*

35.4.2.1 When assessing the strength of a structure or structural member for the limit state of collapse, the values of partial safety factor, γ_m should be taken as 1.5 for concrete and 1.15 for steel.

NOTE — γ_m values are already incorporated in the equations and tables given in this Section for limit state design.

35.4.2.2 When assessing the deflection, the material properties such as modulus of elasticity should be taken as those associated with the characteristic strength of the material.

36 ANALYSIS

36.1 Analysis of Structure

Method of analysis as in **21** shall be used. The material strength to be assumed shall be characteristic values in the determination of elastic properties of members irrespective of the limit state being considered. Redistribution of the calculated moments may be made as given in **36.1.1**.

36.1.1 *Redistribution of Moments in Continuous Beams and Frames*

The redistribution of moments may be carried out satisfying the following conditions:

a) Equilibrium between the internal forces and the external loads is maintained.

b) The ultimate moment of resistance provided at any section of a member is not less than 70 percent of the moment at the section obtained from an elastic maximum moment diagram covering all appropriate combinations of loads.

c) The elastic moment at any section in a member due to a particular combination of loads shall not be reduced by more than 30 percent of the numerically largest moment given anywhere by the elastic maximum moments diagram for the particular member, covering all appropriate combination of loads.

d) At sections where the moment capacity after redistribution is less than that from the elastic maximum moment diagram, the following relationship shall be satisfied:

$$\frac{x_u}{d} + \frac{\delta M}{100} \leq 0.6$$

where

x_u = Depth of neutral axis,

d = Effective depth, and

δM = Percentage reduction in moment.

e) In structures in which the structural frame provides the lateral stability, the reduction in moment allowed by condition given in **36.1.1** (c) shall be restricted to 10 percent for structures over 4 storeys in height.

36.1.2 *Analysis of Slabs Spanning in Two Directions at Right Angles*

Yield line theory or any other acceptable method may be used. Alternatively the provisions given in Annex D may be followed.

37 LIMIT STATE OF COLLAPSE: FLEXURE

37.1 Assumptions

Design for the limit state of collapse in flexure shall be based on the assumptions given below:

a) Plane sections normal to the axis remain plane after bending.

b) The maximum strain in concrete at the outermost compression fibre is taken as 0.003 5 in bending.

c) 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 result in prediction of strength in substantial agreement with the results of test. An acceptable stress strain curve is given in Fig. 21. 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 $\gamma_m = 1.5$ shall be applied in addition to this.

NOTE — For the stress-strain curve in Fig. 21 the design stress block parameters are as follows (*see* Fig. 22).

Area of stress block $= 0.36 f_{ck}.x_u$

Depth of centre of compressive force from the extreme fibre in compression $= 0.42\, x_u$

where

f_{ck} = Characteristic compressive strength of concrete, and

x_u = Depth of neutral axis.

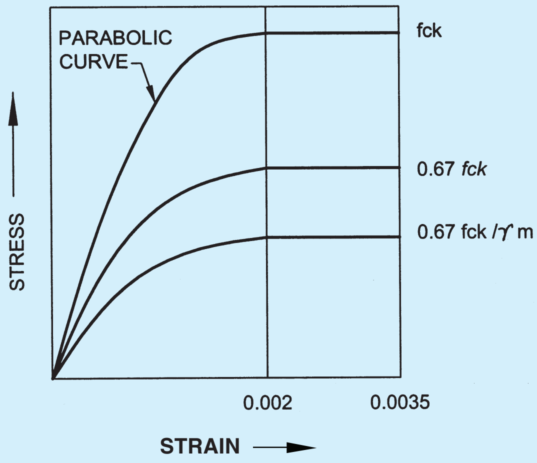


FIG. 21 STRESS-STRAIN CURVE FOR CONCRETE

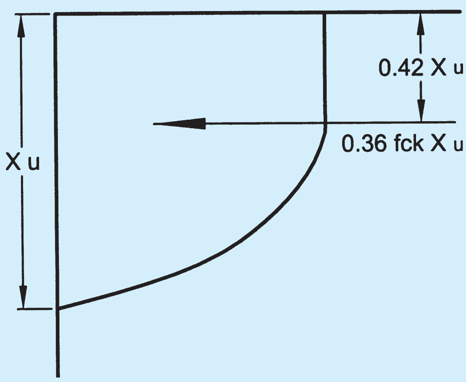


FIG. 22 STRESS BLOCK PARAMETERS

d) The tensile strength of the concrete is ignored.

e) The stresses in the reinforcement are derived from representative stress-strain curve for the type of steel used. Typical curves are given in Fig. 23. For design purposes the partial safety factor γ_m, equal to 1.15 shall be applied.

f) The maximum strain in the tension reinforcement in the section at failure shall not be less than:

$$\frac{f_y}{1.15\,E_s} + 0.002$$

where

f_y = Characteristic strength of steel, and

E_s = Modulus of elasticity of steel.

NOTE — The limiting values of depth of neutral axis for different grades of steel based on the assumptions of **37.1** are as follows:

f_y	$x_{u,\,max}/d$
250	0.53
415	0.48
500	0.46

The expression for obtaining the moments of resistance for rectangular and T-Sections, based on the assumptions of **37.1**, are given in Annex G.

38 LIMIT STATE OF COLLAPSE: COMPRESSION

38.1 Assumptions

In addition to the assumptions given in **37.1**(a) to **37.1**(e) for flexure, the following shall be assumed:

a) The maximum compressive strain in concrete in axial compression is taken as 0.002.

b) The maximum compressive strain at the highly compressed extreme fibre in concrete subjected to axial compression and bending and when there is no tension on the section shall be 0.003 5 minus 0.75 times the strain at the least compressed extreme fibre.

38.2 Minimum Eccentricity

All members in compression shall be designed for the minimum eccentricity in accordance with **24.4**. Where calculated eccentricity is larger, the minimum eccentricity should be ignored.

38.3 Short Axially Loaded Members in Compression

The member shall be designed by considering the assumptions given in **38.1** and the minimum eccentricity. When the minimum eccentricity as per **24.4** does not exceed 0.05 times the lateral dimension, the members may be designed by the following equation:

$$P_u = 0.4\, f_{ck}.A_c + 0.67 f_y.A_{sc}$$

where

P_u = Axial load on the member,

f_{ck} = Characteristic compressive strength of the concrete,

A_c = Area of concrete

f_y = Characteristic strength of the compression reinforcement, and

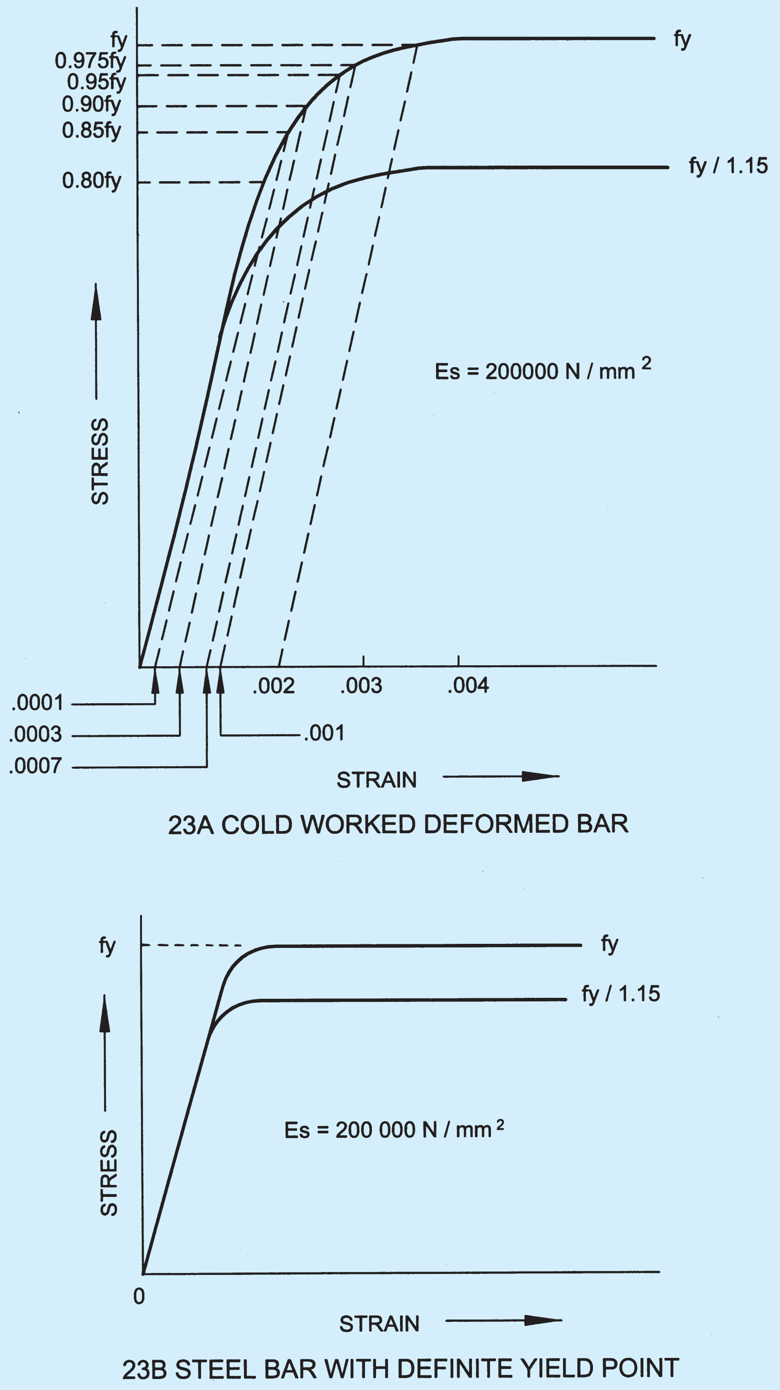


FIG. 23 REPRESENTATIVE STRESS-STRAIN CURVES FOR REINFORCEMENT

A_{sc} = Area of longitudinal reinforcement for columns.

38.4 Compression Members with Helical Reinforcement

The strength of compression members with helical reinforcement satisfying the requirement of **38.4.1** shall be taken as 1.05 times the strength of similar member with lateral ties.

38.4.1 The ratio of the volume of helical reinforcement to the volume of the core shall not be less than $0.36\ (A_g/A_c-1)\ f_{ck}/f_y$

where

A_g = Gross area of the section,

A_c = Area of the core of the helically reinforced column measured to the outside diameter of the helix,

f_{ck} = Characteristic compressive strength of the concrete, and

f_y = Characteristic strength of the helical reinforcement but not exceeding 415 N/mm^2.

38.5 Members Subjected to Combined Axial Load and Uniaxial Bending

A member subjected to axial force and uniaxial bending shall be designed on the basis of **38.1** and **38.2**.

NOTE — The design of member subject to combined axial load and uniaxial bending will involve lengthy calculation by trial and error. In order to overcome these difficulties interaction diagrams may be used. These have been prepared and published by BIS in SP 16 'Design aids for reinforced concrete to IS 456'.

38.6 Members Subjected to Combined Axial Load and Biaxial Bending

The resistance of a member subjected to axial force and biaxial bending shall be obtained on the basis of assumptions given in **38.1** and **38.2** with neutral axis so chosen as to satisfy the equilibrium of load and moments about two axes. Alternatively such members may be designed by the following equation:

$$\left[\frac{M_{ux}}{M_{ux1}}\right]^{\alpha_n} + \left[\frac{M_{uy}}{M_{uy1}}\right]^{\alpha_n} \leq 1.0$$

where

M_{ux}, M_{uy} = Moments about x and y axes due to design loads,

M_{ux1}, M_{uy1} = Maximum uniaxial moment capacity for an axial load of P_u, bending about x and y axes respectively, and α_n is related to P_u/P_{uz}

where

$$P_{uz} = 0.45 f_{ck} \cdot A_c + 0.75 f_y \cdot A_{sc}$$

For values of P_u/P_{uz} = 0.2 to 0.8, the values of α_n vary linerly from 1.0 to 2.0. For values less than 0.2 α_n is 1.0; for values greater than 0.8, α_n is 2.0.

38.7 Slender Compression Members

The design of slender compression members (*see* **24.1.1**) shall be based on the forces and the moments determined from an analysis of the structure, including the effect of deflections on moments and forces. When the effect of deflections are not taken into account in the analysis, additional moment given in **38.7.1** shall be taken into account in the appropriate direction.

38.7.1 The additional moments M_{ax} and M_{ay} shall be calculated by the following formulae:

$$M_{ax} = \frac{P_u D}{2\,000}\left[\frac{l_{ex}}{D}\right]^2$$

$$M_{ay} = \frac{P_u b}{2\,000}\left[\frac{l_{ey}}{b}\right]^2$$

where

P_u = Axial load on the member,

l_{ex} = Effective length in respect of the major axis,

l_{ey} = Effective length in respect of the minor axis,

D = Depth of the cross-section at right angles to the major axis, and

b = Width of the member.

For design of section, **38.5** or **38.6** as appropriate shall apply.

NOTES

1 A column may be considered braced in a given plane if lateral stability to the structure as a whole is provided by walls or bracing or buttressing designed to resist all lateral forces in that plane. It should otherwise be considered as unbraced.

2 In the case of a braced column without any transverse loads occurring in its height, the additional moment shall be added to an initial moment equal to sum of 0.4 M_{u1} and 0.6 M_{u2} where M_{u2} is the larger end moment and M_{u1} is the smaller end moment (assumed negative if the column is bent in double curvature). In no case shall the initial moment be less than 0.4 M_{u2} nor the total moment including the initial moment be less than M_{u2}. For unbraced columns, the additional moment shall be added to the end moments.

3 Unbraced compression members, at any given level or storey, subject to lateral load are usually constrained to deflect equally. In such cases slenderness ratio for each column may be taken as the average for all columns acting in the same direction.

38.7.1.1 The values given by equation **38.7.1** may be multiplied by the following factor:

$$k = \frac{P_{uz} - P_u}{P_{uz} - P_b} \leq 1$$

where

P_u = Axial load on compression member,

P_{uz} = As defined in **38.6**, and

P_b = Axial load corresponding to the condition of maximum compressive strain of 0.003 5 in concrete and tensile strain of 0.002 in outer most layer of tension steel.

39 LIMIT STATE OF COLLAPSE: SHEAR

39.1 Nominal Shear Stress

The nominal shear stress in beams of uniform depth shall be obtained by the following equation:

$$\tau_v = \frac{V_u}{bd}$$

where

V_u = Shear force due to design loads;

b = Breadth of the member, which for flanged section shall be taken as the breadth of the web, b_w; and

d = Effective depth.

39.1.1 *Beams of Varying Depth*

In the case of beams of varying depth the equation shall be modified as:

$$\tau_v = \frac{V_u \pm \dfrac{M_u}{d}\tan\beta}{bd}$$

where

τ_v, V_u, b and d are the same as in **39.1**,

M_u = Bending moment at the section, and

β = Angle between the top and the bottom edges of the beam.

The negative sign in the formula applies when the bending moment M_u increases numerically in the same direction as the effective depth d increases, and the positive sign when the moment deceases numerically in this direction.

39.2 Design Shear Strength of Concrete

39.2.1 The design shear strength of concrete in beams without shear reinforcement is given in Table 19.

39.2.1.1 For solid slabs, the design shear strength for concrete shall be $\tau_c k$, where k has the values given below:

Overall Depth of Slab, mm	300 or more	275	250	225	200	175	150 or less
k	1.00	1.05	1.10	1.15	1.20	1.25	1.30

NOTE — This provision shall not apply to flat slabs for which **30.6** shall apply

39.2.2 *Shear Strength of Members under Axial Compression*

For members subjected to axial compression P_u, the design shear strength of concrete, given in Table 19, shall be multiplied by the following factor:

$$\delta = 1 + \frac{3P_u}{A_g f_{ck}} \text{ but not exceeding } 1.5$$

where

P_u = Axial compressive force in Newtons,

A_g = Gross area of the concrete section in mm^2, and

f_{ck} = Characteristic compressive strength of concrete.

39.2.3 *With Shear Reinforcement*

Under no circumstances, even with shear reinforcement,

Table 19 Design Shear Strength of Concrete, τ_c, N/mm^2

(*Clauses* 39.2.1, 39.2.2, 39.3, 39.4, 39.5.3, 40.3.2, 40.3.3 *and* 40.4.3)

$100\frac{A_g}{bd}$	Concrete Code					
	M 15	M 20	M 25	M 30	M 35	M 40 and above
(1)	(2)	(3)	(4)	(5)	(6)	(7)
≤ 0.15	0.28	0.28	0.29	0.29	0.29	0.30
0.25	0.35	0.36	0.36	0.37	0.37	0.38
0.50	0.46	0.48	0.49	0.50	0.50	0.51
0.75	0.54	0.56	0.57	0.59	0.59	0.60
1.00	0.60	0.62	0.64	0.66	0.67	0.68
1.25	0.64	0.67	0.70	0.71	0.73	0.74
1.50	0.68	0.72	0.74	0.76	0.78	0.79
1.75	0.71	0.75	0.78	0.80	0.82	0.84
2.00	0.71	0.79	0.82	0.84	0.86	0.88
2.25	0.71	0.81	0.85	0.88	0.90	0.92
2.50	0.71	0.82	0.88	0.91	0.93	0.95
2.75	0.71	0.82	0.90	0.94	0.96	0.98
3.00 and above	0.71	0.82	0.92	0.96	0.99	1.01

NOTE — The term A_g is the area of longitudinal tension reinforcement which continues at least one effective depth beyond the section being considered except at support where the full area of tension reinforcement may be used provided the detailing conforms to **25.2.2** and **25.2.3**.

shall be nominal shear stress in beams τ_v exceed $\tau_{c\,max}$ given in Table 20.

39.2.3.1 For solid slabs, the nominal shear stress shall not exceed half the appropriate values given in Table 20.

Table 20 Maximum Shear Stress, $\tau_{c\,max}$, N/mm²

(*Clauses* 39.2.3, 39.2.3.1, 39.5.1 *and* 40.3.1)

Concrete Grade	M 15	M 20	M 25	M 30	M 35	M 40 and above
$\tau_{c\,max}$, N/mm²	2.5	2.8	3.1	3.5	3.7	4.0

39.3 Minimum Shear Reinforcement

When τ_v is less than τ_c given in Table 19, minimum shear reinforcement shall be provided in accordance with **25.5.1.6**.

39.4 Design of Shear Reinforcement

When τ_v exceeds τ_c given in Table 19, shear reinforcement shall be provided in any of the following forms:

a) Vertical stirrups,

b) Bent-up bars along with stirrups, and

c) Inclined stirrups

Where bent-up bars are provided, their contribution towards shear resistance shall not be more than half that of the total shear reinforcement.

Shear reinforcement shall be provided to carry a shear equal to $V_u - \tau_c bd$. The strength of shear reinforcement V_{us} shall be calculated as below:

a) For vertical stirrups:

$$V_{us} = \frac{0.87\, f_y A_{sv} d}{S_v}$$

b) For inclined stirrups or a series of bars bent-up at different cross-sections:

$$V_{us} = \frac{0.87\, f_y A_{sv} d}{S_v}(\sin\alpha + \cos\alpha)$$

c) For single bar or single group of parallel bars, all bent-up at the same cross-section:

$$V_{us} = 0.87 f_y A_{sv} \sin\alpha$$

where

A_{sv} = Total cross-sectional area of stirrup legs or bent-up bars within a distance s_v,

S_v = Spacing of the stirrups or bent-up bars along with the length of the member,

τ_v = Nominal shear stress,

τ_c = Design shear strength of the concrete,

b = Breadth of the member which for flanged beams, shall be taken as the breadth of the web b_w,

f_y = Characteristic strength of the stirrup or bent-up reinforcement which shall not be taken greater than 415 N/mm²,

α = Angle between the inclined stirrup or bent-up bar and the axis of the member, not less than 45°, and

d = Effective depth.

NOTES

1 Where more than one type of shear reinforcement is used to reinforce the same portion of the beam, the total shear resistance shall be computed as the sum of the resistance for the various types separately.

2 The area of the stirrups shall not be less than the minimum specified in **25.5.1.6**.

39.5 Enhanced Shear Strength of Sections Close to Supports

39.5.1 *General*

Shear failure at sections of beams and cantilevers without shear reinforcement will normally occur on plane inclined at an angle 30° to the horizontal. If the angle of failure plane is forced to be inclined more steeply than this [because the section considered ($X - X$) in Fig. 24 is close to a support or for other reasons], the shear force required to produce failure is increased.

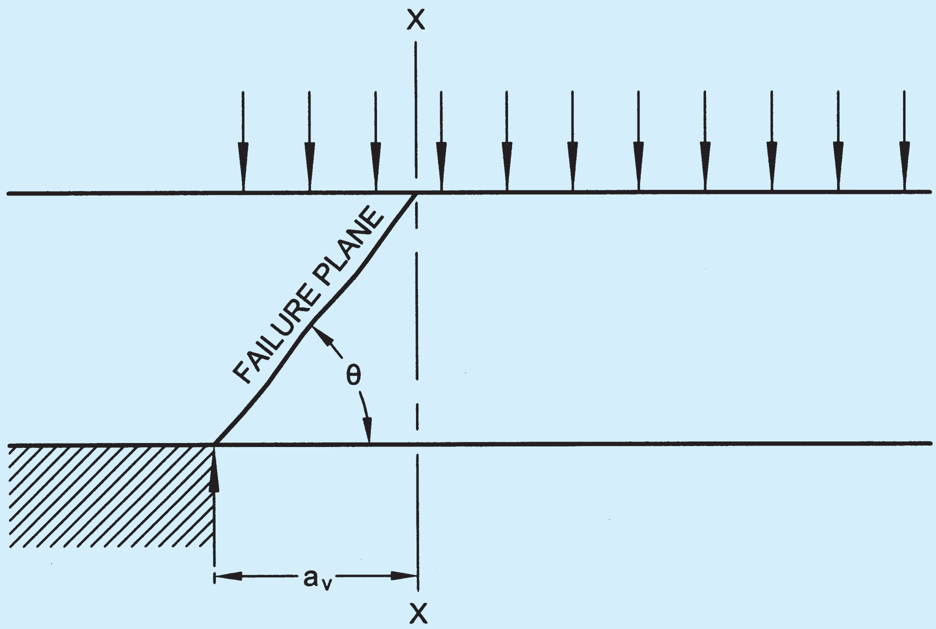
The enhancement of shear strength may be taken into account in the design of sections near a support by increasing design shear strength of concrete to $2d\,\tau_c/a_v$ provided that design shear stress at the face of the support remains less than the values given in Table 20. Account may be taken of the enhancement in any situation here the section considered is closer to the face of a support or concentrated load than twice the effective depth, d. To be effective, tension reinforcement should extend on each side of the point where it is intersected by a possible failure plane for a distance at least equal to the effective depth, or be provided with an equivalent anchorage.

