রেজিস্টার্ড নং ডি এ-১ “জাতির পিতা বঙ্গবন্ধু শেখ মুজিবুর রহমানের
জন্মশতবার্ষিকী উদ্‌যাপন সফল হোক”

বাংলাদেশ গেজেট

অতিরিক্ত সংখ্যা

কর্তৃপক্ষ কর্তৃক প্রকাশিত

বৃহস্পতিবার ফেব্রুয়ারি ১১, ২০২১

Government of the People’s Republic of Bangladesh
Ministry of Housing and Public Works

Notification

Date : 05-11-1426/18-02-2020

S.R.O. No.55-Law/2020.—In exercise of the powers conferred under section 18A of the Building Construction Act, 1952 (Act No. II of 1953) the Government is pleased to make the following Code by repealing the Bangladesh National Building Code, 2006, namely :—

PART I
Chapter 1
Title, Purpose, Scope, Etc

1. **Title and commencement**.—(1) This Code may be called the Bangladesh National Building Code (BNBC) 2020.

(2) It shall come into force at once.

2. **Purpose.**—(1) The purpose of this Code is to establish minimum standards for design, construction, quality of materials, use and occupancy, location and maintenance of all buildings within Bangladesh in order to safeguard, within achievable limits, life, limb, health, property and public welfare.

(2) The installation and use of certain equipment, services and appurtenances related, connected or attached to such buildings are also regulated herein to achieve the same purpose.

(3) The expressed intent of this Code is to ensure public safety, health and general welfare insofar as they are affected by the construction, alteration, repair, removal, demolition, use or occupancy of buildings, structures or premises, through structural strength, stability, means of egress, safety from fire and other hazards, sanitation, light and ventilation.

3. Scope.—(1) The provisions of this Code shall apply to the design, construction, use or occupancy, alteration, moving, demolition and repair of any building or structure and to any appurtenances installed therein or connected or attached thereto, except such matters as are otherwise provided for in other laws controlling and regulating buildings.

(2) If for any case different sections of this Code provide different specifications for materials, methods of design or construction, or other requirements, the most restrictive specification shall govern.

(3) In case of any conflict between a general requirement and a specific requirement, the specific requirement shall prevail.

(4) Unless otherwise explicitly stated in this Code, all references to part, chapter or section numbers or to provisions not specifically identified by number, shall be construed to refer to such part, chapter, section or provision of this Code.

(5) References made to a section without mentioning a part shall be construed to refer to that section of the part in which the reference is made.

(6) The provisions of any appendix in this Code shall not be mandatory unless they are referred to as such in any section of the Code or they are specifically adopted by any regulation.

(7) Inspection conducted or permission granted for any building or plan of building, under the provisions of this Code, shall not be construed as a warranty of the physical condition of such building or the adequacy of such plan.

(8) Neither the Authority nor any employee thereof shall be liable for damages or any defect or hazardous or illegal condition or inadequacy in such building or plan, nor for any failure of any component of such building which may occur subsequent to such inspection or granting of permission under the provisions of the Code.

4. **Existing buildings**.—(1) Buildings which are in existence on the date of commencement of this Code may have their use or occupancy continued without undergoing any alteration, abandonment or removal unless in the opinion of the Authority such continued use is hazardous to life and property and provided such use or occupancy was legal on the date of commencement of this Code.

(2) Buildings approved before commencement of this Code and compliant under the repealed Code may continue to be used or occupied unless any deviation is made thereafter or any deterioration has rendered the building unsafe in the opinion of the Authority.

(3) Additions, alterations, modifications or repair to an existing building may be made without requiring the existing building to comply with all the requirements of this Code, provided the additions, alterations, modifications or repairs conform to that required for a new building and such additions or alterations shall not be permitted when the existing building is not in full compliance with the provisions of this Code except when the addition or alteration will result in the existing building or structure being no more hazardous based on life safety, fire safety and sanitation than it was before the addition or alteration was undertaken.

(4) Any building together with the new additions shall not exceed the height, number of storeys and area specified in this Code for new buildings having the relevant occupancy and type of construction.

(5) Non-structural alterations or repairs to an existing building or structure which do not adversely affect any structural member, nor reduce the strength of any part of the building or structure to result in an unsafe condition shall be made with materials and components having the required fire resistance.

(6) Change in use or occupancy in an existing building may be made when such change complies with the requirements of this Code for a new building and provided such change does not render any part or the whole of the affected building or structure any more hazardous based on life safety, fire safety and sanitation than it was before such change was effected.

5. Historic or architecturally valuable buildings.—A building or structure which has been designated by official action as having special historical or archaeological interest, or a building or structure identified by a legally constituted authority as being architecturally valuable, may be undertaken for repairs, alterations and additions necessary for its preservation, restoration, rehabilitation or continued use, provided :

(a) the proposed repair, alteration or addition to buildings of historical or archaeological significance is approved by the legally constituted authority, such as the Department of Archaeology;

(b) the proposed repair, alteration or addition to buildings of architectural value does not impair the aesthetic quality and architectural character of such buildings; and

(c) the restored building or structure will be no more hazardous, if any, based on life safety, fire safety and sanitation than the existing building.

PART I
Chapter 2
Definitions

6. Definitions.—In this Code, unless there is anything repugnant in the subject or context,—

ACCESSORY USE	means any use subordinate to the major use which is normally incidental to the major use.
ALTERATION	means any change, addition or modification in construction such as structural, dimensional, or any removal of any part of a building or any change to or closing of any required means of ingress or egress or a change to the fixtures or equipment or any change in land use or occupancy or use.
APPLICANT	means a person, a firm, a company, a corporation, or a government, semi-government or non-government agency who intends to undertake any work regulated by this Code and who has filed an application to the Building Official for this purpose in a form prescribed in the Code.
APPROVED	means approved by the Authority.
APPROVED PLAN	means the set of plans, designs and specifications of building submitted to the Authority as per provision of this Code and duly approved and sanctioned by the Authority.
ARCHITECT	means a person who has a Bachelor Degree in Architecture and is a member of the Institute of Architects, Bangladesh (IAB).
AUTHORITY	means the Bangladesh Building Regulatory Authority.
AUTHORIZED OFFICER	means BUILDING OFFICIAL.
BASEMENT	means a floor of a building more than 50 percent of which is situated at a depth of 1 m or more below crown of the main entry road.
BUILDING	means any permanent or semi-permanent structure which is constructed or erected for human habitation or for any other purpose and includes but not limited to the foundation, plinth, walls, floors, roofs, stairs, chimneys, fixed platform, verandah, balcony, cornice, projections, extensions, annexes etc. The term building will also include the sanitary, plumbing, electrical, HVAC, appurtenances and all other building service installations which are constructed or erected as an integral part of a building.

BUILDING LINE	means the line up to which the plinth of a building may lawfully extend. Also known as SETBACK LINE.
BUILDING OFFICIAL	means a person who is the jurisdictional administrator of this Code appointed by the Authority.
COMMITTEE	means a Building Construction Committee constituted for any area in the prescribed manner, if necessary.
CONSTRUCT	means ERECT.
CONVERSION	means the change in occupancy or premises to any occupancy or use requiring new occupancy permit.
COVERED AREA	means the ground area above the plinth level which is covered by a building structure. The covered area of a building shall exclude gardens, wells, cornice, sunshade, pergola, septic tank, soak well, unpaved uncovered water body, fountains, drainage structures, boundary wall, gates, porch, uncovered staircase, watchman's cabin, detached pump house, garbage chutes and other uncovered utility structures.
DEVELOPMENT	means carrying out construction of buildings, engineering, mining or other operations in, or over or under land or water. Includes re-development and layout and subdivision of any land. 'To develop' and other grammatical variations shall be interpreted accordingly.
DIPLOMA ARCHITECT	means a person who has a Diploma in Architecture from any recognized Polytechnic or Technical Institute and is a member of the Institution of Diploma Engineers, Bangladesh (IDEB).
DIPLOMA ENGINEER	means a person who has a Diploma in Engineering from any recognized Polytechnic or Technical Institute and is a member of the Institution of Diploma Engineers, Bangladesh (IDEB).
DRAIN	means a conduit or channel for conveying water, sewage, or other waste liquid for subsequent disposal.
DRAINAGE	means the disposal of any liquid with a system meant for this purpose.
ENGINEER	means a person who has a Bachelor Degree in Engineering and is a member of the Institution of Engineers, Bangladesh (IEB).

ERECT	means to erect a new building or re-erect an existing building or to convert a building from one occupancy to another. Also known as CONSTRUCT.
FORMATION LEVEL	means finished ground level of a plot. For hilly areas formation levels shall be the gradient of the plot surface.
GEOTECHNICAL ENGINEER	means engineer with Masters degree in geotechnical engineering having at least 2 (two) years of experience in geotechnical design/construction or graduate in civil engineering/engineering geology having 10 (ten) years of experience in geotechnical design/construction.
ENGINEERING GEOLOGIST	means a person having a postgraduate degree in engineering geology and having 2 years of experience in geotechnical exploration and interpretation.
GOVERNMENT	means the government of the People's Republic of Bangladesh.
GRADE	means the lowest point of elevation of the finished surface of the ground, pavement or footpath within the area between the building and the property line or a line 1.5 m from the building whichever is nearer the building.
HEIGHT OF BUILDING	means the vertical distance from a reference datum to the highest point of the building which includes all building appurtenances like overhead water tank, machine room, communication tower etc. The reference datum shall be the elevation of the nearest footpath or the elevation of the nearest road or street or public way at its centre line, whichever is higher.
HIGH RISE BUILDING	means any building which is more than 10-storey or 33 m high from reference datum. Building appurtenances like overhead water tank, machine room, communication tower etc. will not be considered in determining the height.
OCCUPANCY or USE GROUP	means the purpose for which a building or a part thereof is used or intended to be used.
OCCUPANCY, MAJOR	means the major or principal occupancy of a building or a part thereof which has attached to it subsidiary occupancy or occupancies contingent upon it.

OCCUPIER	means a person paying or liable to pay rent or any portion of rent of a building in respect of which the word is used, or compensation or premium on account of occupation of such building and also a rent-free tenant. Does not include a lodger and the words 'occupancy' and 'occupation' do not refer to the lodger. In such cases, the owner himself or herself is living in his or her own building, he or she shall be deemed to be the occupier thereof.
OWNER OF A BUILDING	means the person, organization or agency at whose expenses the building is constructed or who has the right to transfer the same and includes his or her heirs, assignees and legal representatives, and a mortgagee in possession.
PERMIT	means a written document or certificate issued by the Authority for carrying out a specific activity under the provisions of this Code.
PLANNER	means a person who has a Bachelor or a Postgraduate Degree in Planning and is a member of the Bangladesh Institute of Planners (BIP).
PLINTH AREA	means the elements from the building bases which are exposed above the formation level to form a covered floor area by joining the peripheral points of the elements which are intersected at finished floor plane at the height of plinth level.
PLINTH LEVEL	means height of a covered finished floor which is not more than 1 m above the formation level nor 1.85 m from the crown of adjacent road level.
PLOT	means SITE.
PLUMBING ENGINEER	means an Engineer (Civil/Mechanical) who has experience in the field of plumbing or sanitation.
PUBLIC WAY	means ROAD.
RELIABLE LITERATURE	means RELIABLE REFERENCE.
RELIABLE REFERENCE	means reference materials such as published article, codes, standards or other material judged to be reliable by the professional users and specialists in the subject concerned. This may also be referred to as RELIABLE LITERATURE.

ROAD	means a thoroughfare or public way which has been dedicated or deeded to the public for public use and also known as STREET.
ROAD LINE	means a line defining the side limits of a road.
ROOM HEIGHT	means the clear head room between the finished floor surface and the finished ceiling surface or the underside of the joists or beams, whichever is lower.
SANCTIONED PLAN	means the set of plans, design and specifications of a building submitted to the Authority as per provision of this Code and duly approved and sanctioned by the Authority.
SERVICE ROAD	means a road or lane provided at the rear or side of a plot for service purposes.
SETBACK LINE	means BUILDING LINE.
SITE	means a piece or parcel of land on which a building is intended to be or has already been constructed and also known as PLOT.
SPECIALIST	means a professional who by education, research, practice and experience specializes in a particular branch of a broader discipline and is generally judged to be so by the professional body in the relevant discipline.
STOREY	means the portion of a structure between tops of two successive finished floor surfaces and for the topmost storey, from surface of the finished floor of topmost floor to the top of the roof above.
STOREY, FIRST	means the lowest storey in a building which qualifies as a storey as defined herein; for a building with a basement, it is the storey just above the basements.
STREET	means ROAD.
STREET LEVEL	means the elevation of the centre line of any road or street which a plot fronts.
STREET LINE	means ROAD LINE.
SUPERVISOR, CONSTRUCTION	means an Architect or Engineer or Diploma Architect or Diploma Engineer having experience in supervision of construction works.
UNSAFE BUILDING	means a building which, in the opinion of the Building Official, is structurally unsafe, or insanitary, or lacks proper means of ingress or egress, or which constitutes a hazard to life or property.

PART I
Chapter 3
Abbreviations

7. Abbreviations of names and words.—(1) Names of institutions, organizations and professional societies referred to in this Code are listed below in an alphabetical order, namely :—

ACI	American Concrete Institute; Box 19150, Redford Station, Detroit, MI 48219, USA.
AISC	American Institute of Steel Construction, Inc.; 400 North Michigan Avenue, Chicago, IL 60611, USA.
AISE	Association of Iron and Steel Engineers; Suite 2350, Three Gateway Center, Pittsburgh, PA 15222, USA.
AISI	American Iron and Steel Institute; Suite 300, 1133 15th Street N.W., Washington, DC 20005, USA.
ANSI	American National Standards Institute; 1430 Broadway, New York, NY 10018, USA.
ASHRAE	American Society of Heating, Refrigerating and Air-conditioning Engineers, Inc.; 345 East 47th Street, New York, NY 10017, USA.
ASME	American Society of Mechanical Engineers; United Engineering Centre, 345 East 47th Street, New York, NY 10017, USA.
ASTM	American Society for Testing and Materials; 1916 Race Street, Philadelphia, PA 19103, USA.
AWS	American Welding Society; 550 N.W. LeJeune Rd., P.O. Box 351040, Miami, FL 33135, USA.
BIP	Bangladesh Institute of Planners, Planner's Tower (Level-7), 13/A, Bir Uttam C.R. Datta (Sonargaon) Road, Bangla Motor, Dhaka-1000, Bangladesh.
BOCA	Building Officials and Code Administrators International Inc.; 1313 East 60th Street, Chicago, IL 60637, USA.
BPDB	Bangladesh Power Development Board; WAPDA Building, Motijheel Commercial Area, Dhaka-1000, Bangladesh.
BSI	British Standards Institution; 2 Park Street, London W1A 2BS, UK.
BSTI	Bangladesh Standards and Testing Institution; 116A Tejgaon Industrial Area, Dhaka-1208, Bangladesh.
BWDB	Bangladesh Water Development Board; WAPDA Building, Motijheel Commercial Area, Dhaka-1000, Bangladesh.

CDA	Chittagong Development Authority; Station Road, Chittagong, Bangladesh.
CGSM	Canadian General Standards Board; Technical Information Unit, Ottawa, CANADA K1A 1G6.
DOA	Department of Architecture; Sthapatya Bhaban, Shahid Capt. Mansur Ali Sarani, Segunbagicha, Dhaka-1000, Bangladesh.
DPHE	Department of Public Health Engineering; DPHE Bhaban, 14, Shaheed Captain Mansur Ali Sarani, Kakrail, Dhaka-1000, Bangladesh.
EED	Education Engineering Department; Shikkha Bhaban, Dhaka-1000, Bangladesh.
HED	Health Engineering Department; Ministry of Health and Family Welfare, 105-106, Motijheel C/A, Dhaka-1000, Bangladesh.
FM	Factory Manual; Standards Laboratories Department, 1151 Boston Providence Turnpike, Norwood, MA 02062, USA.
FSCD	Fire Service and Civil Defence, Kazi Alauddin Road, Dhaka-1000, Bangladesh.
HBRI	Housing and Building Research Institute, 120/3, Darus-Salam, Mirpur, Dhaka, Bangladesh.
IAB	Institute of Architects Bangladesh, Plot-11, Block-E, Road-7, Sher-e-Bangla Nagar, Agargaon, Dhaka.
IEB	The Institution of Engineers, Bangladesh, Ramna, Dhaka-1000.
IDEB	Institution of Diploma Engineers, Bangladesh, IDEB Bhaban, 160/A, Kakrail VIP Road, Dhaka-1000.
ICBO	International Conference of Building Officials, 5360 South Workman Mill Road, Whittier, CA 90601, USA.
ISO	International Organization for Standardization, 1, Rue de Varembé, Case Postal 56, CH-1211, Genève 20, Switzerland.
ISSMFE	International Society of Soil Mechanics and Foundation Engineering, University Engineering Department, Trumpington St, Cambridge CB21PZ, UK.
KDA	Khulna Development Authority, Shib Bari Crossing, Khulna-9100, Bangladesh.
LGED	Local Government Engineering Department, LGED Bhaban, Sher-e-Bangla Nagar, Agargaon, Dhaka-1207. Bangladesh.

NFPA	National Fire Protection Association, Batterymarch Park, Quincy, MA 02269, USA.
NHA	National Housing Authority, Grihayan Bhaban, 82, Segunbagicha, Dhaka, Bangladesh.
PWD	Public Works Department, Purto Bhaban, Shahid Capt. Mansur Ali Sarani, Segunbagicha; Dhaka-1000, Bangladesh.
RAJUK	Rajdhani Unnayan Kartripakkha, Rajuk Avenue, Motijheel, Dhaka-1000, Bangladesh.
RCSC	Research Council on Structural Connections of the Engineering Foundation, American Institute of Steel Construction (AISC).
RDA	Rajshahi Development Authority, Rajshahi-6203, Bangladesh.
RMA	Rubber Manufacturing Association, 1400 K Street N.W., Washington, DC 20005, USA.
SBCCI	Southern Building Code Congress International, 3617 8th Ave, S. Birmingham, AL 35222, USA.
SMACNA	Sheet Metal and Air Conditioning Contractors' National Association, 8224 Old Courthouse Road, Tysons Corner, Vienna, VA 22180, USA.
SPRI	Single Ply Roofing Institute, 104 Wilmont Road, Suite 201, Deerfield, IL 600015-5195, USA.
UDD	Urban Development Directorate, Ministry of Housing and Public Works, 82, Segunbagicha, Dhaka-1000, Bangladesh.
UL	Underwriters Laboratories Inc., 207 East Ohio Street, Chicago, IL 60611, USA.

(2) The abbreviations of words used in this Code are listed below in an alphabetical order. Abbreviations not explicitly defined herein below shall be construed to have their usual meaning as the context implies.

BDS	Bangladesh Standards; published by the BSTI
BNBC	Bangladesh National Building Code; published by HBRI
BS	British Standard; published by the BSI
CBF	Concentric Braced Frame
CFC	Chlorofluorocarbon

CGI	Corrugated Galvanized Iron
CWPC	Cold Drawn Low Carbon Wire Prestressed Concrete
DCP	Dry Chemical Powder (fire extinguisher)
DDT	Dichlorodiphenyltrichloroethane
DPC	Damp-proof Course
EBF	Eccentric Braced Frame
FAR	Floor Area Ratio
FM	Fineness Modulus
FPA	Flood Prone Area
GI	Galvanized Iron
IBC	International Building Code
IMRF	Intermediate Moment Resisting Frame
IS	Indian Standard; published by the Bureau of Indian Standards
LFD	Load Factor Design
LPG	Liquefied Petroleum Gas
MCSP	Multipurpose Cyclone Shelter Program
OMRF	Ordinary Moment Resisting Frame
RC	Reinforced Concrete
RS	Rolled Steel
RSJ	Rolled Steel Joist
SMRF	Special Moment Resisting Frame
SPA	Surge Prone Area
SRSS	Square Root of the Sum of the Squares
UBC	Uniform Building Code; published by the ICBO
WSD	Working Stress Design
cps	Cycles per second

PART II
Chapter 1
Purpose and Applicability

8. Purpose.—The purpose of this Part is to relate the provisions of the Code to different documents for administration and enforcement of the Code and all legal issues shall be referred to the Building Construction Act, 1952.

9. Applicability.—The requirements of this Code shall be complied within any construction, addition, alteration or repair, use and occupancy, location, maintenance, demolition and removal of a building or structure or any appurtenances connected or attached to it as set forth herein below:

(a) **Construction:** For construction of a new building, the provisions of this Code shall apply to its design and construction;

(b) **Removal:** For removal of any portion or the whole of a building, the provisions of this Code shall apply to all parts of the building whether removed or not;

(c) **Demolition:** For dismantling or demolition of any part or the whole of a building, the provisions of this Code shall apply to any remaining portion and to the work involved in the dismantling or demolition process;

(d) **Alteration:** For alteration of a building, the provisions of this Code shall apply to the whole building whether existing or new. If the portion of the building to which the alteration is made is completely self-contained with respect to the facilities and safety measures required by this Code, the provisions of this Code shall apply only to that portion and not to the whole building.

(e) **Maintenance:** Maintenance work shall be undertaken for all new and existing buildings and all parts thereof to continue their compliance with the provisions of this Code. All devices, equipment and safeguards installed as per the requirements of this Code shall be maintained in conformity with the edition of the Code under which installed. The owner of the building or his designated agent shall at all times be responsible for the safe and sanitary maintenance of the building or structure, its means of egress facilities and the safety devices, equipment and services installed therein. The Authorized Officer or his delegated persons as described in relevant documents mentioned in Chapter 2 may cause re-inspection of a building to determine its continued compliance with this Section.

(f) **Repair:** Application or notice to the Authority administering the Code is not necessary for ordinary repairs to buildings or structures, provided such repairs do not involve the cutting away of any wall or portion thereof, the removal or cutting of any structural or bearing element, the removal or alteration of any required means of egress, or the rearrangement of any parts of a structure affecting the access and exit facilities. All works involving addition to, alteration or change of use of any building or structure shall conform to the requirements set forth in Part 9 of this Code.