39.5.2 *Shear Reinforcement for Sections Close to Supports*

If shear reinforcement is required, the total area of this is given by

$$A_s = a_v b(\tau_v - 2\,d\tau_c/a_v)/0.87 f_y \geq 0.4\, a_v b/0.87 f_y$$

This reinforcement should be provided within the middle three quarters of a_v, where a_v is less than d, horizontal shear reinforcement will be effective than vertical.



NOTE — The shear causing failure is that acting on Section $X - X$.

FIG. 24 SHEAR FAILURE NEAR SUPPORTS

39.5.3 *Enhanced Shear Strength Near Supports (Simplified Approach)*

The procedure given in **39.5.1** and **39.5.2** may be used for all beams. However for beams carrying generally uniform load or where the principal load is located farther than 2*d* from the face of support, the shear stress may be calculated at a section a distance *d* from the face of support. The value of τ_c is calculated in accordance with Table 19 and appropriate shear reinforcement is provided at sections closer to the support, no further check for shear at such sections is required.

40 LIMIT STATE OF COLLAPSE: TORSION

40.1 General

In structures, where torsion is required to maintain equilibrium, members shall be designed for torsion in accordance with **40.2** to **40.4**. However, for such indeterminate structures where torsion can be eliminated by releasing redundant restraints, no specific design for torsion is necessary, provided torsional stiffness is neglected in the calculation of internal forces. Adequate control of any torsional cracking is provided by the shear reinforcement as per **39**.

NOTE — The approach to design in this clause is as follows: Torsional reinforcement is not calculated separately from that required for bending and shear. Instead the total longitudinal reinforcement is determined for a fictitious bending moment which is a function of actual bending moment and torsion; similarly web reinforcement is determined for a fictitious shear which is a function of actual shear and torsion.

40.1.1 The design rules laid down in **40.3** and **40.4** shall apply to beams of solid rectangular cross-section. However, these clauses may also be applied to flanged beams, by substituting b_w for *b* in which case they are generally conservative; therefore specialist literature may be referred to.

40.2 Critical Section

Sections located less than a distance *d*, from the face of the support may be designed for the same torsion as computed at a distance *d*, where *d* is the effective depth.

40.3 Shear and Torsion

40.3.1 *Equivalent Shear*

Equivalent shear V_e, shall be calculated from the formula:

$$V_e = V_u + 1.6 \frac{\tau_u}{b}$$

where

V_e = Equivalent shear,
V_u = Shear,
τ_u = Torsional moment, and
b = Breadth of beam.

The equivalent nominal shear stress τ_{ve} in this case shall be calculated as given in **40.1**, except for substituting V_u by V_e. The values of τ_{ve} shall not exceed the values of $\tau_{c\,max}$ given in Table 20.

40.3.2 If the equivalent nominal shear stress τ_{ve} does not exceed τ_c given in Table 19, minimum shear reinforcement shall be provided as per **25.5.1.6**.

40.3.3 If τ_{ve} exceeds τ_c given in Table 19, both longitudinal and transverse reinforcement shall be provided in accordance with **40.4**.

40.4 Reinforcement in Members Subjected to Torsion

40.4.1 Reinforcement for torsion, when required, shall consist of longitudinal and transverse reinforcement.

40.4.2 *Longitudinal Reinforcement*

The longitudinal reinforcement shall be designed to resist an equivalent bending moment, M_{el}, given by

$$M_{el} = M_u + M_1$$

where

M_u = Bending moment at the cross-section, and

$$M_1 = T_u \frac{(1 + D/b)}{1.7}$$

where

T_u is the torsional moment, D is the overall depth of the beam and b is the breadth of the beam.

40.4.2.1 If the numerical value of M_t as defined in **40.4.2** exceeds the numerical value of the moment M_u, longitudinal reinforcement shall be provided on the flexural compression face, such that the beam can also withstand an equivalent M_{e2} given by $M_{e2} = M_t - M_u$, the moment M_{e2} being taken as acting in the opposite sense to the moment M_u.

40.4.3 *Transverse Reinforcement*

Two legged closed hoops enclosing the corner longitudinal bars shall have an area of cross-section A_{sv}, given by

$$A_{sv} = \frac{T_u s_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u s_v}{2.5 d_1 (0.87 f_y)}$$

but the total transverse reinforcement shall not be less than

$$\frac{(\tau_{ve} - \tau_c) b.s_v}{0.87 f_y}$$

where

T_u = Torsional moment,

V_u = Shear force,

s_v = Spacing of the stirrup reinforcement,

b_1 = Centre-to-centre distance between corner bars in the direction of the width,

d_1 = Centre-to-centre distance between corner bars,

b = Breadth of the member,

f_y = Characteristic strength of the stirrup reinforcement,

τ_{ve} = Equivalent shear stress as specified in **40.3.1**, and

τ_c = Shear strength of the concrete as per Table 19.

41 LIMIT STATE OF SERVICEABILITY: DEFLECTION

41.1 Flexural Members

In all normal cases, the deflection of a flexural member will not be excessive if the ratio of its span to its effective depth is not greater than appropriate ratios given in **22.2.1**. When deflections are calculated according to Annex C, they shall not exceed the permissible values given in **22.2**.

42 LIMIT STATE OF SERVICEABILITY: CRACKING

42.1 Flexural Members

In general, compliance with the spacing requirements of reinforcement given in **25.3.2** should be sufficient to control flexural cracking. If greater spacing are required, the expected crack width should be checked by formula given in Annex F.

42.2 Compression Members

Cracks due to bending in a compression member subjected to a design axial load greater than $0.2 f_{ck} A_c$, where f_{ck} is the characteristic compressive strength of concrete and A_c is the area of the gross section of the member, need not be checked. A member subjected to lesser load than $0.2 f_{ck} A_c$ may be considered as flexural member for the purpose of crack control (*see* **42.1**).

ANNEX A

(*Foreword*)

SELF COMPACTING CONCRETE

A-1 Self compacting concrete is concrete that is able to flow under its own weight and completely fill the formwork, even in the presence of dense reinforcement, without segregation, whilst maintaining homogeneity.

A-2 APPLICATION AREA

Self compacting concrete may be used in precast-applications or for concrete placed on site. It can be manufactured in a site batching plant or in a ready mix concrete plant and delivered to site by truck. It can then be placed either by pumping or pouring into horizontal or vertical structures. In designing the mix, the size and the form of the structure, the dimension and density of reinforcement and cover should be taken in consideration.

A-3 CHARACTERISTICS OF FRESH SELF COMPACTING CONCRETE

The level of fluidity of self compacting concrete is governed chiefly by the dosing and type of superplasticizer. Due to the high fluidity of self compacting concrete, the risk of segregation and blocking is very high. Preventing segregation is therefore an important feature of the control regime. The tendency to segregation can be reduced by the use of a sufficient amount of fines (< 0.125 mm), or using a Viscosity Modifying Admixture (VMA).

Features of fresh self compacting concrete

a) Slump about 600 mm
b) Sufficient amount of fines (<0.125 mm)
c) Use of Viscosity Modifying Admixture
d) Segregation resistance

ANNEX B

(*Clauses* 17.2.2, 21.3.1, 21.7, 25.2.1 *and* 31.1)

STRUCTURAL DESIGN (WORKING STRESS METHOD)

B-1 GENERAL

B-1.1 General Design Requirements

The general design requirements of Section 3 shall apply to this Annex.

B-1.2 Redistribution of Moments

Except where the simplified analysis using coefficients (*see* **21.5**) is used, the moments over the supports for any assumed arrangement of loading, including the dead load moments may each be increased or decreased by not more than 15 percent, provided that these modified moments over the supports are used for the calculation of the corresponding moments in the spans.

B-1.3 Assumptions for Design of Members

In the methods based on elastic theory, the following assumptions shall be made:

a) At any cross-section, plane sections before bending remain plain after bending.
b) All tensile stresses are taken up by reinforcement and none by concrete, except as otherwise specifically permitted.
c) The stress-strain relationship of steel and concrete, under working loads, is a straight line.
d) The modular ratio *m* has the value $\dfrac{280}{3\sigma_{cbc}}$

where σ_{cbc} is permissible compressive stress due to bending in concrete in N/mm^2 as specified in Table 21.

NOTE — The expression given for m partially takes into account long-term effects such as creep. Therefore this m is not the same as the modular ratio derived based on the value of E_c given in **5.2.3.1**.

B-2 PERMISSIBLE STRESSES

B-2.1 Permissible Stresses in Concrete

Permissible stresses for the various grades of concrete shall be taken as those given in Tables 21 and 23.

NOTE — For increase in strength with age **5.2.1** shall be applicable. The values of permissible stress shall be obtained by interpolation between the grades of concrete.

B-2.1.1 *Direct Tension*

For members in direct tension, when full tension is taken by the reinforcement alone, the tensile stress shall be not greater than the values given below:

Grade of Concrete	M 10	M 15	M 20	M 25	M 30	M 35	M 40	M 45	M 50	M 55
Tensile Stress, N/mm^2	1.2	2.0	2.8	3.2	3.6	4.0	4.4	4.8	5.2	5.6

The tensile stress shall be calculated as $\dfrac{F_t}{A_e + m\,A_{st}}$

where

F_t = Total tension on the member minus pretension in steel, if any, before concreting;

A_e = Cross-sectional area of concrete excluding any finishing material and reinforcing steel;

m = Modular ratio; and

A_{st} = Cross-sectional area of reinforcing steel in tension.

B-2.1.2 *Bond Stress for Deformed Bars*

In the case of deformed bars conforming to accepted standard [6-5A(40)], the bond stresses given in Table 21 may be increased by 60 percent.

Table 21 Permissible Stresses in Concrete

(*Clauses* B-1.3, B-2.1, B-2.1.2, B-2.3 *and* B-4.2)

Grade of Concrete	Permissible Stress in Compression		Permissible Stress in Bond (Average) for Plain Bars in Tension
	Bending σ_{cbc}	Direct σ_{cc}	τ_{bd}
(1)	(2)	(3)	(4)
M 10	3.0	2.5	—
M 15	5.0	4.0	0.6
M 20	7.0	5.0	0.8
M 25	8.5	6.0	0.9
M 30	10.0	8.0	1.0
M 35	11.5	9.0	1.1
M 40	13.0	10.0	1.2
M 45	14.5	11.0	1.3
M 50	16.0	12.0	1.4
M 55	17.5	13.0	1.5

NOTES

1 The values of permissible shear stress in concrete are given in Table 23.

2 The bond stress given in col 4 shall be increased by 25 percent for bars in compression.

B-2.2 Permissible Stresses in Steel Reinforcement

Permissible stresses in steel reinforcement shall not exceed the values specified in Table 22.

B-2.2.1 In flexural member the value of σ_{st} given in Table 22 is applicable at the centroid of the tensile reinforcement subject to the condition that when more than one layer of tensile reinforcement is provided, the stress at the centroid of the outermost layer shall not exceed by more than 10 percent the value given in Table 22.

B-2.3 Increase in Permissible Stresses

Where stresses due to wind (or earthquake) temperature and shrinkage effects are combined with those due to dead, live and impact load, the stresses specified in Tables 21, 22 and 23 may be exceeded up to a limit of $33\frac{1}{3}$ percent. Wind and seismic forces need not be considered as acting simultaneously.

B-3 PERMISSIBLE LOADS IN COMPRESSION MEMBERS

B-3.1 Pedestals and Short Columns with Lateral Ties

The axial load P permissible on a pedestal or short column reinforced with longitudinal bars and lateral ties shall not exceed that given by the following equation:

$$P = \sigma_{cc} A_c + \sigma_{sc} A_{sc}$$

where

σ_{cc} = Permissible stress in concrete in direct compression,

A_c = Cross-sectional area of concrete excluding any finishing material and reinforcing steel,

A_{sc} = Cross-sectional area of the longitudinal steel.

NOTE — The minimum eccentricity mentioned in **24.4** may be deemed to be incorporated in the above equation.

B-3.2 Short Columns with Helical Reinforcement

The permissible load for columns with helical reinforcement satisfying the requirement of **38.4.1** shall be 1.05 times the permissible load for similar member with lateral ties or rings.

B-3.3 Long Columns

The maximum permissible stress in a reinforced concrete column or part thereof having a ratio of effective column length to least lateral dimension above 12 shall not exceed that which results from the multiplication of the appropriate maximum permissible stress as specified under **B-2.1** and **B-2.2** by the coefficient C_r given by the following formula:

$$C_r = 1.25 - \frac{l_{ef}}{48\,b}$$

where

C_r = Reduction coefficient;

l_{ef} = Effective length of column; and

b = Least lateral dimension of column; for

Table 22 Permissible Stresses in Steel Reinforcement
(*Clauses* B-2.2, B-2.2.1, B-2.3 *and* B-4.2)

Sl No.	Type of Stress in Steel Reinforcement	Permissible Stresses in N/mm^2		
		Mild Steel Bars Conforming to Grade 1 of IS 432 (Part 1)	Medium Tensile Steel Conforming to IS 432 (Part 1)	High Yield Strength Deformed Bars Conforming to IS 1786 (Grade Fe 415)
(1)	(2)	(3)	(4)	(5)
i)	Tension (σ_{st} or σ_{sv})			
	a) Up to and including 20 mm	140	Half the guaranteed yield stress subject to a maximum of 190	230
	b) Over 20 mm	130		230
ii)	Compression in column bars (σ_{sc})	130	130	190
iii)	Compression in bars in a beam or slab when the compressive resistance of the concrete is taken into account	The calculated compressive stress in the surrounding concrete multiplied by 1.5 times the modular ratio or σ_{sc} whichever is lower		
iv)	Compression in bars in a beam or slab where the compressive resistance of the concrete is not taken into account:			
	a) Up to and including 20 mm	140	Half the guaranteed yield stress subject to a maximum of 190	190
	b) Over 20 mm	130		190

NOTES

1 For high yield strength deformed bars of Grade Fe 500 the permissible stress in direct tension and flexural tension shall be 0.55 f_y. The permissible stresses for shear and compression reinforcement shall be as for Grade Fe 415.

2 For welded wire fabric conforming to accepted standard [6-5A(46)], the permissible value in tension σ_{st} is 230 N/mm^2.

3 For the purpose of this Section, the yield stress of steels for which there is no clearly defined yield point should be taken to be 0.2 percent proof stress.

4 When mild steel conforming to Grade II of accepted standard [6-5A(45)] is used, the permissible stresses shall be 90 percent of the permissible stresses in col 3, or if the design details have already been worked out on the basis of mild steel conforming to Grade 1 of accepted standard [6-5A(45)]; the area of reinforcement shall be increased by 10 percent of that required for Grade 1 steel.

column with helical reinforcement, b is the diameter of the core.

For more exact calculations, the maximum permissible stresses in a reinforced concrete column or part thereof having a ratio of effective column length to least lateral radius of gyration above 40 shall not exceed those which result from the multiplication of the appropriate maximum permissible stresses specified under **B-2.1** and **B-2.2** by the coefficient C_r given by the following formula:

$$C_r = 1.25 - \frac{l_{ef}}{160\, i_{min}}$$

where i_{min} is the least radium of gyration.

B-3.4 Composite Columns

a) *Allowable load* — The allowable axial load P on a composite column consisting of structural steel or cast-iron column thoroughly encased in concrete reinforced with both longitudinal and spiral reinforcement, shall not exceed that given by the following formula:

$$P = \sigma_{cc} A_c + \sigma_{sc} A_{sc} + \sigma_{mc} A_m$$

where

σ_{cc} = Permissible stress in concrete in direct compression;

A_c = Net area of concrete section; which is equal to the gross area of the concrete section $-A_{sc} - A_m$;

A_{sc} = Cross-sectional area of longitudinal bar reinforcement;

σ_{mc} = Allowable unit stress in metal core, not to exceed 125 N/mm^2 for a steel core, or 70 N/mm^2 for a steel core, or 70 N/mm^2 for a cast iron core; and

A_m = Cross-sectional area of the steel or cast iron core.

b) *Metal core and reinforcement* — The cross-sectional area of the metal core shall not exceed 20 percent of the gross area of the column. If a hollow metal core is used, it shall be filled with concrete. The amount of longitudinal and spiral reinforcement and the requirements as to spacing of bars, details of splices and thickness of protective shall outside the spiral, shall conform to requirements of **25.5.3**. A clearance of at least

75 mm shall be maintained between the spiral and the metal core at all points, except that when the core consists of a structural steel H-column, the minimum clearance may be reduced to 50 mm.

c) *Splices and connections of metal cores* — Metal cores in composite columns shall be accurately milled at splices and positive provisions shall be made for alignment of one core above another. At the column base, provisions shall be make to transfer the load to the footing at safe unit stresses in accordance with **33**. The base of the metal section shall be designed to transfer the load from the entire composite columns to the footing, or it may be designed to transfer the load from the metal section only, provided it is placed in the pier or pedestal as to leave ample section of concrete above the base for the transfer of load from the reinforced concrete section of the column by means of bond on the vertical reinforcement and by direct compression on the concrete. Transfer of loads to the metal core shall be provided for by the use of bearing members, such as billets, brackets or other positive connections, these shall be provided at the top of the metal core and at intermediate floor levels where required. The column as a whole shall satisfy the requirements of formula given under (a) at any point; in addition to this, the reinforced concrete portion shall be designed to carry, according to **B-3.1** or **B-3.2** as the case may be, all floor loads brought into the column at levels between the metal brackets or connections. In applying the formula under **B-3.1** or **B-3.2** the gross area of column shall be taken to be the area of the concrete section outside the metal core, and the allowable load on the reinforced concrete section shall be further limited to $0.28 f_{ck}$ times gross sectional area of the column.

d) *Allowable load on metal core only* — The metal core of composite columns shall be designed to carry safely any construction or other loads to be placed upon them prior to their encasement in concrete.

B-4 MEMBERS SUBJECTED TO COMBINED AXIAL LOAD AND BENDING

B-4.1 Design Based on Uncracked Section

A member subjected to axial load and bending (due to eccentricity of load, monolithic construction, lateral forces, etc) shall be considered safe provided the following conditions are satisfied.

a) $$\frac{\sigma_{cc,\,cal}}{\sigma_{cc}} + \frac{\sigma_{cbc,\,cal}}{\sigma_{cbc}} \le 1$$

where

$\sigma_{cc,\,cal}$ = Calculated direct compressive stress in concrete,

σ_{cc} = Permissible axial compressive stress in concrete,

$\sigma_{cbc,\,cal}$ = Calculated bending compressive stress in concrete, and

σ_{cbc} = Permissible bending compressive stress in concrete.

b) The resultant tension in concrete is not greater than 35 percent and 25 percent of the resultant compression for biaxial and uniaxial bending respectively, or does not exceed three-fourths, the 7 day modulus of rupture of concrete.

NOTES

1 $\sigma_{cbc,\,cal} = \dfrac{P}{A_c + 1.5m\, A_{sc}}$ for columns with ties where P, A_c and A_{sc} defined in **B-3.1** and m is the modular ratio.

2 $\sigma_{cbc,\,cal} = \dfrac{M}{Z}$ where M equals the moment and Z equals modulus of section. In the case of sections subject to moments in two directions, the stress shall be calculated separately and added algebraically.

B-4.2 Design Based on Cracked Section

If the requirements specified in **B-4.1** are not satisfied, the stresses in concrete and steel shall be calculated by the theory of cracked section in which the tensile resistance of concrete is ignored. If the calculated stresses are within the permissible stress specified in Tables 21, 22 and 23 the section may be assumed to be safe.

NOTE — The maximum stress in concrete and steel may be found from tables and charts based on the cracked section theory or directly by determining the no-stress line which should satisfy the following requirements:

a) The direct load should be equal to the algebraic sum of the forces on concrete and steel,
b) The moment of the external loads about any reference line should be equal to the algebraic sum of the moment of the forces in concrete (ignoring the tensile force in concrete) and steel about the same line, and
c) The moment of the external loads about any other reference lines should be equal to the algebraic sum of the moment of the forces in concrete (ignoring the tensile force in concrete) and steel about the same line.

B-4.3 Members Subjected to Combined Direct Load and Flexure

Members subjected to combined direct load flexure and shall be designed by limit state method as given in **38.5** after applying appropriate load factors as given in Table 18.

B-5 SHEAR

B-5.1 Nominal Shear Stress

The nominal shear stress τ_v in beams or slabs of uniform depth shall be calculated by the following equation:

$$\tau_v = \frac{V}{bd}$$

where

V = Shear force due to design loads,

b = Breadth of the member, which for flanged sections shall be taken as the breadth of the web, and

d = Effective depth.

B-5.1.1 *Beams of Varying Depth*

In the case of beams of varying depth, the equation shall be modified as:

$$\tau_v = \frac{V \pm \frac{M \tan \beta}{d}}{bd}$$

where

τ_v, V, b and d are the same as in **B-5.1**.

M = Bending moment at the section, and

β = Angle between the top and the bottom edges of the beam.

The negative sign in the formula applies when the bending moment M increases numerically in the same direction as the effective depth d increases, and the positive sign when the moment decreases numerically in this direction.

B-5.2 Design Shear Strength of Concrete

B-5.2.1 The permissible shear stress in concrete in beams without shear reinforcement is given in Table 23.

B-5.2.1.1 For solid slabs the permissible shear stress in concrete shall be $k\tau_c$ where k has the value given below:

Overall Depth of Slab, mm	300 or more	275	250	225	200	175	150 or less
k	1.00	1.05	1.10	1.15	1.20	1.25	1.30

NOTE — This does not apply to flat slabs for which **30.6** shall apply.

B-5.2.2 *Shear Strength of Members Under Axial Compression*

For members subjected to axial compression P, the permissible shear stress in concrete ζ_c given in Table 23, shall be multiplied by the following factor:

$$\delta = 1 + \frac{5\,P}{A_g\, f_{ck}}, \text{ but not exceeding 1.5.}$$

Table 23 Permissible Shear Stress in Concrete

(*Clauses* B-2.1, B-2.3, B-4.2, B-5.2.1, B-5.2.2, B-5.3, B-5.4, B-5.5.3, B-6.3.2, B-6.3.3 *and* B-6.4.3)

$100\frac{A_s}{bd}$	Permissible Shear Stress in Concrete τ_c, N/mm^2 Grade of Concrete					
	M 15	M 20	M 25	M 30	M 35	M 40 and above
(1)	(2)	(3)	(4)	(5)	(6)	(7)
≤ 0.15	0.18	0.18	0.19	0.20	0.20	0.20
0.25	0.22	0.22	0.23	0.23	0.23	0.23
0.50	0.29	0.30	0.31	0.31	0.31	0.32
0.75	0.34	0.35	0.36	0.37	0.37	0.38
1.00	0.37	0.39	0.40	0.41	0.42	0.42
1.25	0.40	0.42	0.44	0.45	0.45	0.46
1.50	0.42	0.45	0.46	0.48	0.49	0.49
1.75	0.44	0.47	0.49	0.50	0.52	0.52
2.00	0.44	0.49	0.51	0.53	0.54	0.55
2.25	0.44	0.51	0.53	0.55	0.56	0.57
2.50	0.44	0.51	0.55	0.57	0.58	0.60
2.75	0.44	0.51	0.56	0.58	0.60	0.62
3.00 and above	0.44	0.51	0.57	0.60	0.62	0.63

NOTE — A_s is that area of longitudinal tension reinforcement which continues at least one effective depth beyond the section being considered except at supports where the full area of tension reinforcement may be used provided the detailing conforms to **25.2.2** and **25.2.3.**

where

P = Axial compressive force in N,

A_g = Gross area of the concrete section in mm^2,

f_{ck} = Characteristic compressive strength of concrete.