(g) **Land Development:** For development of a land for construction of a building, the provisions of this Code shall apply to the entire development work. For land development purposes the following laws shall also be applicable:

(i) Building Construction Act 1952;

(ii) Private Residential Land Development Rules 2004;

(iii) Natural Water Body Protection and Preservation of Open Space and Playground Act 2000.

PART II
Chapter 2
Establishment Of Authority, Etc

10. Establishment of Authority.—The Government may, with the approval of the Ministry of Public Administration, Finance Division and other relevant Ministries and Divisions, by a notification in the official Gazette, establish the Bangladesh Building Regulatory Authority (BBRA).

11. Head office of the Authority.—The head office of the Authority shall be in Dhaka.

12. Constitution of Authority.—(1) The Authority shall consist of the following 5 (five) members, namely:-

(a) a civil engineer having professional experience of 30 years in design/ construction/teaching/research related to building;

(b) an architect having professional experience of 30 years in design/ construction/ teaching/ research related to building;

(c) a planner having professional experience of 30 years in planning/ teaching/research related to building;

(d) a judge or legal practitioner having professional experience of 30 years in law including the qualification for appointment of a judge of the High Court Division;

(e) a person having professional experience of 30 years in Bangladesh Civil Service.

(2) The Government shall appoint the members of the Authority and they shall hold office for a period of 3 (three) years.

(3) The Government shall nominate one of the members as the Chairman of the Authority.

13. Responsibilities of the Authority.—The Authority shall—

(a) be the organization responsible for establishing regulatory framework for building design and construction with efficient and effective compliance mechanism;

(b) develop building check and control procedure for ensuring high degree of regulatory compliance in planning and the Code requirements and reduce information asymmetry between the end user (building occupant, home owner) and seller (developers, builders);

(c) streamline and improve transparency through dissemination of information related to built environment including detail land use plan, regulations on safety, water and environmental conservation, health, energy efficiency and urban planning requirements through print and digital media including its website;

(d) develop an effective licensing system, jointly with the professional bodies by forming a National Council for Licensing of Building Professionals (NCLBP) for conducting examinations for the members of those respective professional bodies;

(e) update the requirements of building permit and inspection procedure as per this Code;

(f) require the owner of an existing or under construction high risk building, having major impacts on public safety for inhabitants within and near the building, to carry out review of design and construction by licensed professionals acceptable to the Authority;

(g) introduce IT based automated procedure for permits and online information system to enable the applicants to track the progress of the permitting process;

(h) establish an independent quasi-judicial dispute-resolution body that can make binding decisions in disputes between practitioners, developers, stakeholders and permitting authorities on matters related to interpretation of the Code or sufficiency of compliance, which cannot be appealed except to the High Court Division on matters of law;

(i) recommend punitive and other measures against developers and professionals for violation of the Code and safety measures;

(j) take measures for updating of the Code in light of research, improved building design and construction technique, availability of new products and technology;

(k) advise the Government on policy and administration of building regulations including capacity development;

(l) take up matters from time to time which the Authority deems necessary.

14. Office of the Building Officials, etc.—(1) The Authority shall designate specific geographical jurisdiction as the Office of the Building Official.

(2) The Office of the Building Official shall be established at various local or regional development area or local government levels.

(3) The Authority may, in order to proper functioning of it, subject to the Organogram approved by the Government and having required qualifications, appoint such numbers of Building Officials, technical assistants, inspectors and other employees as required.

(4) The administrative and operational chief of the Code enforcing office shall be designated as the Building Official who shall act on behalf of the Authority.

(5) The Building Official may designated an employee or employees who shall carry out the specified duty and exercise the specified power of the Building Official.

15. Building Construction Committee.—(1) The Building Official shall exercise through a Building Construction Committee comprising four members excluding Building Official.

(2) Building Construction Committee shall consist of one architect, one civil engineer, one town planner and representative from concerned body.

(3) Building Official shall work as ex-officio member-secretary of the Committee.

16. Qualifications of Building Official.—The person to be designated as the Building Official shall be at least an architect, a civil engineer or a town planner in addition to fulfilling any other requirement of the Authority.

17. Administrative jurisdiction of Building Official.—(1) The areas delineated below in Table 2.2.1 shall be under the jurisdiction of the Building Officials located in the offices/authorities mentioned in the right hand column:

Table 2.2.1: Jurisdiction of Building Officials of Designated Offices/Authorities

Sl.	Area	Authority
1	Areas falling under the master plan control of Rajdhani Unnayan Kartipokhkha (RAJUK)	RAJUK
2	Areas falling under the master plan control of Chittagong Development Authority (CDA)	CDA
3	Areas falling under the master plan control of Rajshahi Development Authority (RDA)	RDA
4	Areas falling under the master plan control of Khulna Development Authority (KDA)	KDA
5	Areas falling under the master plan control of any Development Authority to be established in future	Relevant development authority
6	Areas falling under the geographical jurisdiction of any City Corporation where no Development Authority exists	Relevant city corporation
7	Areas falling under the geographical jurisdiction of any Municipality where no Development Authority exists	Relevant municipality

Sl.	Area	Authority
8	Areas not falling under any of the above	Office of The Executive Engineer Public Works Department (PWD)
9	Special areas, if any	To be declared by the government as and when necessary

(2) There may be as many Building Officials as required depending upon the area of jurisdiction, but every Building Official shall be in charge of an independent and well demarcated area.

18. Merging the Jurisdictions under small local bodies.—Small local bodies like Pourashavas, Upazila, Union Parishad, located outside the larger city municipalities and having insufficient funds for individually carrying out the task of the Code enforcing agency may jointly appoint or designate, with the approval of the Authority, a Building Official who shall have a jurisdiction over the combined area of jurisdiction of the concerned local bodies.

19. Restrictions on the Building Official.—(1) The Building Official or any employee designated by him in this behalf shall not in any way, directly or indirectly, be engaged in planning, design, construction, repair, maintenance, modification or alteration of a building, certification of any work or materials, supply of materials, labor, equipment or appliances or any other work regulated by the provisions of this Code.

(2) The Building Official or such designated employee shall not be interested in business, either directly or indirectly, as planner, engineer, architect, builder or supplier or in any other private business transaction or activity within the jurisdiction of the Authority which conflicts with his official duties or with the interest of the Code enforcing agency.

(3) If any Building Official or designated employee violates the restrictions, he shall be liable to punishment as per service rule of the government.

20. Damage Suit.—(1) In the process of discharging the official duties as required and permitted by the Code, the Building Official or any employee shall not be personally liable for any damage that may be caused to any person or property.

(2) Any suit filed against the Building Official or any employee because of an act performed by him in the official discharge of his duties and under the provisions of the Code shall be defended by the legal representative of the Authority until the final decision of the proceedings.

(3) In no case shall the Building Official or any employee be liable for costs in any legal action, suit, or defense proceedings that may be filed in pursuance of the provisions of the Code.

21. Powers and duties of the Building Official.—(1) The Building Official shall be authorized to enforce all the provisions of this Code and for such purposes the Building Official shall have the power of a law enforcing officer.

(2) Applications shall be made in writing to the Building Official for any erection, construction, addition, alteration, modification, repair, improvement, removal, conversion, change of occupancy, and demolition of any building or structure regulated by this Code.

(3) The Building Official shall receive such applications, examine the premises, enforce compliance with this Code and issue permits for the intended work.

(4) All necessary notices and orders to correct illegal or unsafe conditions, to require the specified safeguards during construction, to require adequate access and exit facilities in existing buildings and to ensure compliance with all the requirements of safety, health and general welfare of the public as included in this Code shall be issued by the Building Official.

(5) The Building Official may enter a building or premises at reasonable times to inspect or to perform the duties imposed by this Code if:

(a) it is necessary to make an inspection to enforce the provisions of this Code; or

(b) he has reasonable cause to believe that a condition contrary to or in violation of this Code exists making the building or the premises unsafe, hazardous or dangerous.

(6) If the building or premises is occupied, the Building Official shall present credentials to the occupant and request entry.

(7) If the building or premises is unoccupied, the Building Official shall first make a reasonable effort to locate the owner or any other person having charge or control of the building or premises and request entry.

(8) If entry into the building or premises is refused or the owner of the unoccupied building or premises cannot be located, the Building Official shall secure entry as provided by the law.

(9) The Building Official or an employee designated by him in this behalf shall inspect all construction or work for which a permit is required or he may accept reports of inspection by a licensed engineer, architect or planner provided he satisfies the requirements of Table 2.3.4 and may disapprove the report showing specific reason for disapproval.

(10) The work or construction to be inspected shall remain accessible and exposed for inspection purposes until the approval is obtained.

(11) All reports of inspection shall be in writing and certified by the Building Official or the licensed engineer or the architect making the inspection.

(12) Approval of work or construction as a result of such inspection shall not be interpreted to be an approval of a violation of the provisions of this Code or of other law.

(13) The Building Official may require survey of the site and adjoining areas to verify that the structure is located in accordance with the approved plans.

(14) The Building Official or such designated employee shall carry proper identification when inspecting structure or premises in the performance of duties under the provision of this Code.

(15) The Building Official may issue an order for immediate discontinuation of a work and cancellation of a previous permit for such work at any stage if:

(a) any work is being done contrary to the provision of this Code or other pertinent laws; or

(b) it is determined by him that the construction is not proceeding according to the approved plan, dangerous or unsafe.

(16) In such cases the Building Official shall notify the owner in writing of such an order by showing the reason for the order, and the conditions under which the cited work will be permitted to resume.

(17) When there is insufficient evidence of compliance with the provisions of this Code, a Building Official shall have the authority to require test as evidence of compliance to be made at no expense to the office of the Building Officials and the test shall be performed by an agency approved by the Building Official.

(18) Any person who shall continue any work after having been served with a stop work order, except such work as that person is directed to perform to remove a violation or unsafe condition, shall be subject to penalties as prescribed by law.

(19) The Building Official may order the current uses of a building discontinued and the building or portion thereof vacated by serving a notice on any person if the Building Official determines that the building or structure or equipment therein regulated by this Code is being used contrary to the provisions of this Code, such person shall discontinue the use within the time prescribed by the Building Official after receipt of such notice to make the structure, or portion thereof, comply with the requirements of this Code.

(20) The Building Official shall maintain records of all applications and drawings received, permits and orders issued, inspections made and reports prepared and submitted by other recognized agencies.

(21) Copies of all relevant papers and documents for enforcement of the Code shall be preserved by the Building Official. All such records shall be kept open to public inspection at all suitable times.

(22) The Building Official may engage, subject to the approval of the Authority, an expert or a panel of experts for opinion on unusual technical issues that may arise in administering the provisions of the Code.

22. Board of Appeal.—(1) The Authority may, with the approval of the Government, constitute a Board of Appeal to hear and decide appeals of orders, decisions or determinations made by the Building Officials related to the application and interpretation of this Code.