B-5.2.3 *With Shear Reinforcement*

When shear reinforcement is provided the nominal shear stress τ_c in beams shall not exceed $\tau_{c\,max}$ given in Table 24.

B-5.2.3.1 For slabs, τ_v shall not exceed half the value of $\tau_{c\,max}$ given in Table 24.

Table 24 Maximum Shear Stress $\tau_{c\,max}$ N/mm^2
(*Clauses* B-5.2.3, B-5.2.3.1, B-5.5.1 *and* B-6.3.1)

Concrete Grade	M 15	M 20	M 25	M 30	M 35	M 40 and above
$\tau_{c\,max}$ N/mm^2	1.6	1.8	1.9	2.2	2.3	2.5

B-5.3 Minimum Shear Reinforcement

When τ_v is less than τ_c given in Table 23, minimum shear reinforcement shall be provided in accordance with **25.5.1.6**.

B-5.4 Design of Shear Reinfocement

When τ_v exceeds τ_c given in Table 23, shear reinforcement shall be provided in any of the following forms:

a) Vertical stirrups,
b) Bent-up bars along with stirrups, and
c) Inclined stirrups.

Where bent-up bars are provided, their contribution towards shear resistance shall not be more than half that of the total shear reinforcement.

Shear reinforcement shall be provided to carry a shear equal to $V - \tau_c bd$. The strength of shear reinforcement V_s shall be calculated as below:

a) For vertical stirrups

$$V_s = \frac{\sigma_{sv} A_{sv} d}{S_v}$$

b) For inclined stirrups or a series of bars bent-up at different cross-sections:

$$V_s = \frac{\sigma_{sv} A_{sv} d}{S_v} (\sin\alpha + \cos\alpha)$$

c) For single bar or single group of parallel bars, all bent-up at the same cross-section:

$$V_s = \sigma_{sv} A_{sv} \sin\alpha$$

where

A_{sv} = Total cross-sectional area of stirrup legs or bent-up bars within a distance,

S_v = Spacing of the stirrups or bent-up bars along the length of the member,

τ_c = Design shear strength of the concrete,

b = Breadth of the member which for flanged beams, shall be taken as the breadth of the web b_w,

σ_{sv} = Permissible tensile stress in shear reinforcement which shall not be taken greater than 230 N/mm^2,

α = Angle between the inclined stirrup or bent-up bar and the axis of the member, not less than 45°, and

d = Effective depth.

NOTE — Where more than one type of shear reinforcement is used to reinforce the same portion of the beam, the total shear resistance shall be computed as the sum of the resistance for the various types separately. The area of the stirrups shall not be less than the minimum specified in **25.5.1.6**.

B-5.5 Enhanced Shear Strength of Sections Close to Supports

B-5.5.1 *General*

Shear failure at sections of beams and centilevers without shear reinforcement will normally occur on plane inclined at an angle 30° to the horizontal. If the angle of failure plane is forced to be inclined more steeply than this [because the section considered (X –X) in Fig. 24 is close to a support or for other reasons], the shear force required to produce failure is increased.

The enhancement of shear strength may be taken into account in the design of sections near a support by increasing design shear strength of concrete, τ_c to $2d\,\tau_c/a_v$ provided that the design shear stress at the face of support remains less than the values given in Table 24. Account may be taken of the enhancement in any situation where the section considered is closer to the face of a support of concentrated load than twice the effective depth, d. To be effective, tension reinforcement should extend on each side of the point where it is intersected by a possible failure plane for a distance at least equal to the effective depth, or be provided with an equivalent anchorage.

B-5.5.2 *Shear Reinforcement for Sections Close to Supports*

If shear reinforcement is required, the total area of this is given by:

$$A_s = a_v b\,(\tau_v - 2d\,\tau_c/a_v)/\sigma_{sv} \geq 0.4\, a_v b/0.87 f_y$$

This reinforcement should be provided within the middle three quarters of a_v. Where a_v is less than d, horizontal shear reinforcement will be more effective than vertical.

B-5.5.3 *Enhanced Shear Strength Near Supports (Simplified Approach)*

The procedure given in **B-5.5.1** and **B-5.5.2** may be used for all beams. However for beams carrying generally uniform load or where the principal load is located further than 2 d from the face of support, the shear stress may be calculated at a section a distance d from the face of support. The value of τ_c is calculated in accordance with Table 23 and appropriate shear reinforcement is provided at sections closer to the support, no further check for such section is required.

B-6 TORSION

B-6.1 General

In structures where torsion is required to maintain equilibrium, members shall be designed for torsion in accordance with **B-6.2** to **B-6.4**. However, for such indeterminate structures where torsion can be eliminated by releasing redundant restraints, no specific design for torsion is necessary provided torsional stiffness is neglected in the calculation of internal forces. Adequate control of any torsional, cracking is provided by the shear reinforcement as per **B-5**.

NOTE — The approach to design in this clause for torsion is as follows:

Torsional reinforcement is not calculated separately from that required for bending and shear. Instead the total longitudinal reinforcement is determined for a fictitious bending moment which is a function of actual bending moment and torsion; similarly web reinforcement is determined for a fictitious shear which is a function of actual shear and torsion.

B-6.1.1 The design rules laid down in **B-6.3** and **B-6.4** shall apply to beams of solid rectangular cross-section. However, these clauses may also be applied to flanged beams by substituting b_w for b, in which case they are generally conservative; therefore specialist literature may be referred to.

B-6.2 Critical Section

Sections located less than a distance d, from the face of the support may be designed for the same torsion as computed at a distance d, where d is the effective depth.

B-6.3 Shear and Torison

B-6.3.1 *Equivalent Shear*

Equivalent shear, V_e shall be calculated from the formula:

$$V_e = V + 1.6\frac{T}{b}$$

where

V_e = Equivalent shear,

V = Shear

T = Torsional moment, and

b = Breadth of beam.

The equivalent nominal shear stress, τ_{ve}, in this case shall be calculated as given in **B-5.1**, except for substituting V by V_e. The values of τ_{ve} shall not exceed the values of $\tau_{c\,max}$ given in Table 24.

B-6.3.2 If the equivalent nominal shear stress τ_{ve} does not exceed τ_c, given in Table 23, minimum shear reinforcement shall be provided as specified in **25.5.1.6**.

B-6.3.3 If τ_{ve} exceeds τ_c given in Table 23, both longitudinal and transverse reinforcement shall be provided in accordance with **B-6.4**.

B-6.4 Reinforcement in Members Subjected to Torsion

B-6.4.1 Reinforcement for torsion, when required, shall consist of longitudinal and transverse reinforcement.

B-6.4.2 *Longitudinal Reinforcement*

The longitudinal reinforcement shall be designed to resist an equivalent bending moment, M_{el}, given by

$$M_{el} = M + M_t$$

where

M = Bending moment at the cross-section, and

$M_t = T\dfrac{(1+D/b)}{1.7}$, where T is the torsional moment, D is the overall depth of the beam and b is the breadth of the beam.

B-6.4.2.1 If the numerical value of M_t as defined in **B-6.4.2** exceeds the numerical value of the moment M, longitudinal reinforcement shall be provided on the flexural compression face, such that the beam can also withstand an equivalent moment M_{e2} given by $M_{e2} = M_t - M$, the moment M_{e2} being taken as acting in the opposite sense to the moment M.

B-6.4.3 *Transverse Reinforcement*

Two legged closed hoops enclosing the corner longitudinal bars shall have an area of cross-section A_{sv}, given by

$A_{sv} = \dfrac{T \cdot S_v}{b_1 d_1 \delta_{sv}} + \dfrac{V \cdot S_v}{2.5 d_1 \sigma_{sv}}$, but the total transverse reinforcement shall not be less than

$$\frac{(\tau_{ve}\ \ \tau_c)\ b.s_v}{\delta_{sv}}$$

where

T = Torsional moment,

V = Shear force,

S_v = Spacing of the stirrup reinforcement,

b_1 = Centre-to-centre distance between corner bars in the direction of the depth,

d_1 = Centre-to centre distance between corner bars in the direction of the depth,

b = Breadth of the member

σ_{sv} = Permissible tensile stress in shear reinforcement

τ_{ve} = Equivalent shear stress as specified in **B-6.3.1**, and

τ_c = Shear strength of the concrete as specified in Table 23.

ANNEX C

(*Clauses* 21.3.2, 22.2.1 *and* 41.1)

CALCULATION OF DEFLECTION

C-1 TOTAL DEFLECTION

C-1.1 The total deflection shall be taken as the sum of the short-term deflection determined in accordance with **C-2** and the long-term deflection, in accordance with **C-3** and **C-4**.

C-2 SHORT-TERM DEFLECTION

C-2.1 The short-term deflection may be calculated by the usual methods for elastic deflections using the short-term modulus of elasticity of concrete, E_c and an effective moment of intertia I_{eff} given by the following equation:

$$I_{eff} = \frac{I_r}{1.2\dfrac{M_r}{M}\dfrac{z}{d}\left(1-\dfrac{x}{d}\right)\dfrac{b_w}{b}}\text{; but}$$

$$I_r \le I_{eff} \le I_{gr}$$

where

I_r = Moment of inertia of the cracked section,

M_r = Cracking moment, equal to $\dfrac{f_{cr}\ I_{gr}}{y_t}$ where

f_{cr} = Modulus of rupture of concrete,

I_{gr} = Moment of inertia of the gross section about the centroidal axis, neglecting the reinforcement, and y_t is the distance from centroidal axis of gross section, neglecting the reinforcement, to extreme fibre in tension,

M = Maximum moment under service loads

z = Lever arm,

x = Depth of neutral axis,

d = Effective depth,

b_w = Breadth of web, and

b = Breadth of compression face.

For continuous beams, deflection shall be calculated using the values of I_{gr}, I_{gr} and M_r modified by the following equation:

$$X_e = k_1\left[\frac{X_1+X_2}{2}\right]+(1-k_1)X_o$$

where

X_e = Modified value of X,

X_1, X_2 = Values of X at the supports,

X_o = Value of X at mid span,

k_1 = Coefficient given in Table 25, and

X = Value of I_{gr}, I_{gr} or M_r as appropriate.

C-3 DEFLECTION DUE TO SHRINKAGE

C-3.1 The deflection due to shrinkage a_{cs} may be computed from the following equation:

$$a_{cs} = k_3 \psi_{cs} l^2$$

where

k_3 is a constant depending upon the support conditions,

0.5 for cantilevers,

0.125 for simply supported members,

0.086 for members continuous at one end, and

0.063 for fully continuous members,

ψ_{cs} is shrinkage curvature equal to $k_4\dfrac{\varepsilon_{cs}}{D}$

where ε_{cs} is the ultimate shrinkage strain of concrete (*see* **5.2.4**),

$$k_4 = 0.72\times\frac{P_t-P_c}{\sqrt{P_t}} \le 1.0 \text{ for } 0.25 \le P_t - P_c < 1.0$$

$$= 0.65 \times \frac{P_t - P_c}{\sqrt{P_t}} \leq 1.0 \text{ for } 0.25 \leq P_t - P_c \geq 1.0$$

where

$$P_t = \frac{100\, A_{st}}{bd} \text{ and } P_c \frac{100\, A_{sc}}{bd}$$

Table 25 Values of Coefficient, k_1

(*Clause* C-2.1)

k_2	0.5 or less	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4
k_1	0	0.03	0.08	0.16	0.30	0.50	0.73	0.91	0.97	1.0

NOTE — k_2 is given by

$$k_2 = \frac{M_1 + M_2}{M_{F1} + M_{F2}}$$

where

M_1, M_2 = Support moments, and
M_{F1}, M_{F2} = Fixed end moments

and D is the total depth of the section, and l is the length of span.

C-4 DEFLECTION DUE TO CREEP

C-4.1 The creep deflection due to permanent loads $a_{cc\,(perm)}$ may be obtained from the following equation:

$$a_{cc\,(perm)} = a_{i,\,cc\,(perm)} - a_{i\,(perm)}$$

where

$a_{i,cc(perm)}$ = Initial plus creep deflection due to permanent loads obtained using an elastic analysis with an effective modulus of elasticity;

E_{ce} = $\frac{E_c}{1+\theta}$, θ being the creep coefficient; and

$a_{i\,(perm)}$ = Short-term deflection due to permanent load using E_c.

ANNEX D

(*Clauses* 23.4 *and* 36.1.2)

SLABS SPANNING IN TWO DIRECTIONS

D-1 RESTRAINED SLABS

D-1.0 When the corners of a slab are prevented from lifting, the slab may be designed as specified in **D-1.1** to **D-1.11**.

D-1.1 The maximum bending moments per unit width in a slab are given by the following equations:

$$M_x = \alpha_x w\, l_x^2$$

$$M_y = \alpha_y w\, l_y^2$$

where

α_x and α_y are coefficients given in Table 26,
w = Total design and load per unit area.
M_x, M_y = Moments on strips of unit width spanning l_x and l_y respectively, and
l_x and l_y = Lengths of the shorter span and longer span respectively.

D-1.2 Slabs are considered as divided in each direction into middle strips and edge strips as shown in Fig. 25 the middle strip being three-quarters of the width and each edge strip one-eight of the width.

D-1.3 The maximum moments calculated as in **D-1.1** apply only to the middle strips and no re-distribution shall be made.

D-1.4 Tension reinforcement provided at mid-span in the middle strip shall extend in the lower part of the slab to within 0.25 l of a continuous edge, or 0.15 l of a discontinuous edge.

D-1.5 Over the continuous edges of a middle strip, the tension reinforcement shall extend in the upper part of the slab a distance of 0.15 l from the support, and at least 50 percent shall extend a distance of 0.3 l.

D-1.6 At a discontinuous edge, negative moments may arise. They depend on the degree of fixity at the edge of the slab but, in general, tension reinforcement equal to 50 percent of that provided at mid-span extending 0.1 l into the span will be sufficient.

D-1.7 Reinforcement in edge strip, parallel to that edge, shall comply with the minimum given in Section 3 and the requirements for torsion given in **D-1.8** to **D-1.10**.

D-1.8 Torsion reinforcement shall be provided at any corner where the slab is simply supported on both edges meeting at that corner. It shall consist of top and bottom reinforcement, each with layers of bars placed parallel

Table 26 Bending Moment Coefficients for Rectangular Panels Supported on Four Sides with Provision for Torsion at Corners

(*Clauses* D-1.1 *and* 23.4.1)

Case No.	Type of Panel and Moments Considered	Short Span Coefficients α_x (Values of l_y/l_x)								Long Span Coefficients α_y for All Values of l_y/l_x
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	*Interior Panels*:									
	Negative moment at continuous edge	0.032	0.037	0.043	0.047	0.051	0.053	0.060	0.065	0.032
	Positive moment at mid-span	0.024	0.028	0.032	0.036	0.039	0.041	0.045	0.049	0.024
2	*One Short Edge Discontinuous*:									
	Negative moment at continuous edge	0.037	0.043	0.048	0.051	0.055	0.057	0.064	0.068	0.037
	Positive moment at mid-span	0.028	0.032	0.036	0.039	0.041	0.044	0.048	0.052	0.028
3	*One Long Edge Discontinuous*:									
	Negative moment at continuous edge	0.037	0.044	0.052	0.057	0.063	0.067	0.077	0.085	0.037
	Positive moment at mid-span	0.028	0.033	0.039	0.044	0.047	0.051	0.059	0.065	0.028
4	*Two-Adjacent Edges Discontinuous*:									
	Negative moment at continuous edge	0.047	0.053	0.060	0.065	0.071	0.075	0.084	0.091	0.047
	Positive moment at mid-span	0.035	0.040	0.045	0.049	0.053	0.056	0.063	0.069	0.035
5	*Two Short Edges Discontinuous*:									
	Negative moment at continuous edge	0.045	0.049	0.052	0.056	0.059	0.060	0.065	0.069	—
	Positive moment at mid-span	0.035	0.037	0.040	0.043	0.044	0.045	0.049	0.052	0.035
6	*Two Long Edges Discontinuous*:									
	Negative moment at continuous edge	—	—	—	—	—	—	—	—	0.045
	Positive moment at mid-span	0.035	0.043	0.051	0.057	0.063	0.068	0.080	0.088	0.035
7	*Three Edges Discontinuous*: (One Long Edge Continuous)									
	Negative moment at continuous edge	0.057	0.064	0.071	0.076	0.080	0.084	0.091	0.097	—
	Positive moment at mid-span	0.043	0.048	0.053	0.057	0.060	0.064	0.069	0.073	0.043
8	*Three Edges Discontinuous (One Short Edge Continuous)*:									
	Negative moment at continuous edge	—	—	—	—	—	—	—	—	0.057
	Positive moment at mid-span	0.043	0.051	0.059	0.065	0.071	0.076	0.087	0.096	0.043
9	*Four Edges Discontinuous*:									
	Positive moment at mid-span	0.056	0.064	0.072	0.079	0.085	0.089	0.100	0.107	0.056

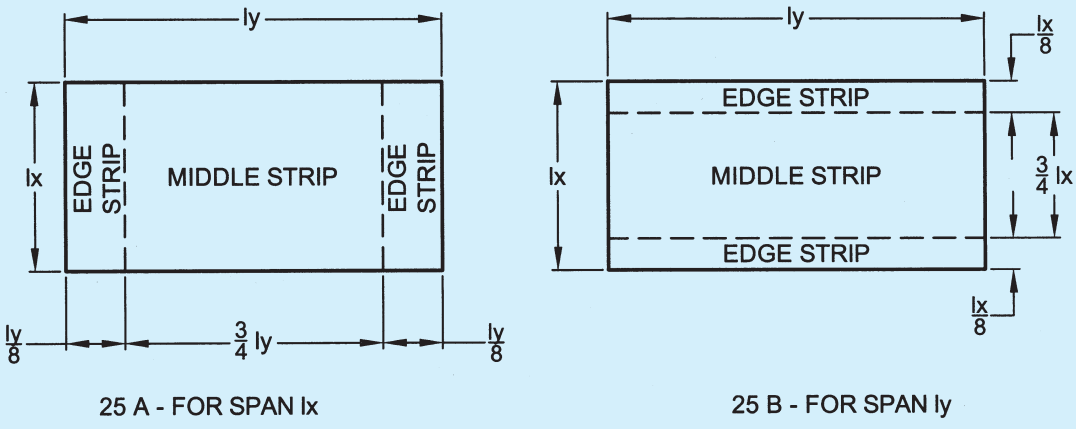


FIG. 25 DIVISION OF SLAB INTO MIDDLE AND EDGE STRIPS

to the sides of the slab and extending from the edges a minimum distance of one-fifth of the shorter span. The area of reinforcement in each of these four layers shall be three-quarters of the area required for the maximum mid-span moment in the slab.

D-1.9 Torsion reinforcement equal to half that described in **D-1.8** shall be provided at a corner contained by edges over only one of which the slab is continuous.

D-1.10 Torsion reinforcements need not be provided at any corner contained by edges over both of which the slab is continuous.

D-1.11 Where l_y/l_x is greater than 2, the slabs shall be designed as spanning one way.

D-2 SIMPLY SUPPORTED SLABS

D-2.1 When simply supported slabs do not have adequate provision to resist torsion at corners and to prevent the corners from lifting, the maximum moments per unit width are given by the following equation:

$$M_x = \alpha_x w l_x^2$$

$$M_y = \alpha_y w l_x^2$$

where

M_x, M_y, l_y, l_x are same as those in **D-1.1**, and α_x and α_y are moment coefficients given in Table 27.

D-2.1.1 At least 50 percent of the tension reinforcement provided at mid-span should extend to the supports. The remaining 50 percent should extend to within 0.1 l_x or 0.1 l_y of the support, as appropriate.

Table 27 Bending Moment Coefficients for Slabs Spanning in Two Directions at Right Angles, Simply Supported on Four Sides

(*Clause* D-2.1)

l_x/l_y	**1.0**	**1.1**	**1.2**	**1.3**	**1.4**	**1.5**	**1.75**	**2.0**	**2.5**	**3.0**
α_x	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118	0.122	0.124
α_y	0.062	0.061	0.059	0.055	0.051	0.046	0.037	0.029	0.020	0.014

ANNEX E

(*Clause* 25.2)

EFFECTIVE LEGNTH OF COLUMNS

E-1 In the absence of more exact analysis, the effective length of columns in framed structures may be obtained from the ratio of effective length to unsupported length l_{ef}/l give in Fig. 26 when relative displacement of the ends of the column is prevented and in Fig. 26 when relative lateral displacement of the ends is not prevented. In the latter case, it is recommended that the effective length ratio l_{ef}/l may not be taken to be less than 1.2.

NOTES

1 Figures 26 and 27 are reproduced from 'The Strutural Engineer' No. 7, Volume 52, July 1974 by the permission of the Council of the Institution of Structural Engineers, U.K.

2 In Fig. 26 and 27 β_1 and β_2 are equal to

$$\frac{K_c}{K_c + K_b}$$

where the summation is to be done for the members framing into a joint at top and bottom respectively; and K_c and K_b being the flexural stiffness for column and beam respectively.

E-2 To determine whether a column is a no sway or a sway column, stability index Q may be computed as given below:

$$Q = \frac{P_u \Delta_u}{H_u h_s}$$

where

P_u = Sum of axial loads on all column in the storey,

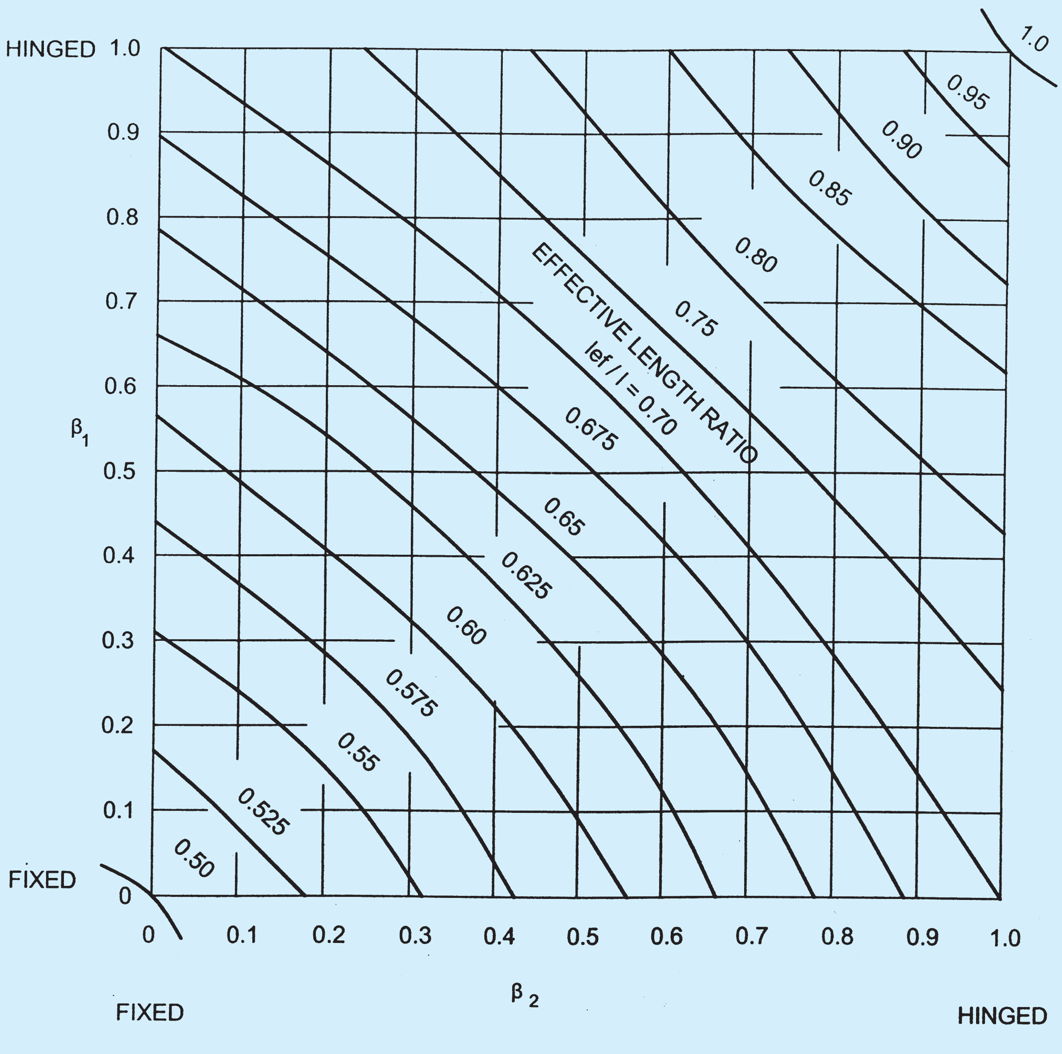


FIG. 26 EFFECTIVE LENGTH RATIOS FOR A COLUMN IN A FRAME WITH NO SWAY

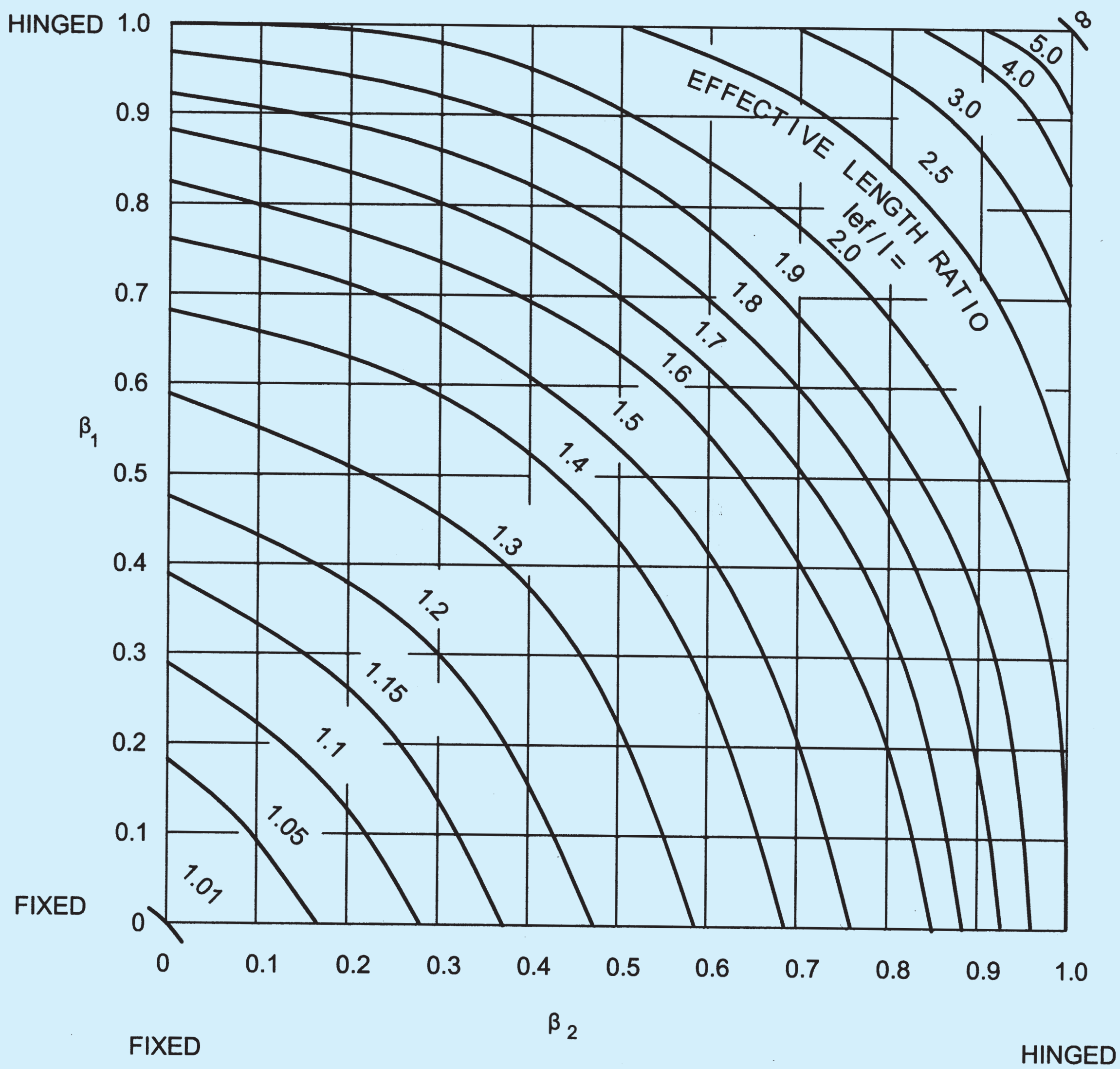


FIG. 27 EFFECTIVE LENGTH RATIOS FOR A COLUMN IN A FRAME WITHOUT RESTRAINT AGAINST SWAY

Δ_u = Elastically computed first order lateral deflection,

H_u = Total lateral force acting within the storey, and

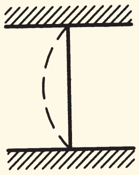
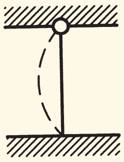
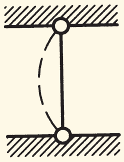
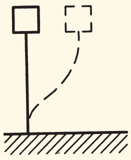
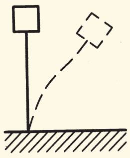
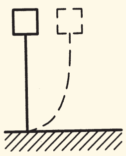
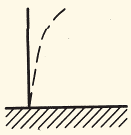
h_s = Height of the storey.

If $Q \leq 0.04$, then the column in the frame may be taken as no sway column, otherwise the column will be considered as sway column.

E-3 For normal usage assuming idealized conditions, the effective length l_{ef} of in a given plane may be assessed on the basis of Table 28.

Table 28 Effective Length of Compression Members

(*Clause* E-3)

Degree of End Restraint of Compression Members	Symbol	Theoretical Value of Effective Length	Recommended Value of Effective Length
(1)	(2)	(3)	(4)
Effectively held in position and restrained against rotation at both ends		0.50 *l*	0.65 *l*
Effectively held in position at both ends, restrained against rotation at one end		0.70 *l*	0.80 *l*
Effectively held in position at both ends, but not restrained against rotation		1.00 *l*	1.00 *l*
Effectively held in position and restrained against rotation at one end, and at the other restrained against rotation but not held in position		1.00 *l*	1.20 *l*
Effectively held in position and restrained against rotation in one end, and at the other partially restrained against rotation but not held in position		—	1.50 *l*
Effectively held in position at one end but not restrained against rotation, and at the other end restrained against rotation but not held in position		2.00 *l*	2.00 *l*
Effectively held in position and restrained against rotation at one end but not held in position nor restrained against rotation at the other end		2.00 *l*	2.00 *l*

NOTE — *l* is the unsupported length of compression member.

ANNEX F

(*Clauses* 34.3.2 *and* 42.1)

CALCULATION OF CRACK WIDTH

Provided that the strain in the tension reinforcement is limited to 0.8 F_y/E_s, the design surface crack width, which should not exceed the appropriate value given in **34.3.2** may be calculated from the following equation:

Design surface crack width

$$W_{cr} = \frac{3a_{cr}\,\varepsilon_m}{1+\dfrac{2(a_{cr}-C_{min})}{h-x}}$$

where

a_{cr} = Distance from the point considered to the surface of the nearest longitudinal bar,

C_{min} = Minimum cover to the longitudinal bar,

ε_m = Average steel strain at the level considered,

h = Overall depth of the member, and

x = Depth of the neutral axis.

The average steel strain ε_m may be calculated on the basis of the following assumption:

The concrete and the steel are both considered to be fully elastic in tension and in compression. The elastic modulus of the steel may be taken as 200 kN/mm^2 and the elastic modulus of the concrete is as derived from the equation given in **5.2.3.1** both in compression and in tension.

Alternatively, as an approximation, it will normally be satisfactory to calculate the steel stress on the basis of a cracked section and then reduce this by an amount equal to the tensile force generated by the triangular distributions, having a value of zero at the neutral axis and a value at the centroid of the tension steel of 1 N/mm^2 instantaneously, reducing to 0.55 N/mm^2 in the long-term, acting over the tension zone divided by the steel area.

These assumptions are illustrated in Fig. 28,

where

h = Overall depth of the section,

x = Depth from the compression face to the neutral axis.

f_c = Maximum compressive stress in the concrete,

f_s = Tensile stress in the reinforcement, and

E_s = Modulus of elasticity of the reinforcement.

For a rectangular tension zone, this gives

$$\varepsilon_m = \varepsilon_1 - \frac{b\,(h-x)\,(a-x)}{3\,E_s A_s (d-x)}$$

where

A_s = Area of tension reinforcement,

b = Width of the section at the centroid of the tension steel,

ε_1 = Strain at the level considered, calculated ignoring the stiffening of the concrete in the tension zone,

a = Distance from the compression face to the point at which the crack width is being calculated, and

d = Effective depth.

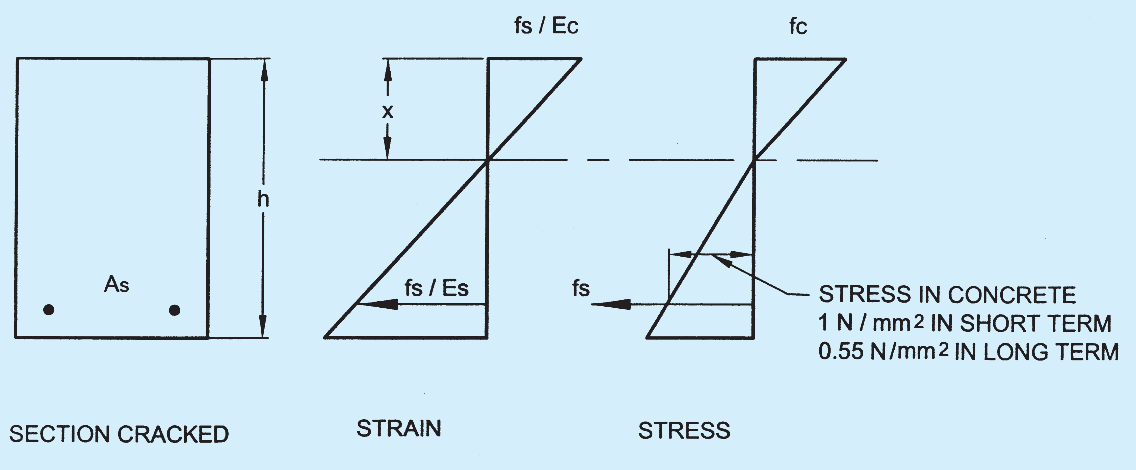


FIG. 28

ANNEX G

(*Clause* 38.1)

MOMENTS OF RESISTANCE FOR RECTANGULAR AND T-SECTIONS

G-0 The moments of resistance of rectangular and T-sections based on the assumption of **37.1** are given in this Annex.

G-1 RECTANGULAR SECTIONS

G-1.1 Sections Without Compression Reinforcement

The moment of resistance of rectangular sections without compression reinforcement should be obtained as follows:

a) Determine the depth of neutral axis from the following equation:

$$\frac{x_u}{d} = \frac{0.87\, f_y\, A_{st}}{0.36\, f_{ck}\, b.d}$$

b) If the value of x_u/d is less than the limiting value (*see* Note below **37.1**), calculate the moment of resistance by the following expression:

$$M_u = 0.87\, f_y A_{st} d \left[1 - \frac{A_{st}\, f_y}{bd\, f_{ck}}\right]$$

c) If the value of x_u/d is equal to the limiting value, the moment of resistance of the section is given by the following expression:

$$M_{u,\,lim} = 0.36\, \frac{x_{u,\,max}}{d}\left[1 - 0.42\frac{x_{u,\,max}}{d}\right] bd^2 f_{ck}$$

d) If x_u/d is greater than the limiting value, the section should be redesigned.

In the above equations,

x_u = Depth of neutral axis,
d = Effective depth,
f_y = Characteristic strength of reinforcement,
A_{st} = Area of tension reinforcement,
f_{ck} = Characteristic compressive strength of concrete,
b = Width of the compression face,
$M_{u;\,lim}$ = Limiting moment of resistance of a section without compression reinforcement, and
$x_{u,\,max}$ = Limiting value of x_u from **37.1**.

G-1.2 Section with Compression Reinforcement

Where the ultimate moment of resistance of section exceeds the limiting value, $M_{u;\,lim}$ compression reinforcement may be obtained from the following equation:

$$M_u - M_{u,\,lim} = f_{sc} A_{sc} (d - d')$$

where

$M_u, M_{u,\,lim}, d$ are same as in **G-1.1**

f_{sc} = Design stress in compression reinforcement corresponding to a strain of

$$0.003\,5\,\frac{(x_{u,\,max} - d')}{x_{u,\,max}}$$

$x_{u,\,max}$ = Limiting value of x_u from **37.1**.
A_{sc} = Area of compression reinforcement, and
d' = Depth of compression reinforcement from compression face.

The total area of tension reinforcement shall be obtained from the following equation:

$$A_{st} = A_{st1} + A_{st2}$$

where

A_{st} = Area of the total tensile reinforcement,
A_{st1} = Area of the tensile reinforcement for a singly reinforced section for $M_{u,\,lim}$, and
$A_{st2} = A_{sc} f_{sc}/0.87 f_y$.

G-2 FLANGED SECTION

G-2.1 For $x_u < D_f$, the moment of resistance may be calculated from the equation given in **G-1.1**.

G-2.1 The limiting value of the moment of resistance of the section may be obtained by the following equation when the ratio D_f/d does not exceed 0.2:

$$M_u = 0.36\,\frac{x_{u,\,max}}{D}\left[1 - 0.42\frac{x_{u,\,max}}{d}\right] f_{ck} b_w d^2$$

$$+\, 0.45\, f_{ck}\, (b_f - b_w)\, D_f \left[d - \frac{D_f}{2}\right]$$

where

M_u, $x_{u,\,max}$, d and f_{ck} are same as in **G-1.1**,
b_f = Breadth of the compression face/flange,
b_w = Breadth of the web,
D_f = Thickness of the flange.

G-2.2.1 When the ratio D_f/d exceeds 0.2, the moment of resistance of the section may be calculated by the following equation:

$$M_u = 0.36 \frac{x_{u,\,max}}{D}\left[1-0.42\frac{x_{u,\,max}}{d}\right] f_{ck} b_w d^2$$

$$+ 0.45\, f_{ck}\,(b_f - b_w)\, y_f \left[d - \frac{y_f}{2}\right]$$

where

y_f = $(0.15\, x_u + 0.65\, D_f)$, but not greater than D_f

and the other symbols are same as in **G-1.1** and **G-2.2**.

G-2.3 For $x_{u,\,max} > x_u > D_f$, the moment of resistance may be calculated by the equations given in **G-2.2** when D_f / x_u does not exceed 0.43 and **G-2.2.1** when D_f / x_u exceeds 0.43; in both cases substituting $x_{u,\,max}$ by x_u.

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 standards listed may be used by the Authority as a guide in conformance with the requirements of the referred clauses in the Code.

	IS No.	*Title*
(1)	3370	Code of practice for concrete structures for the storage of liquid:
	(Part 1) : 1965	General requirements
	(Part 2) : 1965	Reinforced concrete structures
	2210 : 1998	Criteria for design of reinforced concrete shell structures and folded plates (*first revision*)
	3201 : 1988	Criteria for design and construction of precast-trusses and purlins (*first revision*)
	4090 : 1967	Criteria for design of reinforced concrete arches
	4995	Criteria for design of reinforced concrete bins for Storage of granular and powdery materials
	(Part 1) : 1974	General requirements and bin loads
	(Part 2) : 1974	Design criteria
	4998 (Part 1) : 1992	Criteria for design of reinforced concrete chimneys: Part 1 Assessment of loads (*second revision*)
(2)	4845 : 1968	Definitions and terminology relating to hydraulic cement
	6461	Glossary of terms relating to cement:
	(Part 1) : 1972	Concrete aggregates
	(Part 2) : 1972	Materials
	(Part 3) : 1972	Concrete reinforcement
	(Part 4) : 1972	Types of concrete
	(Part 5) : 1972	Formwork for concrete
	(Part 6) : 1972	Equipment, tool and plant
	(Part 7) : 1973	Mixing, laying, compaction, curing and other construction aspect
	(Part 8) : 1973	Properties of concrete
	(Part 9) : 1973	Structural aspects
	(Part 10) : 1973	Tests and testing apparatus
	(Part 11) : 1973	Prestressed concrete
	(Part 12) : 1973	Miscellaneous
(3)	269 : 1989	Specification for ordinary Portland cement, 33 grade (*fourth revision*)
	8112 : 1989	Specification for 43 grade ordinary Portland cement (*fist revision*)
	12269 : 1987	Specification for 53 grade ordinary Portland cement
	8041 : 1990	Specification for rapid hardening Portland cement (*second revision*)
	455 : 1989	Specification for Portland slag cement (*fourth revision*)
	1489	Specification for Portland pozzolana cement:
	(Part 1) : 1991	Fly ash based (*third revision*)
	(Part 2) : 1991	Calcined clay based (*third revision*)
	8043 : 1991	Specification for hydrophobic Portland cement (*second revision*)
	12600 : 1989	Specification for low heat Portland cement
	12330 : 1988	Specification for sulphate resisting Portland cement

IS No.	*Title*
(4) 6452 : 1989	Specification for high alumina cement for structural use
6909 : 1990	Specification for supersulphated cement
(5) 3812	Specification for pulverized fuel ash:
(Part 1) : 2003	For use as pozzolana in cement, cement mortar and concrete (*second revision*)
(Part 2) : 2003	For use as admixture in cement mortar and concrete (*second revision*)
(6) 12089 : 1987	Specification for granulated slag for manufacture of Portland slag cement
(7) 383 : 1970	Specification for coarse and fine aggregates from natural sources for concrete (*second revision*)
(8) 3025	Methods of sampling and test (physical and chemical) for water and waste water:
(Part 17) : 1984	Non-filterable residue (total suspended solids) (*first revision*)
(Part 18) : 1984	Volatile and fixed residue (total filterable and non-filterable) (*first revision*)
(Part 22) : 1986	Acidity (*first revision*)
(Part 23) : 1986	Alkalinity (*first revision*)
(Part 24) : 1986	Sulphates (*first revision*)
(Part 32) : 1988	Chloride (*first revision*)
(9) 516 : 1959	Method of test for strength of concrete
(10) 4031 (Part 5) : 1988	Methods of physical tests for hydraulic cement: Part 5 Determination of initial and final setting times (*first revision*)
(11) 9103 : 1999	Specification for admixtures for concrete (*first revision*)
(12) 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*)
1786 : 1985	Specification for high strength deformed steel bars and wires for concrete reinforcement (*third revision*)
1566 : 1982	Specification for hard-drawn steel wire fabric for concrete reinforcement (*second revision*)
2062 : 1999	Steel for general structural purposes (*fifth revision*)
(13) 4082 : 1996	Recommendations on stacking and storage of construction materials and components at site (*second revision*)
(14) 516 : 1959	Method of test for strength of concrete
5816 : 1999	Method of test for splitting tensile strength of concrete (*first revision*)
(15) 1343 : 1980	Code of practice for prestressed concrete (*first revision*)
(16) 1199 : 1959	Methods of sampling and analysis of concrete
(17) 9013 : 1978	Method of making, curing and determining compressive strength of accelerated cured concrete test specimens
(18) 383 : 1970	Specification for coarse and fine aggregates from natural sources for concrete (*second revision*)
455 : 1989	Specification for Portland slag cement (*fourth revision*)
(19) 1489 (Part 1) : 1991	Specification for Portland pozzolana cement: Part 1 Fly ash based (*third revision*)
(20) 6909 : 1990	Specification for super-sulphated cement
(21) 4925 : 1968	Specification for concrete batching and mixing plant
(22) 4926 : 2003	Code of practice for ready-mixed concrete (*second revision*)
(23) 2386 (Part 3) : 1963	Methods of test for aggregates for concrete: Part 3 Specific gravity, density, voids, absorption and bulking
(24) 1791 : 1985	Specification for batch type concrete mixers (*second revision*)
12119 : 1987	General requirements for pan mixers for concrete
(25) 14687 : 1999	Guidelines for falsework for concrete structure

IS No.	Title
(26) 2502 : 1963	Code of practice for bending and fixing of bars for concrete reinforcement
(27) 2751 : 1979	Recommended practice for welding of mild steel plain and deformed bars for reinforced construction (*first revision*)
9417 : 1989	Recommendations for welding cold worked bars for reinforced concrete construction (*first revision*)
(28) 2505 : 1992	Concrete vibrators — Immersion type — General requirements
2506 : 1985	General requirements for screed board concrete vibrators (*first revision*)
2514 : 1963	Specification for concrete vibrating tables
4656 : 1968	Specification for form vibrators for concrete
(29) 11817 : 1986	Classification of joints in buildings for accommodation of dimensional deviations during construction
(30) 7861	Code of practice for extreme weather concreting:
(Part 1) : 1975	Recommended practice for hot weather concreting
(Part 2) : 1981	Recommended practice for cold weather concreting
(31) 9013 : 1978	Method of making, curing and determining compressive strength of accelerated cured concrete test specimens
(32) 13311	Methods of non-destructive testing of concrete:
(Part 1) : 1992	Ultrasonic pulse velocity
(Part 2) : 1992	Rebound hammer
(33) 875	Code of practice for design loads (other than earthquake) for buildings and structures:
(Part 1) : 1987	Dead loads — Unit weights of building material and stored materials (*second revision*)
(Part 2) : 1987	Imposed loads (*second revision*)
(Part 3) : 1987	Wind loads (*second revision*)
(Part 4) : 1987	Snow loads (*second revision*)
(Part 5) : 1987	Special loads and load combinations (*second revision*)
(34) 1893 (Part 1) : 2002	Criteria for earthquake resistant design of structures: General provisions and buildings (*fifth revision*)
(35) 1904 : 1986	Code of practice for design and construction of foundations in soils: General requirements (*third revision*)
(36) 1641 : 1988	Code of practice for fire safety of buildings (general): General principles of fire grading and classification (*first revision*)
(37) 1642 : 1989	Code of practice for fire safety of buildings (general): Details of construction (*first revision*)
(38) 13920 : 1993	Code of practice for ductile detailing of reinforced concrete structures subjected to seismic forces
(39) 4326 : 1993	Code of practice for earthquake resistant design and construction of buildings (*second revision*)
(40) 1786 : 1985	Specification for high strength deformed steel bars and wires for concrete reinforcement (*third revision*)
(41) 3414 : 1968	Code of practice for design and installation of joints in buildings
(42) 3951 (Part 1) : 1975	Specification for hollow clay tiles for floors and roofs: Part 1 Filler type (*first revision*)
(43) 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*)
(44) 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*)
(45) 1566 : 1982	Specification for hard-drawn steel wire fabric for concrete reinforcement (*second revision*)

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 5 Concrete: 5B Prestressed Concrete

BUREAU OF INDIAN STANDARDS

CONTENTS

National Building Code Sectional Committee, CED 46

FOREWORD

This sub-section covers the structural design aspects of prestressed concrete.