(2) The Board of Appeal shall consist of members appointed by the Authority who are noted for their educations and experience in the relevant field of building construction and whose term of office shall be as decided by the Authority.

(3) The Board of Appeal shall provide reasonable interpretation of the provisions of this Code and determine the suitability of alternative materials or methods of design or construction.

(4) The Board of Appeal shall, with the approval of the Government, adopt rules of procedure for conducting its business, and shall communicate all decisions and findings in writing to the appellant with a copy to the Building Official.

(5) The Board of Appeal shall have no discretion for interpretation of the administrative provisions contained in Part 2 of this Code nor shall be empowered to waive any requirement of this Code.

23. Requirement of certification of work.—Any planning, design, supervision of construction, repair, maintenance, modification and alteration of buildings, or any other work regulated by the Code shall be certified by a licensed engineer, architect or planner for its compliance with the provisions of the Code as per Tables 2.3.3 and 2.3.4.

24. Limits of professional conduct.—(1) Any licensed architect, engineer or planner may take assistance from fellow professionals who are not licensed but is member of professional bodies and who shall work under his direct control and he shall be allowed to plan, design and supervise construction, repair, maintenance, alteration and modification of buildings or structures regulated by this Code provided the licensed professional certify compliance of the work with the provisions of the Code.

(2) In case of any violation of the Code the licensed professionals who shall certify will be liable for action through professional bodies and such person may provide any such certificate as long as his or her services are recognized by the Building Official and such recognition is not withdrawn under the provisions of this Code.

25. Violation and penalties.—Any person, firm, corporation or government department or agency who as owner of the property erects, constructs, enlarges, alters, repairs, moves, improves, removes, converts, demolishes, equips, uses, occupies or maintains any building or structure or cause or permit the same to be done in violation of this Code shall be guilty of an offence and the Authority shall take legal action against such offenders as prescribed by law.

Explanation.—For the purpose of this provisions the term "owner" shall include any developer who by appointment, contract or lease is responsible for such activities.

26. Professional violation.—(1) The engineer, architect or planner responsible for design, supervision or certification of any construction or other work of a building or structure shall ensure compliance of such work with the provisions of this Code.

(2) Any violation of the Code or any other professional misconduct insofar as implementation of the provisions of this Code is concerned including making false statements or issuing false certificates or any incidence of proven professional incapability shall be recommended to the respective professional bodies for necessary disciplinary measure including withdrawal of recognition or registration.

27. Obligation of offender.—A person shall not be relieved from the duty of carrying out the requirements or obligations imposed on him or her by virtue of the provisions of this Code even if such person is convicted for an offence under the provisions of this Section.

28. Conviction no bar to further prosecution.—If a person is convicted under the provisions of this Code for failing to comply with any of its requirements or obligations such conviction shall not act as a bar for further prosecution for any subsequent failure on the part of such person to comply.

PART II
Chapter 3
Permits and Inspections

3.1 Permits

No building or structure regulated by this Code shall be erected, constructed, enlarged, altered, repaired, moved, improved, removed, converted or demolished without obtaining permit for each such work from the Building official.

Exceptions:

The following works are exempted from the requirement of a permit unless they do not otherwise violate the provisions of this Code, for the said work or any other adjacent property, regarding general building requirements, structural stability and fire safety requirements of this Code:

(a) Opening or closing of a window or a door or a ventilator;

(b) Providing internal doors;

(c) Providing partitions;

(d) Providing false ceiling;

(e) Gardening;

(f) Painting;

(g) Plastering and patch work;

(h) Re-flooring;

(i) Construction of sunshades on one's own land;

(j) Re-erection of portion of buildings damaged by earthquake or cyclone or other natural calamities, to the extent and specification as existed prior to such damage; and

(k) Solid boundary walls less than 1.5 m and open boundary wall less than 2.75 m in height.

3.2 Types of Permit

Building permit shall comprise of the following 4 (four) stages:

(a) Land use certificate.

(b) Large and specialized project permit.

(c) Building permit.

(d) Occupancy certificate.

Permit of all or any of the above may be necessary for a particular area/city/town/ municipality. Requirement in this regard shall be incorporated in the building construction byelaws/rules/regulations valid for that particular area/city/town/ municipality.

3.2.1 Validity of Permits from the Date of Issuance

The validity of permits for different purposes from the date of issuance shall be as follows:

(a)	Land use certificate	24 months
(b)	Large and specialized project permit	24 months
(c)	Building permit	36 months (unless construction up-to plinth level is done)
(d)	Occupation certificate	Perpetual (unless any change in use and physical properties)

3.2.2 Permits Obtained Prior to Adoption of Code

If permit for a building or structure or a work regulated by this Code is obtained before adoption of this Code and the building or structure or work for which the permit is obtained is not completed within three years from the date of issuance of such permit, the said permit shall be deemed to have lapsed and fresh permit shall be necessary to proceed further with the work in accordance with the provisions of this Code.

3.3 Constitution of Building Permit Committees

3.3.1 As per the provisions laid out in the Building Construction Act the government may constitute various committees to examine and scrutinize applications mentioned in Clause 3.2 above and approve or refuse permits thereby.

3.3.2 Each committee will have specific Terms of Reference and Work Procedure.

3.4 Application For Permit

3.4.1 Any person who intends to undertake any work on a building or structure or land regulated by this Code shall file application in writing on the prescribed form furnished by the Building official for that purpose.

3.4.2 Application for permit for any work under the provisions of this Code shall be accompanied by necessary documents, drawings, certificates, clearances and other relevant information as required by the Building Official for that particular city/town/municipality/jurisdiction area etc.

3.4.3 The drawings shall have any of the sizes specified in the Table 2.3.1:

Table 2.3.1: Drawing Sizes for Permit Applications

Notation (ISO Standard)	Size (mm)
A 0	841 x 1189
A 1	594 x 841
A 2	420 x 594
A 3	297 x 420
A 4	210 x 297

3.4.4 Operation and Maintenance of Utility Services

The government may undertake works for operation, maintenance, development or execution of any of the following utility services without requiring obtaining permit from the Building Official.

(a) Railways

(b) National Highways

(c) National Waterways

(d) National Gas grid

(e) National Power grid

(f) Major Ports

(g) Airways and Aerodromes

(h) Telecommunications

(i) Electronic Broadcasting Services

(j) Any other services which the Government may, by notification, declare to be a service for the purpose of this Section if the Government is of the opinion that the operation, maintenance, development or execution of such service is essential to the community.

Buildings constructed in connection with these services shall conform to the specifications of this Code.

3.5 Disposal of Application

3.5.1 Subject to the submission of correct and complete application for the permits included in Sec 3.2 above, should be disposed by the Building Official within the time limit as shown in Table 2.3.2:

Table 2.3.2: Time Limit for Disposal of Application for Permits

Type of Permit	Maximum time allowed for disposal (approval or refusal by the Building official)
Land use certificate	15 days
Large and Specialized Project permit	45 days
Building permit	45 days
Occupancy certificate	15 days

3.5.2 The Building Official shall notify the applicant according to above table as the case may be either approval or refusal of the permit for any work. If the Building Official does not notify the applicant of such approval or refusal within this specified period, the application shall be deemed to have been approved provided the fact is brought to the notice of the Building Official. Such approval shall not be interpreted to authorize any person to do anything in contravention of or against the terms of lease or titles of the land or against any other regulations, bylaws or ordinance operating on the site of the work or any of the provisions of this Code.

3.5.3 Refusal of permit shall be accompanied with reason and the Building Official shall quote the relevant sections of this Code which the application/drawings/submissions contravene. The applicant may correct or remove such reasons and reapply for permit with any fee if applicable. The Building Official shall scrutinize the re-submitted application and if there be no further objection it shall be approved and permit issued.

3.6 Preparation And Signing of Drawings

3.6.1 All drawings submitted for approval shall be prepared and signed by registered professionals as specified in Table 2.3.4, which shall be considered as equivalent to certifying that the drawing on which the signature appears conforms to all the requirements of this Code. Registered Professionals shall put his or her signature with date on the title box of the drawing along with his name, address, professional society membership number, registration number and any other information required by the concerned Building Official.

3.6.2 The drawings shall also contain the signature, name and address of the owner.

3.6.3 Subject to the classification and use of buildings, all drawings for approval and execution shall be prepared and signed by the registered professionals as per building category specified in Tables 2.3.3 and 2.3.4 corresponding to relevant work.

Table 2.3.3: Building Classification Based on Height, Floor Area and Occupancy Type

Building Category	Height of Building	Floor Area	Type of Occupancy
I	Up to 2 Stories or 8 m height (without basement) applicable only for areas beyond the jurisdiction of Development Authority, City Corporation and Pourashava	Up to 250 m^2	A (A1-A2)
II	Up to 5 Stories (with or without basement)	Up to 1000 m^2	A (A1-A5)
III	Up to 10 stories or 33 m height for engineering design and supervision and any height for land survey, sub-soil investigation and architectural design	Up to 7500 m^2	A, B, C, E1, E2, F1, F2 and H1
IV	Any height	Any Size	All Occupancy Type

Table 2.3.4: Eligible Registered/Licensed Professionals for Signing of Design, Drawings, Reports and Documents

Types of Work	Registered Professional	Minimum Experience Requirement in Years for Building Category			
		I	II	III	IV
Land Survey	Civil Engineer	NA	NR	NR	NR
	Planner		NR	NR	NR
	Diploma Engineer (Civil)		3	3	3
	Certified Surveyor		3	3	3
Soil Investigation Report	Geotechnical Engineer having experience in soil investigation and soil test report analysis	NA	NR	NR	NR
	Civil Engineer having experience in soil investigation and soil test report analysis	NA	2	2	5

Types of Work	Registered Professional	Minimum Experience Requirement in Years for Building Category			
		I	II	III	IV
Architectural Design	Architect	NA	NR	2	8
	Civil Engineer	NA	NR	NE	NE
	Diploma Architect	NA	5	NE	NE
Structural Design	Civil Engineer with experience in structural design or PEng.	NA	2	4	8 (having 5 years in Structural design)
	Civil Engineer with M.S in Structural Engineering	NA	1	3	8 (having 4 years in Structural design)
Plumbing Design	Plumbing Engineer	NA	NR	4	8
	Architect	NA	NR	NE	
	Diploma Engineer (Civil)	NA	3	NE	NE
Mechanical (HVAC/Vertical Transportation) Design	Mechanical Engineer	NA	2	4	8
Electrical Design	Electrical Engineer	NA	2	4	8
	Diploma Engineer (Electrical)	NA	3	NE	NE
Construction Supervision	Architect/Engineer in their respective field or PEng.	NA	2	4	8
	Diploma Architect/Diploma Engineer in their respective field	NA	2	4	20*
Building Demolition	Civil Engineer	NA	NR	2	8
	Diploma Engineer (Civil)	NA	2	NE	NE
Completion Report	Architect and Engineer with experience in their respective field	NA	2	4	8

Note: NA: Not Applicable, NE: Not Eligible, NR: Not Required.

*Shall be countersigned by registered/licensed Architect/Engineer eligible for Building Category IV.

3.7 Fees

All applications shall be accompanied by fees as specified by the authority from time to time without which the application shall be deemed to be incomplete.

3.8 Responsibilities and Duties of The Owner

3.8.1 General

The owner of a building or structure regulated by the provisions of this Code shall be responsible for carrying out the work in conformity with the provisions of this Code. Granting of permission for any work or approval of plans or inspection by the Building Official or any of the deputies shall not relieve the owner from such responsibility.

3.8.2 Employment of Technical Personnel

Design, execution and supervision work of any building shall be carried out by authorized Registered Professionals as outlined in Table 2.3.4. Owner shall take the services of as many professionals as required according to type and size of the work.

3.8.3 Right of Entry

The owner shall allow the Building Officials to enter the site for the purpose of enforcing the Code as required by the provision of Sec 2.9.6 and for the purpose of inspection as provided in Section 3.10 below.

3.8.4 Permits from Other Agencies

The owner shall obtain permit as may be applicable from other concerned agencies relating to building, zoning, grades, sewers, water mains, plumbing, fire safety, signs, blasting, street occupancy, gas, electricity, highways and all other permits required in connection with the proposed work.

3.8.5 Information on Progressive Work

The owner shall inform the Building Official about attainment of construction work of different stages as required by the Building Official in prescribed form.

3.8.6 Safety Measures

The owner shall take proper safety measures in and around the construction site.

3.8.7 Notice of Completion

The owner shall notify the Building Official the completion of the work for which permit was granted in prescribed form. The work shall not be accepted as complete, without a certification from the Building Official.

3.8.8 Documents at Site

The owner shall preserve at the site a copy of all permits issued and all drawings approved by the Building Official. Results of tests carried out for determination of conformity of the work with the provisions of this Code shall also be preserved and made available for inspection during execution of the work.

3.8.9 Live Load Posted

Where the live loads for which each floor or portion thereof of a commercial or industrial building is or has been designed to exceed 2.4 kN/m^2, such design live loads shall be conspicuously posted by the owner in that part of each storey in which they apply, using durable signs. It shall be unlawful to remove or deface such notices.

3.9 Responsibilities and Duties of Technical Personnel

3.9.1 To qualify as Architect, Engineer, Construction Supervisor (Architect or Engineer or Diploma Architect or Diploma Engineer) of any building works one shall have membership of the respective professional body in the country. In addition they shall have to qualify as registered professional through an examination (written/oral) to be conducted by their respective professional body as per requirement of this Code.

3.9.2 Only technical professionals qualified under Sec 3.9.1 shall design, execute and supervise any building which is subjected to approval granted under this Code.

3.9.3 Any lapses on the part of the technical personnel in delivering the requirements of the Code shall call for punitive actions against him/her in the proper forum.

3.10 Inspection

All works relating to a building or structure regulated by the provisions of this Code for which permits are required shall be subject to inspection by the Building Official. Modalities and frequency of such inspections shall conform to the requirements put forward by the approving authority.

3.11 Unsafe Buildings

3.11.1 General

All buildings considered to constitute danger to public safety or property shall be declared unsafe and shall be repaired or demolished as directed by the Building Official.

3.11.2 Examination

The Building Official shall examine or cause examination of every building reported to pose threat to safety or be damaged by wear and tear or accident and shall make a written record of such examination.

3.11.3 Notification

If a building is found to be unsafe, the Building Official shall notify the owner of the building and specify the defects thereof. The notice shall require the owner within a stated time either to complete the required repair or improvement or demolish and remove the building or portion thereof.

3.11.4 Disregard of Notice

In case the owner fails, neglects or refuses to carry out the repair or improvement of an unsafe building or portion thereof as specified in the notice, the Building Official shall cause the danger to be removed either by demolition or repair of the building or portion thereof or otherwise, the cost of which shall be borne by the owner.

3.11.5 Cases of Emergency

If the Building Official considers that an unsafe building or structure constitute imminent danger to human life or health or public property, the Building Official shall at once or with a notice as may be possible promptly cause such building or structure or portion thereof to be rendered safe or removed. In such cases the decision of the Building Official shall be final and binding and he or any of his assigned deputies may at once enter such structure or land on which it stands or the abutting land or structure, with such assistance from and at such cost to the owner as may be deemed necessary. The Building Official may also get the adjacent structures vacated and protect the public by an appropriate fence or such other means as may be necessary.

3.12 Demolition of Buildings

If a building or structure is to be demolished, the owner shall notify all agencies providing utility services to the building. Such agencies shall remove all their appurtenances and equipment and dismantle all service connections to ensure a safe condition. The Building Official shall not grant any permit for demolition of a building until a release is obtained from the utility services stating that all service connections have been removed in the proper manner. The demolition work shall be done under the supervision of demolition expert as per provisions of Table 2.3.4.

3.13 Validity of This Code

3.13.1 Partial Invalidity

In case any provision of this Code is held to be illegal or void, this shall have no effect on the validity of any other provision of the Code nor on the same provision in different cases nor on the Code as a whole, and they shall remain effective.

3.13.2 Invalidity of Existing Buildings

If any provision of this Code is held to be illegal or void by the Authority as applied to an existing building or structure, validity of that provision or any other provision of the Code in its application to buildings hereafter erected shall not be affected.

3.14 Architectural and Environmental Control

3.14.1 Besides enforcing the provisions of this Code for normal buildings and structures, the Building Official shall, for special structures such as those listed in Sec 3.14.2 below, also examine the aesthetics and environmental issues vis-a-vis the existing structures and the characteristics of the area, and exercise architectural and environmental control in accordance with the provisions of this Section.

3.14.2 Special structures for which architectural and environmental control shall be exerted by the Building Official shall include:

(a) major public building complexes

(b) buildings in the vicinity of monuments and major sculptures

(c) buildings and structures near existing structures identified to be architecturally valuable.

(d) buildings and structures near historic buildings or in a area of historical or archaeological significance.

(e) buildings near any structures that represents the special characteristics of an area

(f) any proposed building or structure that represents the special characteristics or forms part of a larger master plan of an area, and

(g) any development that may have an effect on the environment or characteristics of an area.

3.14.3 The Authority shall, for the purpose of exercising the architectural and environmental control and for identifying existing structures having architectural value, appoint a standing committee comprising noted experts from the fields of Architecture, Archeology, Planning, History, Art, Literature, Engineering or any other discipline which may be deemed relevant. The committee shall examine the aesthetic quality of the proposed building, structure or development and the effect it may have on the characteristics and environment of the area in order to ensure aesthetic continuance of the new structure with the existing ones and aesthetic blending of the new structure with the surroundings. The committee may require additional drawings and information for a detailed study of the proposed work. The committee for the purpose of arriving at their decision, may at their discretion depending on the magnitude of the project and impact it may have on public life, hear the architect of the proposed work who may wish to explain the various features of the project, note comments of other experts in the relevant disciplines, or in exceptional circumstances, institute a public hearing to assess public reaction to the project.

3.14.4 The committee may approve the proposed work, recommend changes in the scheme, or disapprove the scheme, for reasons of aesthetics and environmental control.

3.14.5 The Building Official shall not issue permit for undertaking the proposed work until obtaining a report from the standing committee stating that the intended work is acceptable in respect of its effect on the environment, landscape, architectural characteristics, historical feature or any other aesthetical quality of the locality, area or landscape concerned.

3.15 Making Implementation Procedures

Detailed byelaws and implementation procedure to enforce the provisions of this Code shall be prepared and published by the relevant authorities.

3.16 List of Related Appendices

Appendix A Form for Application of Land Use/Development/Building Permit

Appendix B Form for Certificate of Supervision

Appendix C Form for Sanction or Refusal of Land Use/Development/Building Permit

Appendix D Form for Appeal against Refusal of any Permit

Appendix E Form for Completion Certificate

Appendix F Form for Occupancy Certificate

PART II
Appendix A

(Position and Address of the Building Official)

Form for Land Use/Development/Building Permit
First Application to Develop, Erect, Demolish or to Make Alteration in any Part of the Building

Type of intended work: Develop ☐ Erect ☐ Demolish ☐ Alter ☐

(Check one)

Name of the owner: ______________________________
Contact address: ______________________________
Post code: ______________
Telephone no: ______________

Name, address and qualification of the engineer, architect or planner involved in the proposed work:

For planning: ______________________________
For architectural design: ______________________________
For structural design: ______________________________
For civil works design: ______________________________
For other services design: ______________________________

<u>Address of the site</u>

Plot number: ______________ Holding number: ______________
Dag/Khatian number: ______________ Mouza/Block/Sector: ______________
Street name: ______________ Municipal ward number: ______________

Documents enclosed along with this form:

	Name of document	Number of sheets	Number of copies
1.	Key plan	_____	_____
2.	Site plan	_____	_____
3.	Subdivision/layout plan	_____	_____
4.	Building plan	_____	_____
5.	Services plan	_____	_____
6.	Specifications	_____	_____
7.	Ownership title	_____	_____

______________ ______________________

Date Signature of the owner

For use of the Building Official. Do not write anything below this line.

Reference number: ______________________ Date: ______________
(To be referred to in all subsequent correspondences)
Received by: ______________________

PART II
Appendix B

Form for Certificate of Supervision

Reference number: ___________________________

Address of the site:

Plot number: _______________________________ Holding number: ____________________

Street name: _______________________________

Municipal ward number: ____________________

Type of intended work (Check one) :

Develop ☐ Erect ☐ Demolish ☐ Alter ☐

Name of the owner: ____

Contact address: ____

____

____

Post code: _____________________

Telephone no: _____________________

I hereby certify that the building for which the location, the type of work, and the name and address of owner appear above will be supervised by me as per the provisions of the Bangladesh National Building Code.

____
Signature of the engineer, architect, planner or supervisor

____
Name of the engineer, architect, planner or supervisor

____
Address

___________ _________________
Qualification Date

PART II
Appendix C

(Position and Address of the Building Official)
Form for Sanction or Refusal of Land Use/Development/Building Permit

Reference number: ____________________________

In response to your application whose reference number appears above, I hereby inform that the documents submitted along with your application have been (check as appropriate)

☐ Approved for implementation by the Authority

☐ Refused by the Authority for violation of the following provisions of the Bangladesh National Building Code:

(List of the sections violated)

____________________	____________________
Signature of the Officer	Permit number
____________________	____________________
Name of the Officer	Official stamp
____________________	____________________
Designation	Date

PART II
Appendix D

Form for Appeal against Refusal of any Permit

Reference number: ____________________________

The application whose reference number appears above has been refused by the Authority. I hereby appeal against the refusal for the following reasons.

(List of the justifications for the appeal)

________________	___________________________
Date	Signature of the owner

For use of the Building Official. Do not write anything below this line.

Received by: ______________________________

Date: __________________

PART II
Appendix E

Form for Completion Certificate

Reference number: ________________________
Permit number: __________________________
Address of the site:
Plot number: ________________________ Holding number: ___________________
Dag/Khatian number: _________________ Mouza/Block/Sector: _______________
Street name: ________________________ Municipal ward number: ____________
Documents enclosed along with this form:

____
____

Type of work (Check one)

Develop ☐ Erect ☐ Demolish ☐ Alter ☐

Name of the owner: ______________________________________
Contact address: ______________________________________
____
____
Post code: ____________________
Telephone No: ____________________

I hereby certify that the work having the above mentioned detailed particulars has been supervised by me and completed in accordance with the plan and design approved by the permit number cited and the provisions of the Bangladesh National Building Code.