This sub-section is largely based on IS 1343 : 1980 'Code of practice for prestressed concrete (*first revision*)', which is under revision at the time of publication of this Code. Major changes have been envisaged in the revision of IS 1343. In the absence of availability of finalized version of revised IS 1343, at the time of revision of this Code, the provision of design as per existing IS 1343 : 1980 have been continued through appropriate reference to the same.

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 5 Concrete: 5B Prestressed Concrete

1 SCOPE

This sub-section deals with the general structural use of prestressed concrete. It covers both work carried out on site and the manufacture of precast prestressed concrete units.

2 STRUCTURAL DESIGN USING PRESTRESSED CONCRETE

The provisions relating to design and general structural use of prestressed concrete including on:

a) materials, workmanship, inspection and testing;
b) general design requirements; and
c) structural design: limit state method,

shall be in accordance with good practices contained in IS 1343 : 1980 'Code of practice for prestressed concrete (*first revision*)'.

NOTE— At the time of publication of this sub-section, IS 1343 was under revision; and once the revised IS 1343 is published, the same shall replace the provisions given in this sub-section.

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 6 Steel

BUREAU OF INDIAN STANDARDS

CONTENTS

National Building Code Sectional Committee, CED 46

FOREWORD

This Section covers the structural design aspect of steel structures in buildings.

This Section covers the use of hot-rolled structural steel sections and steel tubes in buildings. It permits the design by working stress method and plastic theory, and now in this revision by limit state method. Further, reference to space frame has now found place in this Section.

This Section is based on IS 800 : 1984 'Code of practice for general construction in steel (*second revision*)' and IS 806 : 1968 'Code of practice for use of steel tubes in general building construction (*first revision*)', and also enables design using limit state method.

More rigorous analytical procedures than envisaged as per this Section are available and can be made use of for finding effective lengths of compression members in determining elastic critical loads.

The Indian Standard IS 800, on which this Section is largely based is under revision at the time of publication of this Code. Major changes have been envisaged in the revision of IS 800 including introduction of limit state method. In the absence of availability of finalized version of revised IS 800 at the time of revision of this Section, in this revision, the provisions of design as per existing IS 800 : 1984 have been continued through appropriate reference to the same; similarly reference has been made to IS 806. At the same time, the limit state method having already gained acceptance has been taken into account in this revision by providing suitable enabling provisions and also covering certain general principles thereof.

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.

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 6 Steel

1 SCOPE

1.1 This Section covers the use of structural steel in general building construction including the use of hot rolled steel sections and steel tubes.

1.2 The provisions of this Section are generally applicable to rivetted, bolted and welded construction.

2 TERMINOLOGY

For the purpose of this Section, the definitions as given in accepted standard [6-6(1)] shall apply.

3 PLANS AND DRAWINGS

3.1 Plans, drawings and stress sheets shall be prepared according to good practice [6-6(2)].

3.2 Plans

The plans (design drawings) shall show the complete design with 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 considerations, the same shall also be indicated in the drawing.

3.3 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, 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 accordance with good practice [6-6(2)]. 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.

4 MATERIALS

All materials used in structural steel construction shall conform to Part 5 'Building Materials'. Structural steel, rivets, welding consumables, steel castings, bolts and nuts, washers and steel tubes shall be in accordance with accepted standards [6-6(3)] and other relevant Indian Standards.

5 DESIGN AND CONSTRUCTION IN STEEL

The design and construction in steel including general design requirements, design of tension members, design of compression members, design of members subjected to bending, design of members subjected to combined stresses, design of connections, plastic design, design of encased members, fabrication and erection, and the steel work tenders and contracts, shall be done in accordance with good practice [6-6(1)] and the design and construction involving use of steel tubes shall be in accordance with good practice [6-6(4)].

6 DESIGN USING LIMIT STATE METHOD

6.1 General

6.1.1 The design in steel may be done using the limit state method, which is generally based on the following basic aspects/principles:

a) It makes use of the plastic range of material for the design of structural members and incorporates load factors to take into account the variability of loading configurations.
b) It considers the good performance of steel in tension compared to compression and specifies variable factors. It takes into account this variance by defining limit states which address strength and serviceability.
c) According to this method, a structure or part of it, is considered unfit for use when it exceeds the limit state beyond which it infringes any one of the criteria governing its performance or use.
d) The two limit states are classified as the Ultimate Limit State and Serviceability Limit State. The limit states take care of the safe operation and adequacy of the structure from strength point of view. The criteria which are used to define the ultimate limit state are yielding, plastic strength, fatigue, buckling, etc. Serviceability limit state takes care of the performance and behaviour of the structure during its service period. Deflection, vibration, drift, etc are considered as serviceability criteria.

e) Limit state method considers the critical local buckling stress of the constituent plate element of a member. This method has provisions to enhance resistance of plate elements against local buckling by suitably reducing the slenderness ratio. Hence, it is possible to develop the full flexural moment capacity of a member subjected to flexure or the Limit State in flexure for a beam. In Limit State Method, based on slenderness ratio of the constituent plate elements, a member can be classified as Slender, Semi-compact, Compact and Plastic. This section classification becomes essential as the moment or load capacities of each of these sections take different values depending upon these classifications.

6.1.2 In this method, the factored loads, in different combinations, are applied to the structure to determine the load effects. The latter are then compared with the design strength of the elements.

This is expressed mathematically as:

The effects of

$$\gamma_L \cdot Q_k \leq \left[\frac{1}{\gamma_m}\right]$$

[Function of σ_y and other geometric variables]

where

γ_m = $\gamma_f \cdot \gamma_{m1} \cdot \gamma_{m2}$

= Partial safety factor for material.

γ_L = Partial factor for loads.

γ_f Factor that takes into account the inaccuracies in assessment of loads, stress distribution and construction tolerances.

γ_{m1}, γ_{m2} = Factors that take into account, uncertainties in material strength and quality, and manufacturing tolerances respectively.

Q_k = Specified nominal load induced based on design stipulations.

σ_y = Yield strength of the material.

6.2 The detailed design procedure shall be as agreed between the parties concerned.

NOTE — At the time of publication of this Section IS 800 was under revision and once the revised IS 800 is published, the same shall replace the provisions given in this section.

7 SPACE FRAME

For analysis and design of space frame along with its components, specialist literature may be referred to, and the methodology for the same may be as agreed between the parties concerned.

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 by the Authority as a guide in 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.

	IS No.	*Title*
(1)	800 : 1984	Code of practice for general construction in steel (*second revision*)
(2)	962 : 1989	Code of practice for architectural and building drawings (*second revision*)
	8000	Geometrical tolerancing on technical drawings:
	(Part 1) : 1985	Tolerances of form, orientation, location and run-out, and appropriate geometrical definitions (*first revision*)
	(Part 2) : 1992	Maximum material principles (*first revision*)
	(Part 3) : 1992	Dimensioning and tolerancing of profiles (*second 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
(3)	*Structural Steel*	
	1977 : 1996	Specification for low tensile structural steels (*third revision*)
	2062 : 1999	Specification for steel for general structural purpose (*fifth revision*)

IS No.	Title
8500 : 1991	Specification for structural steel-microalloyed (medium and high tensile qualities) (*first revision*)
1161 : 1998	Specification for steel tubes for structural purposes (*third revision*)
Rivets	
1929 : 1982	Specification for hot forged steel rivets for hot closing (12 to 36 mm diameter) (*first revision*)
2155 : 1982	Specification for cold forged solid steel rivets for hot closing (6 to 16 mm diameter) (*first revision*)
1148 : 1982	Specification for hot-rolled rivet bars (up to 40 mm diameter) for structural purposes (*third revision*)
1149 : 1982	Specification for high tensile steel rivet bars for structural purposes (*third revision*)
Welding Consumables	
1278 : 1972	Specification for filler rods and wires for gas welding (*second revision*)
814 : 1991	Specification for covered electrodes for manual metal arc welding of carbon and carbon manganese steels (*fifth revision*)
1395 : 1982	Specification for low and medium alloy steel covered electrodes for manual metal arc welding (*third revision*)
7280 : 1974	Specification for bare wire electrodes for submerged arc welding of structural steels
3613 : 1974	Specification for acceptance tests for wire-flux combinations for submerged arc welding (*first revision*)
6419 : 1996	Specification for welding rods and bare electrodes for gas shielded arc welding of structural steel (*first revision*)
6560 : 1996	Specification for molybdenum and chromium-molybdenum low alloy steel welding rods and bare electrodes for gas shielded arc welding (*first revision*)

IS No.	Title
Steel Castings	
1030 : 1998	Specification for carbon steel castings for general engineering purposes (*fifth revision*)
2708 : 1993	Specification for 1.5 percent manganese steel castings for general engineering purposes (*third revision*)
Bolts and Nuts	
1363	Specification for hexagon head bolts, screws and nuts of product grade C:
(Part 1) : 2002	Hexagon head bolts (size range M5 to M64) (*fourth revision*)
(Part 2) : 2002	Hexagon head screws (size range M5 to M64) (*fourth revision*)
(Part 3) : 2002	Hexagon nuts (size range M5 to M64) (*fourth revision*)
IS 1364	Specification for hexagon head bolts, screws and nuts of product grade A and B:
(Part 1) : 1992	Hexagon head bolts (size range M1.6 to M64) (*fourth revision*)
(Part 2) : 2002	Hexagon head screws (size range M1.6 to M64) (*fourth revision*)
IS 1367	Technical supply conditions for threaded steel fasteners:
(Part 1) : 2002	General requirements for bolts, screws and studs (*third revision*)
(Part 2) : 2002	Tolerances for fastners — Bolts, screws, studs and nuts — Product grades A, B and C (*third revision*)
IS 3640 : 1982	Specification for hexagon fit bolts (*first revision*)
IS 3757 : 1985	Specification for high strength structural bolts (*second revision*)
IS 6623 : 1985	Specification for high strength structural nuts (*first revision*)
IS 6639 : 1972	Specification for hexagon bolts for steel structures
Washers	
IS 5369 : 1975	General requirements for plain washers and lock washers (*first revision*)

IS No.	*Title*
5370 : 1969	Specification for plain washers with outside diameter more than three times the diameter
5372 : 1975	Specification for taper washers for channels (ISMC) (*first revision*)
5374 : 1975	Specification for taper washers for I-beams (ISMB) (*first revision*)
5624 : 1993	Specification for foundation bolts (*first revision*)
6610 : 1972	Specification for heavy washers for steel structures
6649 : 1985	Specification for hardened and tempered washers for high strength structural bolts and nuts (*first revision*)
(4) 806 : 1968	Code of use of steel tubes in general building construction

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 7 Prefabrication, Systems Building and Mixed/Composite Construction: 7A Prefabricated Concrete

BUREAU OF INDIAN STANDARDS

CONTENTS

National Building Code Sectional Committee, CED 46

FOREWORD

Prefabrication, though desirable for large scale building activities, has yet to take a firm hold in the country. Two aspects of prefabrication specifically to be borne in mind are the system to be adopted for the different categories of buildings and the sizes of their components. Here the principle of modular co-ordination is of 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. A few recommendations on the need to avoid 'progressive collapse' of the structure have been included. This has become necessary in view of such collapses in the past. A specific point to be borne in mind, therefore, is the need to make the structure reasonably safe against such a collapse.

Prefabricated constructions being comparatively a new technique, 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, improvement in quality and speed in 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 those materials which, due to their innate characteristics like lightweight, easy workability, thermal insulation, non-combustibility, etc, effect economy and improved quality, may be tried. However, this Section pertains to prefabricated elements with cementatious materials.

It is possible to achieve or evolve aesthetically satisfying designs using prefabricated construction. A careful and judicious handling of materials and use of finishes on a prefabricated building can help the designer a great deal in ensuring that the appearance of the building as aesthetically appealing. The purpose of finishes and architectural treatment is not only to give prefabricated buildings an individual character but also to effect better performance and greater user satisfaction.

The design of prefabricated buildings shall include provision for all installations of services and their required piping, wiring and accessories to be installed in the building.

This Section was first published in 1970 and was subsequently revised in 1983. In the last revision the following main changes were made:

a) A brief provision regarding importance of architectural treatment and finishes as applicable to prefabricated buildings were included;
b) A brief clause was added on the requirements of materials for use in prefabrication;
c) The clause on prefabricating systems and structural elements was elaborated;
d) The clause on testing of components was revised to include testing of structure or part of structure; and
e) A brief clause on the manufacture of cellular concrete was added.

In this revision, this Section, earlier named as Prefabrication and Systems Building has been named and restructured as follows:

Section 7 Prefabrication, Systems Building and Mixed/Composite Construction
7A Prefabricated Concrete
7B Systems Building and Mixed/Composite Construction

This sub-section covers Prefabricated concrete. In this revision the following main changes have been made:

a) Modular coordination and modular dimension of the components have been revised to have more flexibility for planning.
b) The provisions on tolerance has been revised to include different types of prefabricated components.

c) A detailed clause on design requirements for safety of prefabricated buildings against progressive clause has been included.
d) A clause on sampling procedure has been added for testing of components.
e) List of Indian Standards referred as good practice has been updated, specially in view of formulation of a large number of new Indian Standards on partially prefabricated components.

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

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 7 Prefabrication, Systems Building and Mixed/Composite Construction: 7A Prefabricated Concrete

1 SCOPE

This sub-section gives recommendations regarding modular planning, component sizes, prefabrication systems, design considerations, joints and manufacture, storage, transport and erection of prefabricated concrete elements for use in buildings and such related requirements for prefabricated concrete.

2 TERMINOLOGY

2.1 For the purpose of this sub-section, the following definitions shall apply.

2.1.1 *Authority Having Jurisdiction* — The Authority which has been created by a statute and which, for the purpose of administering the Code/Part, may authorize a committee or an official or an agency to act on its behalf; hereinafter called the 'Authority'.

2.1.2 *Basic Module* — The fundamental module used in modular co-ordination, 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 100 mm for the maximum flexibility and convenience. The symbol for the basic module is *M*.

2.1.3 *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 steam cured under high pressure in autoclaves.

2.1.4 *Components* — A building product formed as a distinct unit having specified sizes in three dimensions.

2.1.5 *Composite Members* — Structural members comprising 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.

2.1.6 *Increments* — Difference between two homologous dimensions of components of successive sizes.

2.1.7 *Light Weight Concrete* — Concrete of substantially lower unit weight than that made from gravel or crushed stone.

2.1.8 *Module* — A unit of size used in dimensional co-ordination.

2.1.9 *Modular Co-ordination* — Dimensional co-ordination employing the basic module or a multi-module.

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.

2.1.10 *Modular Grid* — A rectangular coordinate reference system in which the distance between consecutive lines is the basic module or a multi-module. This multi-module may differ for each of the two dimensions of the grid.

2.1.11 *Multi-module* — A module whose size is a selected multiple of the basic module.

2.1.12 *Prefabricate* — To fabricate components or assembled units prior to erection or installation in a building.

2.1.13 *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*.

2.1.14 *Sandwich Concrete Panels* — Panels made by sandwiching an insulation material between two layers of reinforced concrete to act as insulation for concrete panels.

2.1.15 *Self Compacting Concrete* — Concrete that is able to flow 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.

2.1.16 *Shear Connectors* — Structural elements, such as anchors, studs, channels and spirals, intended to transmit the horizontal shear between the prefabricated member and the cast *in-situ* concrete and also to prevent vertical separation at the interface.

2.1.17 *System* — It is a particular method of construction of buildings with certain order and discipline using the prefabricated components, tunnel form or large panel shutters which are inter-related in functions and are produced based on a set of instructions.

2.1.18 *Unit* — Building material formed as a simple article with all three dimensions specified, complete in itself but intended to be part of a compound unit or complete building. Examples are brick, block, tile, etc.

3 MATERIALS, PLANS AND SPECIFICATIONS

3.1 Materials

All materials shall conform to Part 5 'Building Materials'.

3.1.1 While selecting the materials for prefabrication, the following characteristics shall be considered:

a) Easy availability;
b) Light weight for easy handling and transport;
c) Thermal insulation property;
d) Easy workability;
e) Durability;
f) Non-combustibility;
g) Sound insulation;
h) Economy; and
j) Any other special requirement in a particular application.

3.2 Plans and Specifications

The detailed plans and specifications shall cover the following:

a) Such drawings shall describe the elements and the structure and assembly including all required data of physical properties of component materials. Material specification, age of concrete for demoulding, casting/erection tolerance and type of curing to be followed.
b) Details of connecting joints of prefabricates shall be given to an enlarged scale.
c) Site or shop location of services, such as installation of piping, wiring or other accessories integral with the total scheme shall be shown separately.
d) Data sheet indicating the location of the inserts and acceptable tolerances for supporting the prefabricate during erection, location and position of doors/windows/ventilators, etc, if any.
e) The drawings shall also clearly indicate location of handling arrangements for lifting and handling the prefabricated elements. Sequence of erection with critical check points and measures to avoid stability failure during construction stage of the building.

4 MODULAR CO-ORDINATION, ARCHITECTURAL TREATMENT AND FINISHES

4.1 Modular Co-ordination

The basic module is to be adopted. After adopting this, further work is necessary to outline suitable range of multi-modules with greater increments, often referred to as preferred increments. A set of rules as detailed below would be adequate for meeting the requirements of conventional and prefabricated construction.

These rules relate to the following basic elements:

a) The planning grid in both directions of the horizontal plan shall be:
   1) 15 *M* for industrial buildings,
   2) 3 *M* for other buildings.

   The centre lines of load bearing walls should preferably coincide with the gridlines.
b) The planning module in the vertical direction shall be 2 *M* for industrial buildings and 1 *M* for other buildings.
c) Preferred increments for sill heights, doors, windows and other fenestration shall be 1 *M*.
d) In the case of internal columns, the grid lines shall coincide with the centre lines of columns. In case of external columns and columns near the lift and stair wells, the grid lines shall coincide with centre lines of the column in the topmost storey.

4.2 Architectural Treatment and Finishes

Treatment and finishes have to be specified keeping in view the requirements of protection, function and aesthetics of internal and external spaces and surfaces.

While deciding the type of architectural treatment and finishes for prefabricated buildings, the following points should be kept in view:

a) Suitability for mass production techniques;
b) Recognition of the constraints imposed by the level of workmanship available;
c) Possibility of using different types of finishes;
d) Use of finishes and architectural treatment for the creation of a particular architectural character in individual buildings and in groups of buildings by the use of colour, texture, projections and recesses on surfaces, etc;
e) Incorporation of structural elements like joists, columns, beams, etc, as architectural features and the treatment of these for better overall performance and appearance;
f) Satisfactory finishing of surfaces; and
g) Use of light weight materials to effect economy in the structural system.

Some of the acceptable methods of finishes integral with the precasting are:

a) Concrete surface moulded to design; shape;

b) Laid-on finishing tiles fixed during casting;

c) Finishes obtained by washing, tooling, grinding, grooving of hardened concrete;

d) Exposed aggregates; and

e) Other integral finishes.

5 COMPONENTS

5.1 The preferred dimensions of precast elements shall be as follows:

a) *Flooring and Roofing Scheme* — Precast slabs or other precast structural flooring units:

1) *Length* — Nominal length shall be in multiples of 1 *M*;
2) *Width* — Nominal width shall be in multiples of 0.5 *M*; and
3) *Overall Thickness* — Overall thickness shall be in multiples of 0.1 *M*.

b) *Beams*

1) *Length* — Nominal length shall be in multiples of 1 *M*;
2) *Width* — Nominal width shall be in multiples of 0.1 *M*; and
3) *Overall Depth* — Overall depth of the floor zone shall be in multiples of 0.1 *M*.

c) *Columns*

1) *Height* — Height of columns for industrial and other building 1 *M*; and
2) *Lateral Dimensions* — Overall lateral dimension or diameter of columns shall be in multiples of 0.1 *M*.

d) *Walls*

Thickness — The nominal thickness of walls shall be in multiples of 0.1 *M*.

e) *Staircase*

Width — Nominal width shall be in multiples of 1 *M*.

f) *Lintels*

1) *Length* — Nominal length shall be in multiples of 1 *M*;
2) *Width* — Nominal width shall be in multiples of 0.1 *M*; and
3) *Depth* — Nominal depth shall be in multiples of 0.1 *M*.

g) *Sunshades/Chajja Projections*

1) *Length* — Nominal length shall be in multiples of 1 *M*.
2) *Projection* — Nominal length shall be in multiples of 0.5 *M*.

5.2 Casting Tolerances of Precast Components

Sl No.	*Product Tolerances*	*Product (see Note)*
(1)	(2)	(3)
i)	*Length*	
	± 5 mm	1, 7
	± 5 mm or ± 0.1 percent whichever is greater	2, 3, 8
	± 0.1 percent subject to maximum of $^{+5}_{-10}$ mm	4
	± 2 mm for length below and up to 500 mm	5
	± 5 mm for length over 500 mm	5
	± 10 mm	6, 9,10
ii)	*Thickness/Cross-sectional dimensions*	
	± 3 mm	1
	± 3 mm or 0.1 percent whichever is greater	2, 8
	± 2 mm up to 300 mm wide ± 3 mm greater than 300 mm wide	4
	± 2 mm	3, 7
	± 4 mm	6, 9, 10
iii)	*Straightness/Bow*	
	± 5 mm or 1/750 of length whichever is greater	2, 4, 8
	± 3 mm	1, 5
	± 2 mm	7
iv)	*Squareness*	
	When considering the squareness of the corner, the longer of two adjacent sides being checked shall be taken as the base line.	
	The shorter side shall not vary in length from the perpendicular by more than 5 mm	2, 5, 8
	The shorter side shall not vary in length from the perpendicular by more than 3 mm	1, 7
	The shorter side shall not be out of square line for more than $^{+2}_{-5}$ mm	4
v)	*Twist*	
	Any corner shall not be more than the tolerance given below from the plane containing the other three corners:	
	± 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)	2, 8
	± 1/1 500 of dimension of ± 5 mm whichever is less	4

(1)	(2)	(3)
	± 3 mm	1
	± 1 mm	7
vi)	*Flatness*	
	The maximum deviation from 1.5 m straight edge placed in any position on a nominal plane surface shall not exceed	
	± 5 mm	2, 8
	± 3 mm	4
	± 2 mm	1, 7
	± 4 or maximum of 0.1 percent length	5

NOTES — Key for product reference

1 Channel unit
2 Ribbed slab unit/hollow slab
3 Waffle unit
4 Large panel prefabrication
5 Cellular concrete floor/roof slabs
6 Prefabricated brick panel
7 Precast planks
8 Ribbed/plain wall panel
9 Column
10 Step unit

6 PREFABRICATION SYSTEMS AND STRUCTURAL SCHEMES

6.1 The word 'system' is referred to a particular method of construction of buildings using the prefabricated components which are inter-related in functions and are produced to a set of instructions. With certain constraints, several plans are possible, using the same set of components. The degree of flexibility varies from system to system. However, in all the systems there is a certain order and discipline.

6.2 The following aspects, among others, are to be considered in devising a system:

a) Effective utilization of spaces;
b) Straight and simple walling scheme;
c) Limited sizes and numbers of components;
d) Limited opening in bearing walls;
e) Regulated locations of partitions;
f) Standardized service and stair units;
g) Limited sizes of doors and windows with regulated positions;
h) Structural clarity and efficiency;
j) Suitability for adoption in low rise and high rise building;
k) Ease of manufacturing, storing and transporting;
m) Speed and ease of erection; and
n) Simple jointing system.

6.3 Prefabrication Systems

The system of prefabricated 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.

6.3.1 *Types of Prefabrication Components*

The prefabricated concrete components such as those given below may be used which shall be in accordance with Part 5 'Building Materials' and the accepted standards [6-7A(1)], 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 blocks and battens,
j) Precast planks and joists for flooring and roofing,
k) Precast joists and trussed girders,
m) Light weight/cellular concrete slabs,
n) Precast lintel and chajjas,
p) Large panel prefabricates,
q) Reinforced/Prestressed concrete trusses,
r) Reinforced/Prestressed roof purlins,
s) Precast concrete L-panel unit,
t) Prefabricated brick panel unit,
u) Prefabricated sandwich concrete panel, and
v) Precast foundation.

There may be other types of components which may be used with the approval of the Authority.

NOTE — The elements may be cast at the site or off the site.

6.3.2 There are two categories of open prefab system depending on the extent of prefabrication used in the construction as given in **6.3.2.1** and **6.3.2.2**.

6.3.2.1 *Partial prefabrication system*

This system basically uses precast roofing and flooring components and other minor elements like lintels, *CHAJJAS*, kitchen sills in conventional building construction. The structural system could be in the form of *in-situ* framework or load bearing walls.

6.3.2.2 *Full prefabrication system*

In this system almost all the structural components are prefabricated. The filler walls may be of brick/block masonry or of any other locally available material.

6.3.3 *Large Panel Prefabrication System*

This system is based on the use of large prefab components. The components used are precast concrete large panels for walls, floors, roofs, balconies, staircases, etc. The casting of the components could be at the site or off the site.

Depending upon the extent of prefabrication, this system can also lend itself to partial prefab system and full prefab system.

Structural scheme with precast large panel walls can be classified as given in **6.3.3.1** to **6.3.3.3**.

6.3.3.1 *Precast Walls*

6.3.3.1.1 Based on the structural functions of the walls, the precast walls may be classified as:

a) load bearing walls,
b) non-load bearing walls, and
c) shear walls.

6.3.3.1.2 Based on construction, the precast walls may be classified as:

a) *Homogeneous walls* — which could be solid, hollow or ribbed; and
b) *Non-homogeneous walls* — these could be composite or sandwich panels.

6.3.3.1.3 Based on their locations and functional requirements the precast walls may also classified as:

a) external walls, which may be load bearing or non-load bearing depending upon the lay-out; these are usually non-homogeneous walls of sandwiched type to impart better thermal comforts; and
b) internal walls providing resistance against vertical loads, horizontal loads, fire, etc; these are normally homogeneous walls.

6.3.3.2 *Precast floors*

6.3.3.2.1 Depending upon the composition of units, precast flooring units may be homogeneous or non-homogeneous.

a) Homogeneous floors may be solid slabs, cored slabs, ribbed or waffle slabs.
b) Non-homogeneous floors may be multi-layered ones with combinations of light weight concrete or reinforced/prestressed concrete, with filler blocks.

6.3.3.2.2 Depending upon the way the loads are transferred, the precast floors may be classified as one way or two way systems:

a) One way system transfers loads to supporting members in one direction only. The precast elements which come under this category are channel slabs, hollow core slabs, channels and ties system, light weight/cellular concrete slabs, etc.
b) Two way systems transfer loads in both the directions imparting loads on the four edges. The precast elements under this category are room sized panels, two way ribbed or waffle slab systems, etc.

6.3.3.3 *Staircase systems*

Staircase system may consist of single flights with in-built risers and treads in the element. The flights are normally unidirectional transferring the loads to supporting landing slabs or load bearing walls.

6.3.4 *Box Type Construction*

In this system, room size units are prefabricated and erected at site. Toilet and kitchen blocks could also be similarly prefabricated and erected at site.

NOTE — This system derives its stability and stiffness from the box units which are formed by four adjacent walls. Walls are jointed to make rigid connections among themselves. The box unit rests on foundation which may be of conventional type or precast type.

6.4 Design Considerations

The precast structure should be analyzed as a monolithic one and the joints in them designed to take the forces of an equivalent discrete system. Resistance to horizontal loading shall be provided by having appropriate moment and shear resisting joints or placing shear walls (in diaphragm braced frame type of construction) in two directions at right angles or otherwise. No account is to be taken of rotational stiffness, if any, of the floor-wall joint in case of precast bearing wall buildings. The individual components shall be designed, taking into consideration the appropriate end conditions and loads at various stages of construction. The components of the structure shall be designed for loads in accordance with Part 6 'Structural Design, Section 1 Loads, Forces and Effects'. In addition members shall be designed for handling, erection and impact loads that might be expected during handling and erection.

6.4.1 In some conventional forms of construction, experience has shown that the structures are capable of safely sustaining abnormal conditions of loading and remaining stable after the removal of primary structural members. It has been shown that some forms of building structure and particularly some industrialized large panel systems have little reserve strength to resist forces not specifically catered for in the design. In the light of this, therefore, recommendations made in **6.4.2** to **6.4.9** should be kept in mind for ensuring stability of such structure.

6.4.2 Adequate buttressing of external wall panels is important since these elements are not fully restrained on both sides by floor panels. Adequate design precautions may be taken by the designer. Experience shows that the external wall panel connections are the weakest points of a precast panel building.

6.4.3 It is equally important to provide restraint to all load bearing elements at the corners of the building. These elements and the external ends of cross-wall units should be stiffened either by introducing columns as connecting units or by jointing them to non-structural wall units which in emergency may support the load. Jointing of these units should be done bearing in mind the need for load support in an emergency.

6.4.4 In prefabricated construction, the possibility of gas or other explosions which can remove primary structural elements leading to progressive collapse of the structure shall be taken into account. 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.

6.4.5 Provision in the design to reduce the probability of progressive collapse is essential in buildings of over six storeys and is of relatively higher priority than for buildings of lower height.

6.4.6 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 specifically requires this to be done.

6.4.7 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.

6.4.8 In all aspects of erection that affect structural design, it is essential that the designer should maintain a close liaison with the builder/contractor regarding the erection procedures to be followed.

6.4.9 Failures that have occurred during construction appear to be of two types. The first of these is the pack-of-cards type of collapse in which the absence of restraining elements, such as partitions, cladding or shear walls, means that the structure is not stable during the construction period. The second is the situation in which one element falls during erection and lands on an element below. The connections of the lower element then give way under the loading, both static and dynamic, and a chain reaction of further collapse is set up.

6.4.9.1 A precaution against the first form of failure is that the overall stability of a building shall be considered in all its erection stages as well as in its completed state. All joints that may be required to resist moments and shears during the erection stage only, shall be designed with these in mind. Temporary works required to provide stability during construction shall be designed carefully.

6.4.9.2 To guard against the second form of failure, that is, the dropping of a unit during erection, particular attention shall be given to the details of all pre-formed units and their seatings to ensure that they are sufficiently robust to withstand the maximum stresses that can arise from site conditions. Precast concrete construction generally shall be capable of withstanding the impact forces that can arise from bad workmanship on site.

6.5 Design Requirements for Safety Against Progressive Collapse

6.5.1 Prefabricated buildings shall be designed with proper structural integrity to avoid situations where damage to small areas of a structure or failure of single elements may lead to collapse of major parts of the structure.

The following precaution may generally provide adequate structural integrity:

a) All buildings should be capable of safely resisting the minimum horizontal load of 1.5 percent of characteristic dead load applied at each floor or roof level simultaneously (*see* Fig. 1).

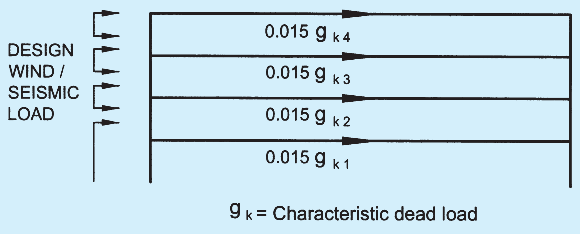


FIG. 1 HORIZONTAL LOADS

b) All buildings are provided with effective horizontal ties
   1) Around the periphery
   2) Internally (in both directions)
   3) To columns and walls

c) Vertical ties for buildings of five or more storeys.

In proportioning the ties, it may be assumed that no other forces are acting and the reinforcement is acting at its characteristic strength.

Normal procedure may be to design the structure for the usual loads and then carry out a check for the tie forces.

6.5.2 *Continuity and Anchorage of Ties*

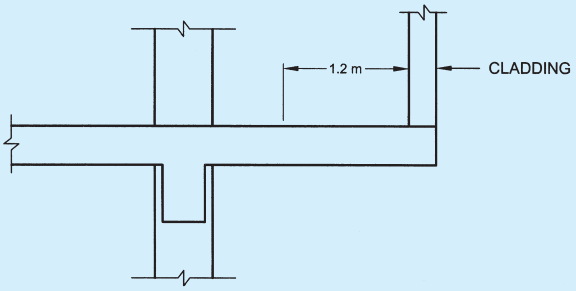
Bars shall be lapped, welded or mechanically joined as in accordance with Part 6 'Structural Design, Section 5 Plain, Reinforced and Prestressed Concrete: 5A Plain and Reinforced Concrete'.

6.5.3 *Design of Ties*

6.5.3.1 *Peripheral ties*

At each floor and roof level an effectively continuous tie should be provided within 1.2 m of the edge of the building or within the perimeter wall (*see* Fig. 2).

The tie should be capable to resisting 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)



NOTE — If there are cantilever slabs, supporting external cladding, projecting in front of the columns and these are more than 1.2 m, than the peripheral tie shall go in the slab.

FIG. 2 POSITION FOR PERIPHERAL TIE

6.5.3.2 *Internal ties*

These are to be provided at each floor and roof level in two directions approximately at right angles. Ties should 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. 3). The tensile strength, in kN/m width shall be the greater of

$$\frac{(g_k + q_k)}{7.5}\frac{l_r\ F_t}{5} \text{ and } 1.5\ F_t$$

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.

The bars providing these ties may be distributed evenly in the slabs (*see* Fig. 4) or may be grouped at or in the beams, walls or other appropriate positions but at spacings generally not greater than 1.5 l_r.

6.5.3.3 *Horizontal ties to column and wall*

All external load-bearing members such as columns and walls should be anchored or tied horizontally into the structure at each floor and roof level. The design force for the tie is to be greater of:

a) 2 F_t kN or $l_s \times F_t$/2.5 kN whichever is less for a column or for each metre length if there is a wall l_s is the floor to ceiling height in metres.
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.

6.5.3.4 *Vertical ties (for buildings of five or more storeys)*

Each column and each wall carrying vertical load should be tied continuously from the foundation to the roof level. The reinforcement provided is required only to resist a tensile force equal to the maximum design ultimate load (dead and imposed) received from any one storey.

In situation where provision of vertical ties cannot be done, the element should be considered to be removed and the surrounding members designed to bridge the gap.

6.5.4 *Key Elements*

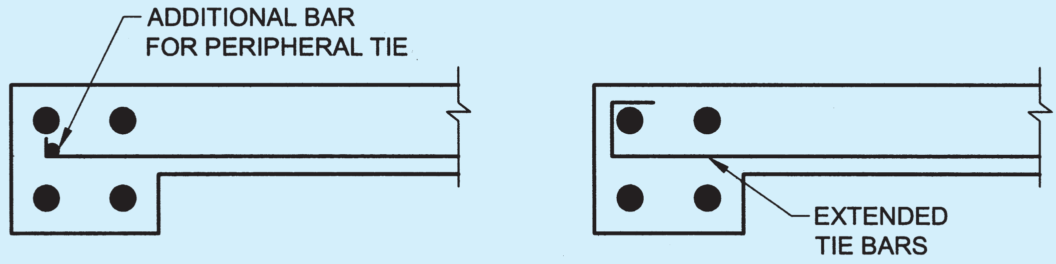
For buildings of five or more storeys, the layout should be checked to identify key elements. A key element is such that its failure would cause the collapse of more than a limited area close to it.

The limited area defined above may be taken equal to 70 m^2 or 15 percent of the area of the storey whichever is lesser.

If key elements exists, it is preferable to modify the layout so that the key element is avoided.

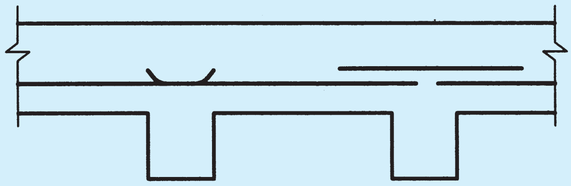
6.6 Bearing for Precast Units

Precast units shall have a bearing at least of 100 mm on masonry supports and of 75 mm at least on steel or concrete. Steel angle shelf bearings shall have a 100 mm horizontal leg to allow for a 50 mm bearing exclusive of fixing clearance. When deciding to what extent, if any, the bearing width may be reduced in special circumstances, factors, such as, loading, span, height of wall and provision of continuity, shall be taken into 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 the peripheral tie or the internal tie bars be extended and anchored around the top bar in the beam.

FIG. 3 ANCHORING OF TIES IN SLABS



NOTE — For continuity in continuous slabs, bars are distributed evenly in a floor slab by means of lapping some bottom steel at supports, either by extending existing bars or by the addition of splice bars.

FIG. 4 CONTINUITY REQUIREMENT FOR SLAB

7 JOINTS

7.1 The design of joints shall be made in the light of their assessment with respect to the following considerations:

a) *Feasibility* — The feasibility of a joint shall be determined by its load-carrying capacity in the particular situation in which the joint is to function.

b) *Practicability* — Practicability of joint shall be determined by the amount and type of material required in construction; cost of material, fabrication and erection and the time for fabrication and erection.

c) *Serviceability* — Serviceability shall be determined by the joints/expected behaviour to repeated or possible overloading and exposure to climatic or chemical conditions.

d) *Fire Rating* — The fire rating for joints of precast components shall be higher or at least equal to connecting members.

e) *Appearance* — The appearance of precast components joint shall merge with architectural aesthetic appearance and shall not be physically prominent compared to other parts of structural components.

7.2 The following are the requirements of a structural joint:

a) It shall be capable of being designed to transfer the imposed load and moments with a known margin of safety;

b) It shall occur at logical locations in the structure and at points which may be most readily analysed and easily reinforced;

c) It shall accept the loads without marked displacement or rotation and avoid high local stresses;

d) It shall accommodate tolerances in elements;

e) It shall require little temporary support, permit adjustment and demand only a few distinct operation to make;

f) It shall permit effective inspection and rectification;

g) It shall be reliable in service with other parts of the building; and

h) It shall enable the structure to absorb sufficient energy during earthquakes so as to avoid sudden failure of the structure.

7.2.1 Precast structures may have continuous or hinged connections subject to providing sufficient rigidity to withstand horizontal loading. When only compressive forces are to be taken, hinged joints may be adopted. In case of prefabricated concrete elements, load is transmitted via the concrete. When both compressive force and bending moment are to be taken, rigid or welded joints may be adopted; the shearing force is usually small in the column and can be taken up by the friction resistance of the joint. Here load transmission is accomplished by steel inserted parts together with concrete.

7.2.2 When considering thermal shrinkage and heat effects, provision of freedom of movement or introduction of restraint may be considered.

7.3 Joining techniques/materials normally employed are:

a) Welding of cleats or projecting steel,
b) Overlapping reinforcement, loops and linking steel grouted by concrete,
c) Reinforced concrete ties all round a slab,
d) Prestressing,
e) Epoxy grouting,
f) Bolts and nuts connection,
g) A combination of the above, and
h) Any other method proven by test.

8 TESTS FOR COMPONENTS/STRUCTURES

8.1 Sampling Procedure

8.1.1 *Lot*

All the precast units of the same size, manufactured from the same material under similar conditions of production shall be grouped together to constitute a lot.

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 1 and 2 of Table 1.

Table 1 Sample Size and Rejection Number
(*Clauses* 8.1.1 *and* 8.1.2)

Lot Size	First Sample Size	Second Sample Size	First Rejection Number	Second Rejection Number
(1)	(2)	(3)	(4)	(5)
Up to 100	5	5	2	2
101 to 300	8	8	2	2
301 to 500	13	13	2	2
500 and above	20	20	3	4

The units shall be selected from the lot at random. In order to ensure the randomness of selection, reference may be made to good practice [6-7A(2)].

8.1.2 *Number of Tests and Criteria for Conformity*

All the units selected at random in accordance with col 1 and 2 of Table 1 shall be subjected to the dimensional requirements. A unit failing to satisfy any of the dimensional requirements shall be termed as defective. The lot shall be considered as conforming to the dimensions requirements if no defective is found in the sample, and shall be rejected if the number of defectives is greater than or equal to the first rejection number. If the number of defectives is less than the first rejection number the second sample of the same size as taken in the first stage shall be selected from the lot at random and subjected to the dimensional requirements. The number of defectives in the first sample and the second sample shall be combined and if the combined number of defectives is less than the second rejection number, the lot shall be considered as conforming to the dimensional requirements; otherwise not.

The lot which has been found as satisfactory with respect to the dimensional requirements shall then be tested for load test. For this purpose one unit shall be selected for every 300 units or part thereof. The lot shall be considered as conforming to the strength requirement if all the units meet the requirement; otherwise not.

8.2 Testing on Individual Components

The component should be loaded for one hour at its full span with a total load (including its own self weight) of 1.25 times the sum of the dead and imposed loads used in design. At the end of this time it should not show any sign of weakness, faulty construction or excessive deflection. Its recovery one hour after the removal of the test load, should not be less than 75 percent of the maximum deflection recorded during the test. If prestressed, it should not show any visible cracks up to working load and should have a recovery of not less than 85 percent in 1 h.

8.3 Load Testing of Structure or Part of Structure

Loading test on a completed structure should be made if required by the specification or if there is a reasonable doubt as to the adequacy of the strength of the structure.

8.3.1 In such tests the structure should be subjected to full dead load of the structures plus an imposed load equal to 1.25 times the specified imposed load used in design, for a period of 24 h and then the imposed load shall be removed. During the tests, vertical struts equal in strength to take the whole load should be placed in position leaving a gap under the member.

NOTE — Dead load includes self weight of the structural members plus weight of finishes and walls or partitions, if any, as considered in the design.

8.3.1.1 If within 24 h of the removal of the load, a reinforced concrete structure does not show a recovery of at least 75 percent of the maximum deflection shown during the 24 h under load, test loading should be repeated after a lapse of 72 h. If the recovery is less than 80 percent in second test, the structure shall be deemed to be unacceptable.

8.3.1.2 If within 24 h of the removal of the load, prestressed concrete structure does not show a recovery of at least 85 percent of the maximum deflection shown during the 24 h under load, the test loading should be repeated. The structure should be considered to have failed, if the recovery after the second test is not at least 85 percent of the maximum deflection shown during the second test.

8.3.1.3 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, the overall depth of the section in mm, it is not necessary for the recovery to be measured and the recovery provisions of **8.3.1.1** and **8.3.1.2** shall not apply.

9 MANUFACTURE, STORAGE, TRANSPORT AND ERECTION OF PRECAST ELEMENTS

9.1 Manufacture of Precast Concrete Elements

9.1.1 A judicious location of precasting yard with concreting, initial curing (required for demoulding), storage facilities, suitable transporting and erection equipments and availability of raw materials are the crucial factors which should be carefully planned and provided for effective and economic use of precast concrete components in constructions.

9.1.2 *Manufacture*

The manufacture of the components can be done in a factory for the commercial production established at the focal point based on the market potential or in a site precasting yard set up at or near the site of work.

9.1.2.1 *Factory prefabrication*

Factory prefabrication is resorted to in a *factory for the commercial production for the* manufacture of standardized components on a long-term basis. It is a capital intensive production where work is done throughout the year preferably under a closed shed to avoid effects of seasonal variations. High level of mechanization can always be introduced in this system where the work can be organized in a factory-like manner with the help of a constant team of workmen.

9.1.2.2 *Site prefabrication*

Prefabricated components produced at site or near the site of work as possible.

This system is normally adopted for a specific job order for a limited period. Under this category there are two types that is semi-mechanized and fully-mechanized.