____
Signature of the engineer, architect, planner or supervisor

____
Name of the engineer, architect, planner or supervisor

____
Address

____ ___________________
Qualification Signature of the owner

________________________ ___________________
Date Date

This Part to be completed by the Building Official.

The work identified by the reference number and permit number at the top of the form is hereby accepted as complete in accordance with the approved plan and design.

____
Signature of the Officer

________________________ ___________________
Name of the Officer Official stamp

________________________ ___________________
Designation Date

PART II
Appendix F

(Position and Address of the Building Official)

Form for Occupancy Certificate

Reference number: ________________________

Permit number: __________________________

Address of the site:

Plot number: ____________________________ Holding number: _____________

Dag/Khatian number: ____________________ Mouza/Block/Sector: __________

Street name: _____________________________ Municipal ward number: ________

Documents enclosed along with this form:

____

____

Type of work (Check one)

Develop ☐ Erect ☐ Demolish ☐ Alter ☐

Name of the owner: ____

Contact address: ____

____

____

Post code: ____________________

Telephone no: ____________________

I hereby certify that the work having the above mentioned detailed particulars has been completed in accordance with the plan and design approved by the permit number cited and the provisions of the Bangladesh National Building Code. The owner has submitted all the required documents for issuance of occupancy certificate. Thus, I, hereby, certify that the holding may be occupied as ____________________________________ (mention the occupancy type).

____
Signature of the Officer

____
Name of the Officer

____
Official stamp

____
Designation

____
Date

PART III
Chapter 1
General Building Requirements

1.1 Scope

This Part of the Code puts forward classification of buildings based on occupancy or nature of use and deals with the general and specific requirements of each of the occupancy groups. Fire resistance requirements are expressed in terms of type of construction which shall conform to the specified fire-resistive properties.

1.2 Terminology

This Section provides an alphabetical list of the terms used in and applicable to this Part of this Code. In case of any conflict or contradiction between a definition given in this Section and that in any other Part of this Code, the meaning provided in this Part shall govern for interpretation of the provisions of this Part.

Accessibility	The provision in a plot or a building or a facility or any part thereof that can be approached, entered and used without assistance by persons with temporary or permanent physical limitations.
Accessibility Route	A continuous unobstructed path that starts from the entry and shall continue through all accessible elements and spaces within a plot and buildings or facilities thereof up to the exit termination.
Accessible	The term accessible or adaptable shall be used as a prefix for spaces or features which are designed for persons having physical limitation; such as accessible toilet, accessible kitchen, accessible lift, and so on.
Adaptable	See ACCESSIBLE
Area Planning Authority	A government or semi-government agency or a local body which has been legally designated to formulate land use or plans of the area under their jurisdiction.
Assembly	In a building or a portion thereof used for gathering of 50 or more persons for deliberation, worship, reading, entertainment, eating, drinking, awaiting transportation, or similar uses not limited to these; or used as a special amusement building, regardless of occupant load.
Atrium	A large volume space within a multistoried building having series of floor openings or corridors or similar elements in and around and floors are connected from there and series of openings or a glazing on roof or a portion thereof constructed with glazing and having a minimum two stories high. The word Atria or Atriums are the plural form of Atrium.

Balcony	A covered and hanging platform at a height of minimum 2.286 m from the plinth level of a building and having access from any floor level and which is laterally open to outer air by three sides up to 2.06 m in height and edges are protected with guards. Within an interior space, a balcony is a portion which are positioned sidewise as similar as Mezzanine.
Baluster	Single vertical member of a guardrail or a Handrail or a member of both which shall be complied with the provisions of this Code.
Balustrade	Plural form of BALUSTER.
Barrier	A wall or a partition or a floor slab or a ceiling within a building which confines and protects flow of smoke and fire from the exposed side of the barrier. The fire rating of barriers shall be complied with the provisions of this Code.
Basement	A floor of a building or a portion thereof which is situated as a whole or partially at depth of minimum 50 percent of ceiling height below formation level shall be called as a basement.
Building Line	The peripheral lines of a building mass or volume up to which the plinth area or any floor area may be lawfully extended within a plot.
Carriageway	A path including over bridge or bridge which is open to the outer air and may or may not be covered or roofed or an underpass, design and designated for vehicles only.
Ceiling Height	Height measured from the top of finished surface of floor level up to the bottom of roof or ceiling or suspended or false ceiling level or Beam drops. In case of multistoried building, Vertical distance in between two slabs from which deduction shall be made for any suspended or false ceiling or Beam drops. For slope or pitch ceiling or roof, the minimum value shall be the ceiling height.
Common Space Condition	See NON-SEPARATED SPACE CONDITION
Control Area	A space or a room within a building enclosed by barriers with the fire rated walls, floor and ceiling, where the quantity of hazardous material shall not be exceeded the maximum allowable quantity per control area for storing, displaying, handling, dispensing or using as per provisions of this Code.
Detached Occupancy	A building separated by distance in a same plot to accommodate different type of occupancies shall be termed as Detached Occupancy.

Development Authority	A government or semi-government agency or a local body which has been legally designated to carry out and/or control any works of land development of an area having jurisdiction.
Far (Floor Area Ratio)	FAR is a ratio between the area of a plot and the sum of floor areas of building or buildings are erected or intended to be erected thereof. In the buildings, there may have some specific and calculated floor areas which shall be treated as bonus or exempted from the total floor area calculation and such areas shall be specified by the authorities having jurisdiction.
Fire	An uncontrolled fire which poses threat to safety of life or property or both.
Fire Separation Distance	A minimum distance which to be maintained between potential sources and/or between structures for fire safety. In case of differences between building setback and the required minimum fire separation distance measurement; the higher value shall be implied.
Flood	A Land or a plot normally dry but submerges or drowns as whole or partially by over flown water from any source.
Flood Level	A measurement of height from an existing ground level or from top level of river water of an area or a locality recorded in a Flood Hazard Map by the authorities having jurisdiction.
Flood Prone Area	At least once in a year a dry ground of an area or a plot or a portion thereof flooded at a height of 1m or more shall be designated as a Flood Prone Area.
Floor Height	In a multistoried building, floor height shall be measured from the top of finished surface of the two successive floor slabs and the measurement of the top most floor shall be from the top of finished surface of the floor slab and the top of the finished roof, in case of the slope roof, measurement shall be taken up to pick of that slope.
Frontage	Irrespective of the entry provision to a plot, full or partial length of any sides of a plot which are abutted to roads or streets shall be designated as frontage.
Formation Level	Finished ground level of a plot. For hilly areas formation levels shall be the gradient of the plot surface.
Gallery	A special type of seating arrangement where each and every row or tier of seats are successively elevated to provide a clear view to audiences or spectators within and around a playground or outdoor or indoor stadium or within an auditorium or in a hall.

Guard	A vertical protective barrier erected up to a height along exposed edges of stairways, balconies and similar areas.
Head Room Clearance	A vertical distance measured from the top of finished floor level up to the bottom of ceiling or lowest roof level or bottom of beam drop or bottom of any hanging element within a space. In case of a stairway, a vertical distance measured from the bottom surface of flight or ceiling or beam drop to any outer edge point of a tread below and for the landings ceiling height measurement system shall be adopted to determine head room clearance.
Helistop	A designated area on ground or on water or on a portion of a building for helicopter landing or takeoff without servicing, repairing and refueling facilities.
High Rise Building	Any building which is more than 10-storey or 33 m high from reference datum. Building appurtenances like overhead water tank, machine room, communication tower etc. will not be considered in determining the height.
Lighting Shaft	A space within a building which is fully enclosed by all sides and shall be open to the sky to provide daylight to adjacent interiors and less than the dimensions that stipulated for minimum closed or internal courts of corresponding to the building heights.
Loft	An intermediate space in-between a floor or a ceiling and under a pitch or a slope roof of a building.
Mandatory Open Space	The spaces within a plot which shall remain unpaved with or without vegetation to allow water penetration and uncovered up to the sky from formation level of the building. No underground or above ground construction is allowed in such spaces.
Mezzanine Floor	Within one space where more than one floor exists, the floor at the lowest level shall be designated as main floor and each Intermediate floor is limited to an area which is not more than one third of the main floor under one roof or one ceiling, thus gives two or more useable floor levels. These types of intermediate floors shall be designated as mezzanine floors. Mezzanine floor may be as gallery or flat floor type and which also includes interior balcony.
Mixed Occupancy	When two or more occupancies are amalgamated in a building shall be termed as Mixed Occupancy.
Non Separated Space Condition	Walls or partitions between compartments, rooms, spaces or areas within a building or part of a building which are not separated by an approved fire rated barrier walls or partitions shall be designated as non-separated space condition or effective undivided single space.

Openings	Apertures or holes in any wall of a building that allow air to flow through and which are designed as open.
Opening, Vertical	An opening through a floor or roof of a building.
Open Space	Open space within a plot includes all spaces other than spaces covered by the Maximum Ground Coverage (MGC).
Plinth	Bases of the building and the elements that negotiate with the ground.
Plinth Level	Height of a covered finished floor which is just above the formation level and measured from the formation level up to the top of that finished floor.
Plinth Area	The elements from the building bases which are exposed above the formation level to form a covered floor area by joining the peripheral points of the elements which are intersected finished floor plane at the height of plinth level shall be designated as Plinth Area.
Plot	A scheduled piece or parcel of land which is classified and restricted to its intended use.
Ramp	A sloping walkway which is steeper than 1 in 20 but not steeper than 1 in 8 and shall have guard and handrail.
Ramp, Accessibility	A sloping walkway not steeper than 1 in 12.
Ramped Driveway	Ramped Driveways are inclined floors that provide access to vehicles between two levels. Ramped walkway when provided side by side of a ramped driveway shall be separated by safety guard rails and curbs. A sloping driveway or Ramped Driveway steeper than 1 in 8 shall not be credited as a component of means of exit.
Ramp Gradient	Ramp gradient refers to the ratio of the inclination of a ramp (height by length ratio) measured along the center line of the ramp.
Road Level	The road level means top surface at the center point of the road width which is used for site entry and shall be considered as the reference point for measuring height or depth of any development.
Roof	Weather exposed and uncovered surface of the topmost or the terminal ceiling of a building which may be horizontal or pitched or may have slopes shall be treated as the roof of a building.

Separated Occupancy	A building or a portion thereof separated by barriers with wall or ceiling slab that into two or more parts to accommodate different type of occupancies in different parts.
Separate Space Condition	Rooms, spaces or areas within a building when separated by approved barrier wall.
Separation Wall	This is a peripheral wall of a building or a building which shall be divided into two or more or a common wall between two buildings to control spreading of fire as per provisions of this Code.
Site	See PLOT
Smoke Draft Barrier	A vertical panel dropped from the ceiling of a building or portion thereof to protect and control the movement of smoke draft during fire. The construction of such smoke draft barriers shall be complied with the provisions of this Code.
Stage	An elevated platform which is designed or used for presentation of plays or lectures or other entertainments in front an assembly of spectators or audiences.
Stage, Interior	An elevated platform within a building which is designed or used for presentation of plays or lectures or other entertainment in front an assembly of spectators or audiences.
Stage, Legitimate	Ceiling Height of a stage from the top surface of the platform is 15.24 m or more shall be designated as a legitimate stage.
Storage Density	A storage or display of solid or liquid merchandises shall not be exceeded 976 kg/m^2 or 814 L/m^2 respectively and shall be limited to the exempted quantity of an actually occupied net floor area. Maximum height of display or storing of merchandises shall not be exceeded 1829 mm or 2438 mm respectively. Allowable Height and Quantity may be less depending on the total area and the ceiling height of a store or a display.
Street Or Road	An open to outer air and unobstructed space having required width and used by the public as pedestrian or walkway, or animal or vehicular movement or any combination of these for the purpose of access to a plot or plots and is connected with the national public transportation system other than railway track shall be designated as street or road which may or may not be paved.
Street Or Road Width	The width of any street or road shall be measured form any plot to its opposite or face to face plot distance. For the determination of a road width, measurements shall be taken up to the connection of the national public transportation system other than railway track from any plot and the least width shall be the road width.

Street Floor Level	A story or floor level of a building which is accessible at the main entrance of a building from the street or from the outside at ground level and the floor shall not be more than three risers above or below the grade level.
Structural Frame	All members or elements such as columns, girders, beams, trusses and spandrels which forms a frame and have direct connections with bearing and transferring as an integral and essential elements for the stability of a building or a structure as a whole.
Surge Prone Area	Expected occurrence of a surge or wave of water may flow above 1 m or higher from the formation level.
Tall Structure	A building used for human occupancy located more than 80m high from the center of the adjacent road level or from lowest level of the fire department vehicle access.
Terrace	A paved surface not steeper than 1 in 20 and adjacent to a building which is connected by a stairway or a walking ramp or at the same level of any floor below the roof level of a building and at least one side of that area is exposed to the weather and having the guards and open to the sky.
Universal Accessibility	See ACCESSIBILITY
Unprotected	The element that shall have no prerequisites of fire protection rating.
Ventilation Shaft, Natural	A space sidewise enclosed but open to sky used to provide ventilation as inlet and/or outlet to adjacent interiors of dimensions less than that stipulated for internal courts of corresponding to building heights.
Verandah	Portions of a building at any level which have ceiling or roof and at least one side open up to 2.13 m height to the outside air and have guards as per provisions of this Code.
Walkup Building	A multi storied building which does not have any mechanical means of vertical circulation other than stairway shall be designated as a walkup building and the maximum height of the walkup building shall be as per provision of this Code or as approved by the authority having jurisdiction.

1.3 Land Use Classification

A city or a township or a municipality or a union or any other habitat development shall be brought under a structured planning including detailed area planning to implement the intended land use pattern, transportation and maintaining environmental conditions by the development or planning authorities and shall be approved by the government. This land use classification may divide an area into zones such as residential, commercial, industrial, storage, green park, agricultural land, reserved area etc. or any combination of these. The land use zones shall be shown on the approved master plan of the area and the planning regulation shall clearly state the permitted occupancies, restricted occupancies and conditionally permitted occupancies for each zone.

1.4 Occupancy and Construction Classification of Buildings

Every building or portion there of shall be classified according to its use or character of occupancy. A brief description of such occupancy groups is presented in Table 3.1.1. Details of all occupancy group and sub-divisions are set forth in Sec 2.1 of Chapter 2 of this Part. Types of construction based on fire resistance are specified in Table 3.1.2. Details of such types of construction are set forth in Chapter 3 of this Part. Any development permit for a site or a location shall clearly mention the permitted occupancy and construction type in accordance to Tables 3.1.1 and 3.1.2 for the existing or proposed building.

Table 3.1.1: Summary of Occupancy Classification

Occupancy Type	Subdivision	Nature of Use or Occupancy	Fire Index*
A: Residential	A1	Single family dwelling	1
	A2	Two families dwelling	1
	A3	Flats or apartments	1
	A4	Mess, boarding houses, dormitories and hostels	1
	A5	Hotels and lodging houses	1
B: Educational Facilities	B1	Educational facilities up to higher secondary levels	1
	B2	Facilities for training and above higher secondary education	1
	B3	Pre-school facilities	1

Occupancy Type	Subdivision	Nature of Use or Occupancy	Fire Index*
C: Institution for Care	C1	Institution for care of children	1
	C2	Custodial institution for physically capable adults	1
	C3	Custodial institution for the incapable adults	1
	C4	Penal and mental institutions for children	1
	C5	Penal and mental institutions for adults	1
D: Healthcare Facilities	D1	Normal medical facilities	2
	D2	Emergency medical facilities	2
E: Business	E1	Offices	2
	E2	Research and testing laboratories	2
	E3	Essential services	2
F: Mercantile	F1	Small shops and market	2
	F2	Large shops and market	2
	F3	Refueling station	2
G: Industrial Buildings	G1	Low hazard industries	3
	G2	Moderate hazard industries	3
H: Storage Buildings	H1	Low fire risk storage	3
	H2	Moderate fire risk storage	3
I: Assembly	I1	Large assembly with fixed seats	1
	I2	Small assembly with fixed seats	1
	I3	Large assembly without fixed seats	1
	I4	Small assembly without fixed seats	1
	I5	Sports facilities	1
J: Hazardous Building	J1	Explosion hazard building	4
	J2	Chemical hazard building	4
	J3	Biological hazard building	4
	J4	Radiation hazard building	4

Occupancy Type	Subdivision	Nature of Use or Occupancy	Fire Index*
K: Garage	K1	Parking garage	2
	K2	Private garage	1
	K3	Repair garage	3
L: Utility	L	Utility	2
M: Miscellaneous	M1	Special structures	2
	M2	Fences, tanks and towers	1

* Fire Index: fire index is an absolute number, Occupancy group having same fire index may be permitted as mixed occupancy and different fire index shall be separated or detached as per provisions of this Code.

Table 3.1.2: Summary of Classification of Buildings Based on Types of Construction

Construction Group	Construction Type	Description
Group I: Non-combustible	Type I-A	4 hour protected
	Type I-B	3 hour protected
	Type I-C	2 hour protected
	Type I-D	1 hour protected
	Type I-E	Unprotected
Group II: Combustible	Type II-A	Heavy timber
	Type II-B	Protected wood joist
	Type II-C	Unprotected wood joist
	Type II-D	Protected wood frame
	Type II-E	Unprotected wood frame

1.5 Requirements of Plots

1.5.1 General Requirements

1.5.1.1 No building shall be constructed on any site which is water logged, or on any part of which is deposited refuse, excreta or other objectionable material, until such site has been effectively drained and cleared to the satisfaction of the Authority.

1.5.1.2 Provision shall be kept for any space within the plot left vacant after the erection of the building to be effectively drained by means of surface or underground drainage system.

1.5.1.3 Basic minimum sanitary waste and excreta disposal facility shall be created on the premises, whether or not the plot is served by a disposal system provided by any utility service authority or agency.

1.5.1.4 Written approval of the Authority or the appropriate drainage and sanitation authority shall be obtained for connecting any soil or surface water drain to the sewer line.

1.5.2 Clearance from Overhead Electric Lines

A building or any part thereof shall not be erected within, nor any auxiliary part of the building be allowed to come closer to the distance shown in Table 3.1.3 from any overhead electric line.

Table 3.1.3: Minimum Distances from Overhead Electric Lines

Line Voltage	Vertically (m)	Horizontally (m)
Low to medium voltage lines and Service lines	2.5	1.25
High voltage lines up to 33 kV	3.5	1.75
High voltage lines beyond 33 kV	3.5 plus 0.3 for each additional 33 kV or part thereof.	1.75 plus 0.3 for each additional 33 kV or part thereof.

1.5.3 Road Level, Formation Level and Plinth Levels

1.5.3.1 Road level shall be lower than the habitable formation level of an area, except that of a hilly region. When a road is designed and designated as a part of national disaster management system, formation levels shall be determined by the authorities having jurisdiction.

1.5.3.2 The formation level of a plot shall not be lower than the adjacent road levels, except that of a hilly region. For hilly region, the elevation of the formation level shall be determined by the authority having jurisdiction. Where areas are not susceptible to flood or water logging, the formation level shall not be more than 450 mm high from the surface level of the center line of the adjacent roads.

1.5.3.3 The plinth level of a building shall be at least 450 mm above the surface level of the center line of the adjacent road. In Flood or Surge prone area plinth level shall be determined by the development authority having jurisdiction.

1.5.4 Boundary Wall

1.5.4.1 Solid boundary walls of a plot or in between plots shall not be higher than 1.5 m or a boundary made of grill, screen, balustrade etc. with a maximum height of 2.75 m shall not require the permission of the Authority. For boundary walls made of a combination of solid wall and grill or screen, the solid wall portion shall not be higher than 1.5 m. The Authority may, on specific application, permit the use of higher boundary walls.

1.5.4.2 Construction of a boundary wall shall be capable to resist collapsing as per provision of this Code.

1.6 Plot Sizes

Plot divisions and plot sizes are part of integrated planning decision of detail area plan and shall be determined by the Area Development Authority having jurisdiction. Where no such guideline exists or yet to be undertaken, the criteria mentioned in Sec A.5 of Appendix A regarding plot size shall be applicable.

1.7 Means of Access

The provision of means of access is implied on an area or a plot when more than one plots are intended to be created or when more than one buildings are intended to be erected respectively, where such plots or buildings do not have frontage to or not approachable by a public or a private road or street. All buildings within such area or a plot shall have access facilities which shall be connected with national road transportation system. The components of means of access shall comply with the followings:

(a) The access facilities shall meet the requirements of fire service vehicles and engines movement for rescue and fire extinguishment operation.

(b) Where required for fire apparatus access roads shall have an unobstructed carriageway width of 4.8 m and the minimum vertical clearance shall be 5m. The width and vertical clearance of fire apparatus access roads may be increased as per requirement of the fire authority, if the clearances are not adequate to provide fire apparatus access.

(c) Access roads longer than 30 m having a dead end shall be provided with appropriate provisions for turning around of the fire apparatus at the dead end.

(d) The provision of fire apparatus stall be marked by approved sign.

(e) For large Assembly Occupancy of I1, I3 and I5, width of the approach road shall not be less than 15 m.

(f) The minimum width of the approach road for all plots other than residential and assembly occupancies mentioned in Sec 1.7(e) and Sec 1.7(g) shall be 10.8 m.

(g) For area fully covered by private hydrant system with street side hydrant points and/or hydrants within the building equivalent to fire service and civil defense department's specification and the buildings have fire stairs as per provisions of this Code, the requirements of Sections 1.7(a), (b) and (c) may be exempted. This provision shall not be applicable for planning new developments. The minimum width of access roads for plot divisions in new developments shall follow guidelines of Table 3.F.1 of Appendix F.