9.1.2.2.1 *Semi-mechanized*

The work is normally carried out in open space with locally available labour force. The equipment machinery used may be minor in nature and moulds are of mobile or stationary in nature.

9.1.2.2.2 *Fully-mechanized*

The work will be carried out under shed with skilled labour. The equipments used will be similar to one of factory production. This type of precast yards will be set up for the production of precast components of high quality, high rate of production.

Though there is definite economy with respect to cost of transportation, this system suffers from basic drawback of its non-suitability to any high degree of mechanization and no elaborate arrangements for quality control. Normal benefits of continuity of work is not available in this system of construction.

9.1.3 The various processes involved in the manufacture of precast elements may be classified as follows:

9.1.3.1 *Main process*

a) Providing and assembling the moulds, placing reinforcement cage in position for reinforced concrete work, and stressing the wires in the case of prestressed elements;
b) Fixing of inserts and tubes, where necessary (for handling);
c) Pouring the concrete into the moulds;
d) Vibrating the concrete and finishing;
e) Curing (steam curing, if necessary); and
f) Demoulding the forms and stacking the precast products.

9.1.3.2 *Auxiliary process*

Process necessary for the successful completion of the processes covered by the main process:

a) Mixing and manufacture of fresh concrete (done in a mixing station or by a batching plant);
b) Prefabrication of reinforcement cage (done in a steel yard or workshop);
c) Manufacture of inserts and other finishing items to be incorporated in the main precast products;
d) Finishing the precast products; and
e) Testing of products.

9.1.3.3 *Subsidiary process*

All other work involved in keeping the main production work to a cyclic working:

a) Storage of materials;
b) Transport of cement and aggregates;

c) Transport of green concrete and reinforcement cages;
d) Transport and stacking the precast elements;
e) Repairs and maintenance of tools, tackles and machines;
f) Repairs and maintenance of moulds, and
g) Generation of steam, etc.

9.1.4 For the manufacture of precast elements all the above processes shall be planned in a systematic way to achieve the following:

a) A cyclic technological method of working to bring in speed and economy in manufacture;
b) Mechanization of the process to increase productivity and to improve quality;
c) The optimum production satisfying the quality control requirements and to keep up the expected speed of construction aimed;
d) Better working conditions for the people on the job; and
e) To minimize the effect of weather on the manufacturing schedule.

9.1.5 The various stages of precasting can be classified as in Table 2 on the basis of the equipments required for the various stages. This permits mechanization and rationalization of work in the various stages. In the precasting, stages 6 and 7 given in Table 2 form the main process in the manufacture of precast concrete elements. For these precasting stages there are many technological processes to suit the concrete product under consideration which have been proved rational, economical and time saving. The technological line or process is the theoretical solution for the method of planning the work involved by using machine complexes. Figure 5 illustrates diagramatically the various stages involved in a plant process.

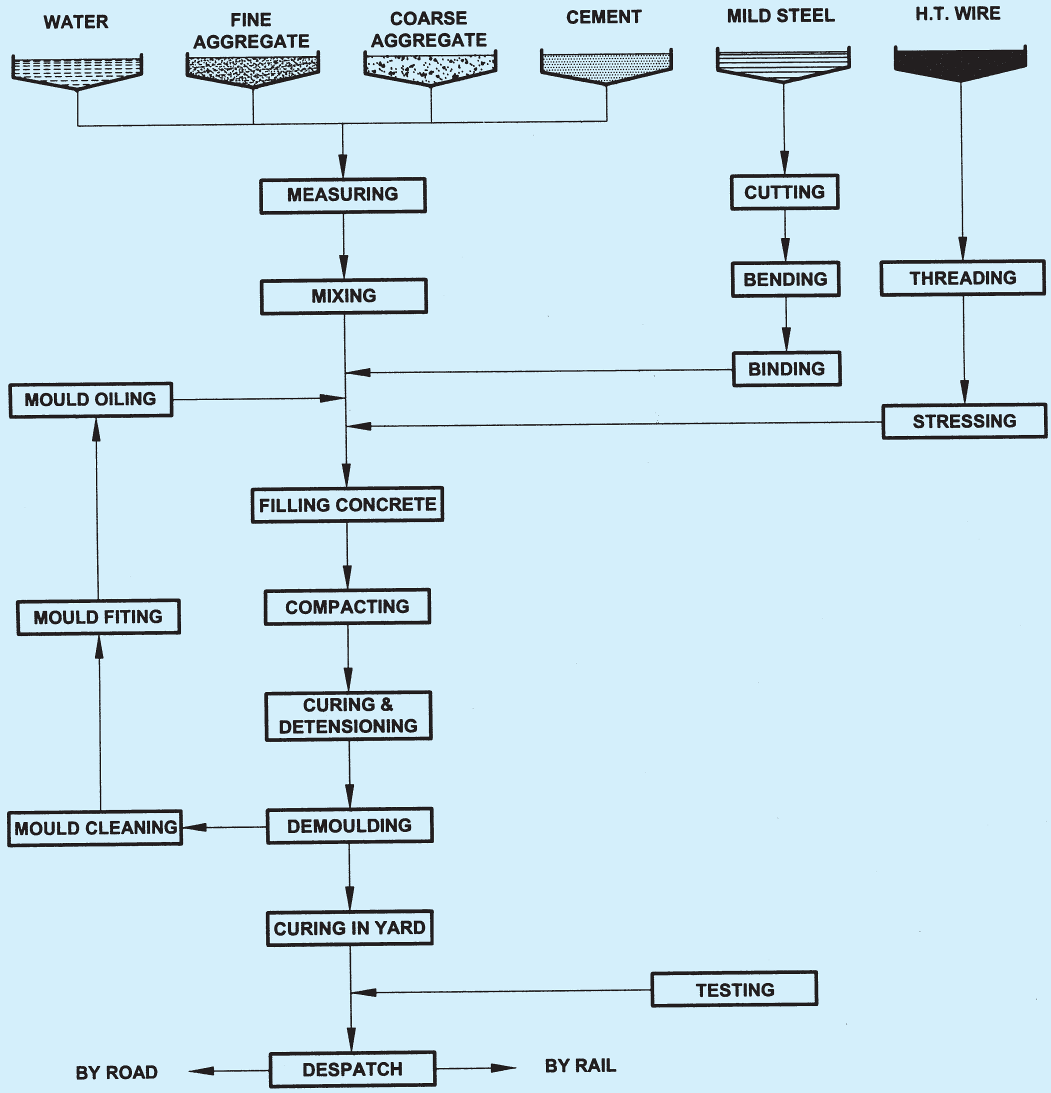


FIG. 5 PLANT PROCESS

Table 2 Stages of Precasting of Concrete Products

[*Clauses* 9.1.5 *and* 9.11(g)]

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 the aggregates, and steel, and storing them in bins, silos or storage sheds
ii)	2	Testing of raw 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 of 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 particular prefabricate.
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 a natural curing with water or an accelerated curing using steam curing 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 initial set. The total demoulding is done after a certain period and the components are then allowed to be cured. All these fall in this operation.
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 technological lines. b) Repair of machines used in the production.

9.1.6 The various accepted methods of manufacture of precast units can be broadly classified into two methods:

a) The 'Stand Method' where the moulds remain stationary at places, when the various processes involved are carried out in a cyclic order at the same place, and

b) The 'Flow Method' where the precast unit under consideration is in movement according to the various processes involved in the work which are carried out in an assembly-line method.

The various accepted precasting methods are listed in Table 3 with details regarding the elements that can be manufactured by these methods.

9.2 Preparation and Storage of Materials

Storage of materials is of considerable importance in the precasting industry, as a mistake in planning in this aspect can greatly influence the economics of production. From experience in construction, it is clear that there will be very high percentages of loss of materials as well as poor quality due to improper storage and transport. So, in a precast factory where everything is produced with special emphasis on quality, proper storage and preservation of building materials, especially cement, coarse and fine aggregates, is of prime importance. Storage of materials shall be done in accordance with Part 7 'Constructional Practices and Safety'.

9.3 Moulds

9.3.1 Moulds for the manufacture of precast elements

Table 3 Precasting Methods

(*Clauses* 9.1.6 *and* 9.9.1)

Sl No.	Precasting Method	Where Used	Dimensions and Weights	Advantages and Remarks
(1)	(2)	(3)	(4)	(5)
i)	Individual Mould Method (precasting method 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 prestressing forces)	a) Ribbed slabs, beams, girders, window panels, box type units and special elements. b) Prestressed railway sleepers, parts of pre-stressed girders, etc.	No limit in size and weight. Depends on the equipment used for demoulding, transporting and placing	a) Strengthening of the cross-section possible b) Openings are possible in two planes
ii)	Battery Form 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 and, roof and floor slabs	Length : 18 m Breadth : 3 m Mass : 5 t	Specially suitable for mass production of wall panels where shuttering cost is reduced to a large extent and autoclave or trench 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 prestressed 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 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 code slab casting)	Roof slabs, foam concrete wall panels and beams cross-section where concreting and vibration are done automatically just as in hollow cored slab casting.	Length : Any desired Breadth : Less than 2 m Height : Less than 3 m	May be used with advantage in the case of un-reinforced blocks, foam concrete panels

may be of steel, timber, concrete and plastic or a combination thereof. For the design of moulds for the various elements, special importance should be given to easy demoulding and assembly of the various parts. At the same time rigidity, strength and watertightness of the mould, taking into consideration forces due to pouring of green concrete and vibrating, are also important.

9.3.2 *Tolerances*

The moulds have to be designed in such a way to take into consideration the tolerances given in **5**.

9.3.3 *Slopes of the Mould Walls*

For easy demoulding of the elements from the mould with fixed sides, the required slopes have to be maintained. Otherwise there is a possibility of the elements getting stuck up with the mould at the time of demoulding.

9.4 *Accelerated Hardening*

In most of the precasting factories, it is economical to use faster curing methods or artificial curing methods, which in turn will allow the elements to be demoulded much earlier permitting early re-use of the forms. Any of the following methods may be adopted:

a) *By Heating the Aggregates and Water Before Mixing the Concrete* — By heating of the aggregates as well as water to about 70°C to 80°C before making the concrete mix and placing the same in the moulds, sufficiently high earlier strengths are developed to allow the elements to be stripped and transported.

b) *Steam Curing* — Steam curing may be done under high pressure and high temperature in an autoclave. This technique is more suited to smaller elements. Alternatively, this could be done using low pressure steam having temperature around 80°C. This type of curing shall be done as specified in **9.5.2**. For light weight concrete products when steam cured under high pressure, the drying shrinkage is reduced considerably. Due to this reason, high pressure steam curing in autoclave is specified for light weight low densities ranging from 300 to 1 000 kg/m^3. For normal heavy concretes as well as light weight concretes of higher densities, low pressure steam curing may be desirable as it does not involve using high pressures and temperatures requiring high investment in an autoclave (*see also* **9.5.2**).

c) *Steam Injection During Mixing of Concrete* — In this method low pressure saturated steam is injected into the mixer while the aggregates are being mixed. This enables the heating up of concrete to approximately 60°C. Such a concrete after being placed in the moulds attains high early strength.

d) *Heated Air Method* — In this method, the concrete elements are kept in contact with hot air with a relative humidity not less than 80 percent. This method is specially useful for light weight concrete products using porous coarse aggregates.

e) *Hot Water Method* — In this method, the concrete elements are kept in a bath of hot water around 50°C to 80°C. The general principles of this type of curing are not much different from steam curing.

f) *Electrical Method* — The passage of current through the concrete panels generates heat through its electro-resistivity and accelerates curing. In this method, the concrete is heated up by an alternating current ranging from 50 volts for a plastic concrete and gradually increasing to 230 V for the set concrete. This method is normally used for massive concrete products.

9.4.1 After the accelerated hardening of the above products by any of the above accepted methods, the elements shall be cured further by normal curing methods to attain full final strength.

9.4.2 Accelerated hardening may also be achieved by the following techniques:

a) *Construction Chemicals* — Suitable construction chemicals may be used.

b) *Consolidation by Spinning* — Such a method is generally used in the centrifugal moulding of pipes and such units. The spinning motion removes excess water, effects consolidation and permits earlier demoulding.

c) *Pressed Concrete* — This method is suitable for fabrication of small or large products at high speed of production. A 100-200 tonnes press compresses the wet concrete in rigid moulds and expels water. Early handling and a dense wear resistant concrete is obtained.

d) *Vacuum Treatment* — This method removes the surplus air and water from the newly placed concrete as in slabs and similar elements. A suction up to about 70 percent of an atmosphere is applied for 20 to 30 minutes per centimetre thickness of the units.

e) *Consolidation by Shock* — This method is suitable for small concrete units dropped repeatedly from a height in strong moulds. The number of shocks required to remove excess water and air may vary from 6 to 20 and the height of lift may be up to as much as half the depth of the mould.

9.4.3 After the accelerated curing of the above products by any of the above accepted methods, the elements shall be cured further by normal curing methods to attain full final strength.

9.5 Curing

9.5.1 The curing of the prefabricated elements can be effected by the normal methods of curing by sprinkling water and keeping the elements moist. This can also be done in the case of smaller elements by immersing them in a specially made water tanks.

9.5.2 *Steam Curing*

9.5.2.1 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 quality throughout the length of the member. The precast elements shall be so stacked, with sufficient clearance between each other and the bounding enclosure, so as to allow proper circulation of steam.

Before the concrete products are subjected to any accelerated method of curing, the cement to be used shall be tested in accordance with accepted standards (*see* Part 5 Building Materials) especially for soundness, setting time and suitability for steam curing.

In the case of elements manufactured by accelerated curing methods, concrete admixtures to reduce the water content can 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 should be avoided for reinforced concrete elements.

9.5.2.2 The surrounding walls, the top cover and the floor of steam curing chamber or tunnel or hood shall be so designed as not to allow more than 1 kcal/m^2/h/°C.

9.5.2.3 The inside face of the steam curing chamber, tunnel or hood shall have a damp-proof layer to maintain the humidity of steam. Moreover, proper slope shall be given to the floor and the roof to allow the condensed water to be easily drained away. At first, when steam is 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.

9.5.2.4 It is preferable to let in steam at the top of the chamber through perforated pipelines to allow uniform entry of steam throughout the chamber.

9.5.2.5 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 product from about 20°C to 35°C should start only after a waiting period ranging from 2 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.

9.5.2.6 The second stage in steam curing process is to heat up the concrete elements, moulds and the surroundings in the chamber:

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 per hour.

   This process takes around 1 h to 1½ h depending upon outside temperature.
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.

9.5.2.7 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.

9.5.2.8 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 per hour.

9.5.3 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 concretes up to M 30 and 75°C for concretes greater than M 45. In the case of light weight concrete, the difference in temperature should not be more than 60°C for concretes less than M 25. For concretes greater than M 50, the temperature differences can go up to 75°C.

9.6 Stacking During Transport and Storage

Every precaution shall be taken against overstress or damage, by the provision of suitable packings at agreed points of support. Particular attention is directed to the inherent dangers of breakage and damage caused by supporting other than at two positions, and also by the careless placing of packings (for example, not vertically one above the other). Ribs, corners and intricate projections from solid section should be adequately protected. Packing pieces shall not discolour, disfigure or otherwise permanently cause mark on units or members. Stacking shall be arranged or the precast units should be protected, so as to prevent the accumulation of trapped water or rubbish, and if necessary to reduce the risk of efflorescence.

9.6.1 The following points shall be kept in view during stacking:

a) Care should be taken to ensure that the flat elements are stacked with right side up. For identification, top surfaces should be clearly marked.
b) Stacking should be done on a 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.

9.7 Handling Arrangements

9.7.1 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.

9.7.2 For precast prestressed concrete members, the residual prestress at the age of particular 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 6 'Structural Design, Section 5 Concrete' shall be permissible.

9.8 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.

9.8.1 The identification markings on the drawings shall be the same as that indicated in the manufacturer's literature and shall be shown in a table on the setting schedule together with the length, type, size of the unit and the sizes and arrangement of all reinforcement.

9.9 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 economy but also from the point of view of design and efficient management. Transport of precast elements must be carried out with extreme care to avoid any jerk and distress in elements and handled as far as possible in the same orientation as it is to be placed in final position.

9.9.1 *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 3.

9.9.2 *Transport from Stacking Yard Inside the Factory to the Site of Erection*

Transport of precast concrete elements from the factory to the site of erection should be planned in such a way so as 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.

9.9.2.1 While transporting elements in various systems, that is, wagons, trucks, bullock carts, care should be taken to avoid excessive cantilever actions and desired supports are maintained. Special care should be taken at location of sharp bends and on uneven or slushy roads to avoid undesirable stresses in elements.

9.9.2.2 Before loading the elements in the transporting media, care should be taken to ensure that the base packing for supporting the elements are located at specified positions only. Subsequent packings must be kept strictly one over the other.

9.10 Erection

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 up of the elements, setting them down into the correct envisaged position;
e) Adjustment to get the stipulated level, line and plumb;
f) Welding of cleats;
g) Changing of the erection tackles;
h) Putting up and removing of 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 a workmanlike finished product.

9.10.1 In view of the fact that 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. For additional information, *see* Part 7 'Constructional Practices and Safety'.

9.10.2 Transport of people, workers or visitors, by using cranes and hoists should be strictly prohibited on an erection site.

9.10.3 In the case of tower rail mounted cranes running on rails, the track shall not have a slope more than 0.2 percent in the longitudinal direction. In the transverse direction the rails shall lie in a horizontal plane.

9.10.4 The track of the crane should be daily checked to see that all fish plates and bolts connecting them to the sleepers are in place and in good condition.

9.10.5 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 EQUIPMENT

10.1 General

The equipment used in the precast concrete industry/

construction may be classified into the following categories:

a) Machinery required for quarrying of coarse and fine aggregates;
b) Conveying equipment, such as, belt conveyors, chain conveyors, screw conveyors, bucket elevators, hoists, etc;
c) Concrete mixing machines;
d) Concrete vibrating machines;
e) Erection equipment, such as, cranes, derricks, hoists, chain pulley blocks, etc;
f) Transport machinery, such as, tractor-cum-trailers, dumpers, lorries, locomotives, motor boats and rarely even helicopters;
g) Workshop machinery for making and repairing steel and timber moulds;
h) Bar straightening, bending and welding machines to make reinforcement cages;
j) Minor tools and tackles, such as, wheel barrows, concrete buckets, etc; and
k) Steam generation plant for accelerated curing.

In addition to the above, pumps and soil compacting machinery are required at the building site for the execution of civil engineering projects involving prefabricated components.

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.

10.2 Mechanization of the Construction and Erection Processes

The various processes can be mechanized as in any other industry for attaining the advantages of mass production of identical elements which in turn will increase productivity and reduce the cost of production in the long run, at the same time guaranteeing quality for the end-product. On the basis of the degree of mechanization used, the various precasting factories can be divided into three categories:

a) With simple mechanization,
b) With partial mechanization, and
c) With complex mechanization leading to automation.

10.2.1 In simple mechanization, simple mechanically operated implements are used to reduce the manual labour and increase the speed.

10.2.2 In partial mechanization, the manual work is more or less eliminated in the part of a process. For example, the batching plant for mixing concrete, hoists to lift materials to a great height and bagger and bulldozer to do earthwork come under this category.

10.2.3 In the case of complex mechanization leading to automation, a number of processes leading to the end-product are all mechanized to a large extent (without or with a little manual or human element involved). This type of mechanization reduces manual work to the absolute minimum and guarantee the mass production at a very fast rate and minimum cost.

10.2.4 The equipment shall conform to accepted standards as listed in Part 7 'Constructional Practices and Safety'.

11 PREFABRICATED STRUCTURAL UNITS

11.1 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 [6-7A(3)].

11.2 For design and construction of precast reinforced and prestressed concrete triangulated trusses reference may be made to good practice [6-7A(4)].

11.3 For design and construction of floors and roofs using various precast units, reference may be made to good practice [6-7A(5)].

11.4 For construction with large panel prefabricates, reference may be made to good practice [6-7A(6)].

11.5 For construction of floors and roofs with joists and filler blocks, reference may be made to good practice [6-7A(7)].

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 by the Authority as a guide in 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.

IS No.	*Title*
(1) 2185	Specification for concrete masonry units:
(Part 1) : 1979	Hollow and solid concrete blocks (*second revision*)

IS No.	*Title*
(Part 2) : 1983	Hollow and solid light weight concrete blocks (*first revision*)
(Part 3) : 1984	Autoclaved cellular aerated concrete blocks (*first revision*)
3201 : 1988	Criteria for design and construction of precast trusses and purlins (*first revision*)
6072 : 1971	Specification for autoclaved reinforced cellular concrete wall slabs
6073 : 1971	Specification for autoclaved reinforced cellular concrete floor and roof slabs
9893 : 1981	Specification for precast concrete blocks for lintels and sills
10297 : 1982	Code of practice for design and construction of floors and roofs using precast reinforced/ prestressed concrete ribbed or cored slab unit
10505 : 1983	Code of practice for construction of floors and roofs using precast concrete waffle units
11447 : 1985	Code of practice for construction with large panel prefabricates
12440 : 1988	Specification for precast concrete stone masonry blocks
13990 : 1994	Specification for precast reinforced concrete planks and joists for flooring and roofing
14143 : 1994	Specification for prefabricated brick panel and partially precast concrete joist for flooring and roofing
14201 : 1994	Specification for precast reinforced concrete channel unit for construction of floors and roofs
14241 : 1994	Specification for precast L-Panel units for roofing
(2) 4905 : 1968	Methods for random sampling
(3) 3935 : 1966	Code of practice for composite construction
(4) 3201 : 1988	Criteria for design and construction of precast trusses and purlins (*first revision*)
(5) 6332 : 1984	Code of practice for construction of floor and roofs using precast doubly-curved shell units (*first revision*)
10297 : 1982	Code of practice for design and construction of floors and roofs using precast reinforced/ prestressed concrete ribbed or cored slab units
10505 : 1983	Code of practice for construction of floors and roofs using precast reinforced concrete waffle units
13994 : 1994	Code of practice for design and construction of floor and roof with precast reinforced concrete planks and RC joists
14142 : 1994	Code of practice for design and construction of floors and roofs with prefabricated brick panel
14215 : 1994	Code of practice for construction of floor and roof with RC channel units
14242 : 1994	Code of practice for design and construction of roof with L-Panel units
(6) 11447 : 1985	Code of practice for construction with large panel prefabricates
(7) 6061	Code of practice for construction of floor and roof with joists and hollow filler blocks:
(Part 1) : 1971	With hollow concrete filler blocks
(Part 2) : 1981	With hollow clay filler blocks (*first revision*)

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 7 Prefabrication, Systems Building and Mixed/Composite Construction: 7B Systems Building and Mixed/Composite Construction

BUREAU OF INDIAN STANDARDS

C O N T E N T S

National Building Code Sectional Committee, CED 46

FOREWORD

Systems building and mixed/composite construction is an upcoming field as far as its development and use in the country is concerned. Two aspects specifically to be borne in mind are the system to be adopted for the different categories of buildings and the sizes of their components. Here the principle of modular co-ordination is of value and its use is recommended.

This section was first published in 1970 and was subsequently revised in 1983.

In this second revision, this section, earlier named as Prefabrication and Systems Building has been renamed and restructured as follows:

Section 7	Prefabrication, Systems Buildings and Mixed/Composite Construction
7A	Prefabricated Concrete
7B	Systems Buildings and Mixed/Composite Construction

This sub-section covers systems building and mixed/composite construction, while such systems approach using predominantly concrete as material for components is being dealt with in sub-section 7A.

In this sub-section, an attempt has been made to prescribe general requirements applicable to all valid existing systems and mixed/composite constructions as also to accommodate any new system introduced in the country in future.

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

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 7 Prefabrication, Systems Building and Mixed/Composite Construction: 7B Systems Building and Mixed/Composite Construction

1 SCOPE

This sub-section covers recommendations regarding modular planning, component sizes, joints, manufacture, storage, transport and erection of prefabricated elements for use in buildings and such related requirements for systems building and mixed/composite construction.

2 TERMINOLOGY

2.1 For the purpose of this sub-section, the following definitions shall apply.

2.1.1 *Authority Having Jurisdiction* — The Authority which has been created by a statute and which, for the purpose of administering the Code/Part, may authorize a committee or an official or an agency to act on its behalf; hereinafter called the 'Authority'.

2.1.2 *Basic Module* — The fundamental module used in modular co-ordination, 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 100 mm for the maximum flexibility and convenience. The symbol for the basic module is *M*.

2.1.3 *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 acid (such as sand), gas generating material (for example, aluminium powder), water and harmless additives (optional); and steam cured under pressure in autoclaves.

2.1.4 *Components* — A building product formed as a distinct unit having specified sizes in three dimensions.

2.1.5 *Composite/Mixed Construction* — Construction involving two or more components, such as, prefabricated structural units of steel, prestressed concrete or reinforced concrete and cast *in-situ* concrete, timber, masonry in brickwork and blockwork, glass and glazing connected together in such a manner that they act integrally.

2.1.6 *Increments* — Difference between two homologous dimensions of components of successive sizes.

2.1.7 *Module* — A unit of size used in dimensional co-ordination.

2.1.8 *Modular Co-ordination* — Dimensional co-ordination employing the basic module or a multi-module.

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.

2.1.9 *Modular Grid* — A rectangular coordinate reference system in which the distance between consecutive lines is the basic module or a multi-module. This multi-module may differ for each of the three orthogonal dimensions of the grid, two in plan and one in vertical direction.

2.1.10 *Multi-module* — A module whose size is a selected multiple of the basic module.

2.1.11 *Prefabricate* — To fabricate components or assembled units prior to erection or installation in a building.

2.1.12 *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*.

2.1.13 *Sandwich Panels* — Panels made by sandwiching a layer of insulation material between two outer layers of hard durable materials like steel, dense concrete, plastic, cement based sheet, ceramic, etc. The hard coverings on two outer faces may be of same or different materials; the three layers being bonded with each other to behave as a composite panel.

2.1.14 *Self-Compacting Concrete* — Concrete that is able to flow 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.

2.1.15 *Shear Connectors* — Structural elements, such as anchors, studs, channels 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.

2.1.16 *System* — The method of construction of buildings with certain order and discipline and repetitive operations using the prefabricated components, tunnel form or engineered shuttering, where the work is organized and follows a defined procedure.

2.1.17 *Unit* — Building material formed as a simple article with all three dimensions specified, complete in itself but intended to be part of a compound unit or complete building. Examples are brick, block, tile, etc.

3 MATERIALS, PLANS AND SPECIFICATIONS

3.1 Materials

3.1.1 *See* Part 6 'Structural Design, Sub-section 7A Prefabricated Concrete', regarding materials and the characteristics to be considered in their selection.

3.1.2 The materials used in prefabricated components may be many and the modern trend is to use concrete, steel, treated wood, aluminium, cellular concrete, light weight concrete, ceramic products, etc. However, this section pertains to mixed/composite construction.

3.2 Plans and Specifications

See Part 6 'Structural Design, Sub-section 7A Prefabricated Concrete'.

4 MODULAR CO-ORDINATION, ARCHITECTURAL TREATMENT AND FINISHES

4.1 Modular Co-ordination

See Part 6 'Structural Design, Sub-section 7A Prefabricated Concrete'.

4.2 Architectural Treatment and Finishes

See Part 6 'Structural Design, Sub-section 7A Prefabricated Concrete'.

5 COMPONENTS

5.1 The preferred dimensions of precast elements used and their casting tolerances shall be in accordance with Part 6 'Structural Design, Sub-section 7A Prefabricated Concrete'.

5.2 The permissible tolerances of timber used shall be in accordance with Part 6 'Structural Design, Sub-section 3A Timber'.

5.3 For permissible tolerances of steel and masonry, reference may be made to relevant Indian Standards.

6 FORMWORK SYSTEMS

The formwork systems which are utilized in buildings shall be as given in **6.1** to **6.5**.

6.1 Tunnel Form

This is a system which casts walls and slab together like a portal in a single pour. Façade walls are precast or of block masonry to enable removal of tunnel form. All components are made up of steel. This produces very rapid construction. Accelerated curing if required is possible enabling early stripping of formwork.

6.2 Slipform

Slipform is a continuously moving form at such a speed that the concrete when exposed has already achieved enough strength to support the vertical pressure from concrete still in the form as well as to withstand nominal lateral forces. Slipform may be classified as straight slipform, tapering slipform and slipform for special applications. Construction of lift cores and stairwell using slipform technique comes under special applications because of their complex sizes, shapes and loads to be lifted alongwith the slipform like walkway truss, etc, which is essential for construction. This system uses hydraulic jacks avoiding crane for lifting of assembly during construction operation. This system facilitates rapid construction and continual casting, creating a monolithic structure thereby avoiding construction joints.

6.3 Aluminium Formwork

This system of formwork uses aluminium, which is light and rust free material, in both sheathing and framework. It may be used for a broad range of applications from wall to slab construction panels to more complicated structures involving bay windows, stairs and hoods. Every component is light enough to be handled easily thereby minimizing the need for heavy lifting equipment.

6.4 Large Panel Shuttering System

This is a system, which gives an advantage of combining speed and quality of construction. The vertical load carrying members are made of steel whereas the horizontal members are of plywood inserted into two wooden beams thereby forming a web flange. All the formwork and support systems shall be designed for the loads coming during the actual execution stage.

6.5 Other/New Systems

Any other/new system may be used for systems building after due examination and approval by the Authority.

7 SYSTEM AND STRUCTURAL SCHEMES

7.1 Several schemes are possible, with certain constraints, using the same set of components. The degree of flexibility varies from system to system. However, in all the systems there is a certain order and discipline.

7.2 The following aspects, among others, are to be considered in devising a system:

a) Effective utilization of spaces;
b) Straight and simple walling scheme;
c) Limited sizes and numbers of components;
d) Limited opening in bearing walls;
e) Regulated locations of partitions;
f) Standardized service and stair units;
g) Limited sizes of doors and windows with regulated positions;
h) Structural clarity and efficiency;
j) Suitability for adoption in low and high rise building;
k) Ease of manufacturing, storing and transporting;
m) Speed and ease of erection; and
n) Simple jointing system.

7.3 Systems for Mixed/Composite Construction

The system of mixed/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.

7.3.1 *Combinations of System Components for Mixed/ Composite Construction*

The following combinations may be used in mixed/ composite construction:

a) Structural steel work and timber roofs on precast frames.
b) Precast floors onto steel and concrete beams, and masonry walls.
c) Profiled metal decking on precast beams.
d) Precast frames onto cast *in-situ* foundations, retaining walls, etc.
e) Precast frames stabilized by masonry walls, steel bracing, etc.
f) Precast cladding in steel or cast *in-situ* frames and *vice versa*.
g) Glass curtain walling, stone cladding or metal sheeting onto precast concrete frames, etc.
h) Reinforced concrete and structural steel as composite columns and beams.

7.3.1.1 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.

7.3.1.2 Structural steelwork is largely used in long span prestressed concrete floors supported on rolled and prefabricated steel beams and also as steel roof trusses supported on concrete columns.

7.3.1.3 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.

7.3.1.4 Brick and block masonry may be combined with precast concrete structures and floors. The most common combinations is to use prestressed floors on load bearing walls.

7.4 Design Considerations

The mixed/composite structures shall be analyzed 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 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 Part 6 'Structural Design, Section 1 Loads, Forces and Effects'. In addition, members shall be designed for handling, erection and impact loads that may be expected during handling and erection.

7.4.1 For mixed and composite construction the following points shall be considered:

a) *Positions of stability cores, walls, bracing, etc.* — In high rise buildings the most popular method is a cast *in-situ* core constructed several storeys ahead of the framework. In medium height buildings this may be precast concrete or brick infill, steel cross bracing or precast concrete diagonal bracing.
b) *Maturity of connections* — This may be decisive for or alter planned site progress unless it is properly managed. Cast *in-situ* grouted joints need a few days of temporary propping unless combined mechanical connections are also used.
c) As a consequence of the above, the need to design some of the key components to achieve temporary stability.
d) *The availability and/or positioning of equipments 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.
e) *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.

f) *Tolerances for economical construction* — This is particularly important where different manufacturers are producing components in different materials.

7.4.2 Other design considerations and safety requirements against progressive collapse shall be in accordance with Part 6 'Structural Design, Sub-section 7A Prefabricated Concrete'.

8 JOINTS

Design of joints shall be in accordance with Part 6 'Structural Design, Sub-section 7A Prefabricated Concrete'.

9 TESTS FOR COMPONENTS/STRUCTURES

Sampling procedure, testing on individual components and load testing of structure shall be in accordance with Part 6 'Structural Design, Sub-section 7A Prefabricated Concrete'.

10 CONSTRUCTIONAL ASPECTS

10.1 Manufacture, Storage, Transport and Erection of Precast Elements

The requirements relating to manufacture, storage, transport and erection of precast concrete elements shall be in accordance with Part 6 'Structural Design, Sub-section 7A Prefabricated Concrete'.

10.2 Decking

Constructional practices relating to decking shall be as given in Annex A.

10.3 Concreting on Decking

Concreting on decking shall be carried out in accordance with Annex B.

11 EQUIPMENT

The requirements relating to equipment used in the precast concrete construction shall be in accordance with Part 6 'Structural Design, Sub-section 7A Prefabricated Concrete'.

12 PREFABRICATED STRUCTURAL UNITS

12.1 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 [6-7B(1)].

12.2 For design and construction of precast reinforced and prestressed concrete triangulated trusses reference may be made to good practice [6-7B(2)].

12.3 For design and construction of floors and roofs using various precast units, reference may be made to good practice [6-7B(3)].

12.4 For construction with large panel prefabricates, reference may be made to good practice [6-7B(4)].

12.5 For construction of floors and roofs with joists and filler blocks reference may be made to good practice [6-7B(5)].

ANNEX A

(*Clause* 10.2)

CONSTRUCTION PRACTICE FOR DECKING

A-1 RECEIVING, STORING AND LIFTING DECKING

A-1.1 Receiving Decking

Decking is packed by the manufacturer into bundles of up to 24 sheets, and the sheets are normally secured with metal banding. Each bundle may be up to 1 m wide (the width of a single sheet) by 750 mm deep, and may weigh up to 2.5 t, depending on sheet length (average mass of sheet being about 1.5 t). Loads are normally delivered by articulated vehicles approximately 16 m long with a maximum gross mass of up to 40 t, and a turning circle of approximately 19 m. It shall be ensured that there is suitable access and appropriate standing and off-loading areas.

Each bundle will be given an identification tag by the manufacturer. The information on each tag shall be checked immediately upon arrival, to prevent incorrect sheets being used, or unnecessary delays if changes are necessary. In particular, the stated sheet thickness shall be checked against the requirement specified on the drawings, and a visual inspection shall be made to ensure that there is no damage.

The bundles shall be lifted from the vehicle. Bundles shall never be off-loaded by tipping, dragging, dropping or other improvized means.

A-1.2 Storing Decking

The decking shall not be delivered more than one month before its anticipated use, as it may be vulnerable to abuse and damage if stored for longer periods on site. If it is not for immediate use, the decking shall be stored on the steel frame. If this is not possible, it shall be located in an area where it will not be contaminated by site traffic, and placed on bearers, which provide a gentle slope to the bundle. This will allow any condensation or rain to drain and a free flow of air around the bundle. Bundles shall not be stacked more than 4 m high, and no other materials shall be stored on top of them. Bearers shall be placed between bundles, and positioned to prevent bending of the sheets.

A-1.3 Lifting and Positioning the Decking

The support steelwork shall be prepared to receive the decking before lifting the bundles onto it. The top surface of the underlying beams shall be reasonably clean. When through-deck welding of shear studs is specified, the tops of the flanges shall be free of primer, paint and galvanising.

The identification tags shall be used to ensure that bundles are positioned on the frame at the correct floor level, and in the nominated bay shown on the deck layout drawing. The bundles shall be positioned such that the interlocking side laps are on the same side. This will enable the decking to be laid progressively without the need to turn the sheets. The bundles shall also be positioned in the correct span orientation, and not at 90° to it. Care shall be taken to ensure that the bundles are not upside down, particularly with trapezoidal profiles. For most trapezoidal decking profiles, the embossments shall be oriented so that they project upwards.

Care is needed when lifting the decking bundles; protected chain slings are recommended for the same. Unprotected chain slings can damage the bundle during lifting. When synthetic slings are used there is a risk of the severing them on the edges of the decking sheets.

If timber packers are used, they shall be secured to the bundle before lifting so that when the slings are released they do not fall to the ground (with potentially disastrous results). Bundles shall never be lifted using metal banding.

A-2 DECK INSTALLATION

A-2.1 Placement of Decking

Breaking open the bundles and installing the decking shall be done only when all the sheets can be positioned and secured. The decking layout drawing shall also be checked to ensure that any temporary support that need to be in position prior to deck laying, is in place.

Access for installation may normally be achieved using ladders connected to the steel frame. Once the laying out the sheets is started by erectors, they shall create working platform by securely fixing the decking as they progress.

The laying of sheets shall begin at the locations indicated on the decking layout drawings. These would normally be at the corner of the building at each level, to reduce the number of 'leading edges', that is unprotected edges where the decking is being laid. When the bundles have been properly positioned, as provided above, there shall be no need to turn the sheets manually, and there shall be no doubt which way up the sheet shall be fixed.

Individual sheets shall be slid into place and, where possible, fixed to the steelwork before moving onto the next sheet. This will minimize the risk of an accident occurring as a result of movement of a sheet when it is being used as a platform. However, for setting-out purposes, it may be necessary to lay out an entire bay using a minimum number of temporary fixings before fully securing the sheets later.

Sheets shall be positioned to provide a minimum bearing of 50 mm on the steel support beams. The ends of adjacent sheets shall be butted together. A gap of up to 5 mm is generally considered not to allow excessive seepage, but, if necessary, the ends of the sheets may be taped together. When end gaps are greater than 5 mm, it is normally sufficient to seal them with an expanding foam filler. The longitudinal edges shall be overlapped, to minimize concrete seepage along the seams. Although not normally required, seam fixings may be necessary in some circumstances. Sheets projecting freely more than 600 mm shall be avoided.

If necessary, sheets shall be cut using a grinder or a nibbler. However, field cutting shall be kept to a minimum and shall only be necessary where a column or other obstruction interrupts the decking. Gaps adjacent to the webs of columns shall be filled in with off-cuts or thin strips of steel. Decking sheets shown as continuous on the decking layout drawing shall never be cut into more than one length. Also, sheets shall never be severed at the location of a temporary support, and the decking shall never be fastened to a temporary support.

As the work progresses, scraps and off-cuts shall be disposed of in a skip placed alongside the appropriate level of working. The skip shall be positioned carefully over a support beam to avoid overloading the decking. If a skip is not available, scraps shall be gathered for collection as soon as is possible. Partially used bundles shall be secured, to avoid individual sheets moving in strong winds.

A-2.2 Fixing of Decking

Decking sheets shall be fixed to the top of the supporting structure. All fixings shall be made through the troughs in the decking. Fixings shall be at approximately 300 mm centres (or in every trough) along the end supports, and at 600 mm centres (or in alternate troughs) along the internal supports. As an absolute minimum, each sheet shall be connected at least twice to each permanent support. The number and placement of fasteners will normally be given on the decking layout drawing. Fixings shall not be made to temporary supports.

The fixings, together with 'through-deck' welded studs (if present) normally provide lateral restraint to the beams during the construction stages.

ANNEX B

(*Clause* 10.3)

CONSTRUCTION PRACTICE FOR CONCRETING ON DECKING

B-1 PLACING CONCRETE

B-1.1 Preparation

Prior to beginning work on the decking, guardrails shall be in position at all perimeters, internal edges and voids. The positions of any props (and back props) shall be checked against the details shown on the decking layout drawings to ensure that adequate support has been provided.

B-1.2 Cleaning the Decking

The surface of the decking shall be reasonably free of dirt, oil, etc prior to concreting.

B-1.3 Construction Joints

Although there is no technical limitation to the area that may be concreted, the usual pour area is up to 1 000 m^2/day. Where the limits of the pour do not coincide with permanent slab edges, construction joints are used to define the extent of the pour.

The locations and details of the construction joints may have an effect on the cracking. The layout and details of the joints shall be determined by the structural designer. For example, when brittle bonded finishes are used, the relationship between the joints in the concrete and the joints in the finishes shall be considered at the outset, to reduce the risk of cracking in undesirable locations.

Where possible, the construction joints shall be located close to butt joints in the decking. Where shear connectors are used, it is preferable to create the joint to one side of the line of the shear connectors, to ensure sound concrete around the studs. If the construction joint cannot be made near a butt joint, it is suggested that no more than one-third of the decking span from a butt joint shall be left unpoured. Concreting shall not be stopped within a sheet length, because excessive deflections may occur when the loads on a continuous decking sheet are not balanced either side of the intermediate support beam.

Stop ends, usually in the form of timber or plastic inserts, are used to create the construction joints. As with all the joints and ends of the decking, they shall be checked for potential grout loss.

B-1.4 Reinforcement

All reinforcement shall be properly supported so that it does not get displaced during concreting. Plastic stools, loops or preformed mesh may be used as 'chairs', but not plastic channels, which can induce cracking. Chairs shall be robust. In particular, the handling and movement of concrete carrying pipes during pumping can cause significant local impacts on the reinforcement.

The reinforcement that has been fixed shall be checked. Particular attention shall be given to checking any additional bar reinforcement, such as may be needed around openings.

B-1.5 Grout Loss

The decking joints shall be closely butted and exposed ends shall be 'stopped' with proprietary filler pieces to avoid grout loss. Gaps greater than 5 mm shall be sealed.

B-2 PLACEMENT

B-2.1 Concrete shall be placed in a way that minimizes the permanent deformation of the decking. This is particularly important for spans greater than 3 m. When concreting is progressed in the same direction as the span of the decking (that is, parallel to the decking ribs), it shall be placed first over supports where the decking is continuous, followed by the mid-span region and finally the areas above the end supports. When concreting is progressed in a direction perpendicular to the decking span (that is, transverse to the decking ribs), it shall be placed first at the edge where a decking sheet is supported by the underlap of an adjacent sheet. This helps to ensure that longitudinal seams between panels remain closed.

The concrete shall be well compacted, particularly near and around any shear connectors. This may be done using a vibrating beam, which may require adequate supports at either ends, or an immersion poker vibrator. Hand tamping is not recommended as a way of compacting the concrete. For slim floors with deep decking, or for other partially encased beams, a poker is needed to ensure proper concrete flow around the beams, beyond the ends of the decking.

B-2.2 Concrete Pumping

Pumping may be adopted for both normal and lightweight concrete mixes. Flow rates in the order of 0.5 m^3 to 1 m^3 of concrete per minute may be achieved, although, clearly, the longer the pump lines and the higher the concrete is to be pumped, the slower the operation. A pump may normally lift the concrete up to 30 m. Secondary pumps, placed at intermediate levels, may be necessary for higher lifts.

Pumplines are normally 150 mm in diameter and are assembled in segments. As the force exerted at bends may be significant, straight line pumping is preferred. The lines shall be supported on timber blocks at intervals of 2 m to 3 m. Re-setting of pumplines is required at frequent intervals as the pour progresses. This means that the outlet pipe shall be moved frequently and carefully so that concrete heaping is minimized. A minimum of two operatives are necessary for this operation, one to hold and manoeuvre the outlet pipe, the other to shovel away excess concrete. No more than 4 workmen shall be present around the pipe outlet during pumping, because of the potential for overloading the decking. The concrete shall not be dropped from the outlet pipe onto the decking from a height of more than about 1 m.

B-2.3 Skip and Barrow

Placing concrete from a skip hung from a crane may be difficult because of obstructions from beams and decking at higher floor levels. However, despite being time consuming, it is sometimes efficient to use the skip and barrow technique for small infill bays.

Skips shall have a means of controlling the rate of discharge, and shall not be discharged from more than 0.5 m above the decking or barrow. When discharging into a barrow, the barrow shall be supported by thick (about 30 mm) boards covering a 2 m × 2 m area, or by a finished part of the slab. Either provision limits impact loads. Barrows shall be run over thick boards placed on the mesh, which shall be supported locally.

B-3 FINISHING, CURING AND DRYING

If power floating is to be carried out, this shall be done within 2 h to 3 h of casting. This allows time for the concrete to harden sufficiently.

As the concrete is only exposed on one surface of a composite floor, it can take longer than a traditional reinforced concrete slab to dry out.

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 by the Authority as a guide in conformance with the requirements of the referred clauses in the Code.

	IS No.	*Title*
(1)	3935 : 1966	Code of practice for composite construction
(2)	3201 : 1988	Criteria for the design and construction of precast — trusses and purlins (*first revision*)
(3)	6332 : 1984	Code of practice for construction of floor and roofs using precast doubly-curved shell units (*first revision*)
	10297 : 1982	Code of practice for design and construction of floors and roofs using precast reinforced/prestressed concrete ribbed or cored units
	10505 : 1983	Code of practice for construction of floors and roofs using precast reinforced concrete waffle units
	13994 : 1994	Code of practice for design and construction of floor and roof with precast reinforced concrete planks and RC joists

IS No.	*Title*
14142 : 1994	Code of practice for design and construction of floors and roofs with prefabricated brick panel
14215 : 1994	Code of practice for construction of floor and roof with RC channel units
14242 : 1994	Code of practice for design and construction of roof with L-Panel units

	IS No.	*Title*
(4)	11447 : 1985	Code of practice for construction with large panel prefabricates
(5)	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*)