1.7.1 Internal Access Road

Internal access road is legally restricted for thoroughfare to the citizens and/or reserved for a group of people of a plot or an area that shall have access provisions for the department of fire service and civil defense.

1.7.1.1 The width of access roads and drive ways in a plot or an area shall be decided by the number and height of the buildings served thereby.

Table 3.1.4: Maximum Permissible Length of Internal Access Roads in Non-Residential Plots

Width (m)	Maximum Permissible Length (m)
6	80
7	150
8	300
10.8 or more	Unlimited

1.7.2 Pedestrian Path or Walkway or Footpath

Any path including over bridge or bridge which is open to the outer air and may or may not be covered or roofed or an underpass design and designated for walkers only shall be designated as pedestrian path or walkway or footpath.

1.7.2.1 An uncovered paved pedestrian path that links buildings and the approach road shall not be included as a floor area of a building.

1.7.2.2 The walkways shall not be used for any other purpose than pedestrian movement and as accessibility route.

1.7.2.3 The minimum width of the pedestrian path shall not be less than the calculated width of connected corridor or passage or walking ramp of a building for entry or exit provided it is not enclosed by adjacent walls on both sides; for pedestrian paths enclosed by adjacent walls on both sides the minimum width shall be 1.25 m. For public buildings and places where high pedestrian movement is expected, Table 3.F.1 of Appendix F may be followed.

1.7.2.4 Pedestrian walkways as accessibility route in public buildings shall comply with the provisions of this Code. Any changes in elevation in accessibility route shall comply with the provisions of Appendix D (Universal Accessibility).

1.8 Open Spaces Within a Plot

1.8.1 Minimum open space requirements for the sides, rear and frontages of a plot shall be as per the provisions of this Code or the authority having jurisdiction. In absence of such guideline, provisions of Sec 1.8.2 to Sec 1.8.11 shall decide the provisions of open space for any building or buildings within a site. All such open spaces shall ensure access of the users.

1.8.2 At least 50 percent of the minimum open space in a plot shall remain unpaved with or without vegetation to allow water penetration.

1.8.3 The total open area in a plot on which a building of educational, institutional, health care occupancy is constructed shall not be less than 50 percent of the plot area.

1.8.4 The total open area in a plot on which a building of any occupancy, except those mentioned in Sec 1.8.3, is constructed shall not be less than 33 percent of the plot area.

1.8.5 For the purpose of Sec 1.8.2, Sec 1.8.3 and Sec 1.8.4, the total open area shall include all exterior open spaces and interior courtyards, but exclude the area of any lighting and ventilation shaft.

1.8.6 For approved row type or cluster type housing or site and service schemes, the requirement of Sec 1.8.3 shall be applicable.

1.8.7 Separation of Buildings in the Same Plot

1.8.7.1 More than one building in a plot shall comply with the requirements of means of access and setback distances in relation with the corresponding building height and the occupancy classification as per provisions of this Code and laws of the land.

1.8.7.2 To determine the separation distance between buildings of same height and same occupancy an equidistant imaginary line shall be drawn between the buildings where each building shall comply with requirement of setback and fire separation distance from that imaginary line.

1.8.7.3 Exception: Utilities under Occupancy L is incidental to operation in all type of occupancy except Occupancy J and shall not require the separation distance from the main occupancy. This exception shall not be applicable for Occupancy J.

1.8.7.4 When variation in either height or occupancy occurs, the imaginary line shall satisfy the setback distances for each individual building separately as shown in Figure 3.1.1.

1.8.7.5 Due to the common walls, row or semi-detached houses shall be treated as one building. For semi-detached houses separation distance in the detached sides shall comply with Sec 1.8.7.2 and Sec 1.8.7.3.

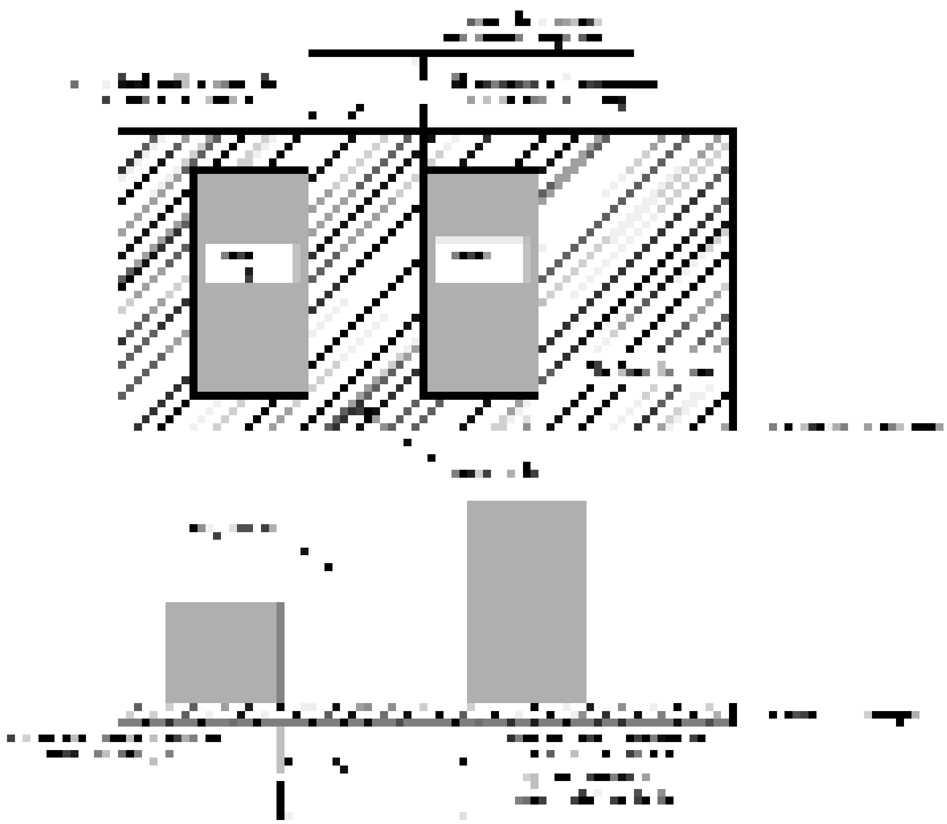


Figure 3.1.1 Separation distance for variation in occupancies and heights

1.8.8 Front Open Space for All Buildings

1.8.8.1 Irrespective of the height of building frontage open space, as defined in Figure 3.1.2, shall be constructed at a distance of at least 4.5 m from the center of the street or at least 1.5 m from the street-front property line whichever is larger.

1.8.8.2 In a corner situation where two frontages of a plot intersects each other and form a sharp corner a turning clearance with a minimum radius of 2 m shall be required as per guidelines of Figure 3.1.3. No construction or visual obstruction shall be allowed within such turning clearance space.

1.8.9 Side and Rear Separation Distances

1.8.9.1 The minimum side and rear open space, as defined as Figure 3.1.2, requirements of a plot for buildings of various occupancy classes shall be as specified in Table 3.1.5.

1.8.9.2 For approved row type residential, mercantile or office as may be permitted by the respective city or development authority and for approved affordable row type, cluster or site and service schemes, the requirement of side separation distance may be waived as per provisions of this Code.

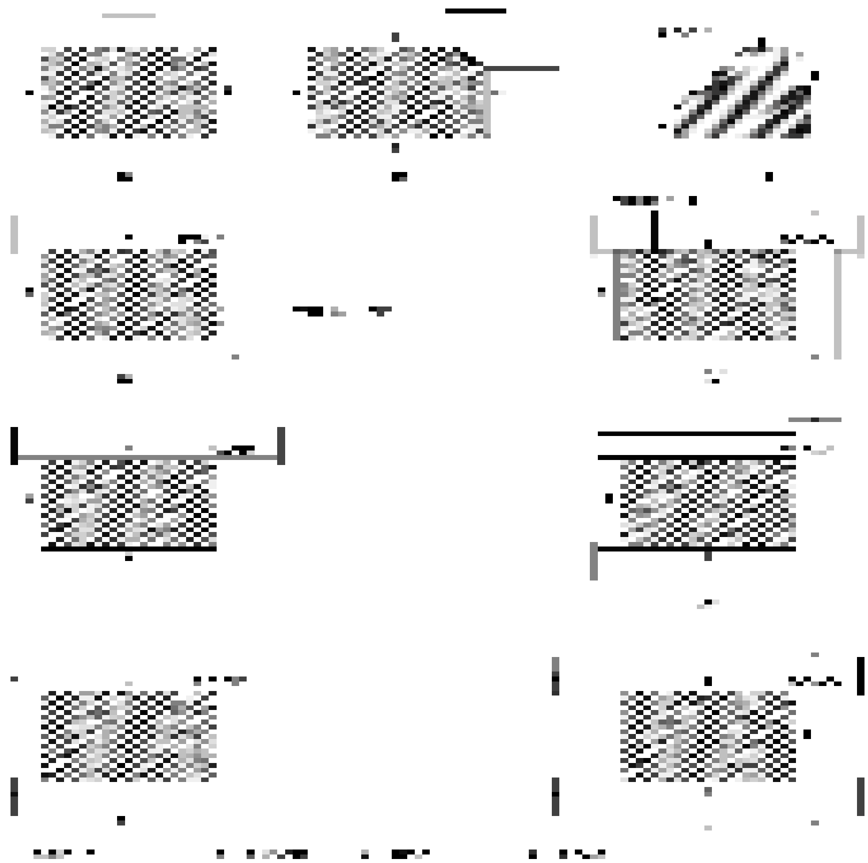


Figure 3.1.2 Definition of front, side and rear of a plot

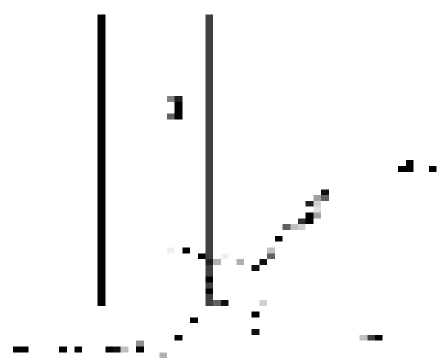


Figure 3.1.3 Restrictions for corner-plots

1.8.9.3 For semi-detached buildings approved by the city or development authority, which are permitted to be constructed with one side on the property line or with pounding gap, the minimum requirements of open space, specified in Sections 1.8.9.1 and 1.8.9.2, for the side opposite to that property line shall be increased as per Table 3.1.5. The requirement of separation distance for the remaining sides shall remain unchanged.

Table 3.1.5: Minimum Rear and Side Open Space Requirements of a Plot

Occupancy	Plot Size* (m^2)	Rear Separation Distance (m)	Side Separation Distance [a](m)
Residential (Row type, not higher than 15m or 4 stories)	Not over 67 Over 67 to below 134	1.25 1.5	Nil [b] Nil [b]
Residential (Semi-detached, not higher than 10 stories or 33 m)	134 to 268 Over 268	2.5 3.0	PG[c], 2.5 PG[c], 2.5
Residential (Detached, Not higher than 10 stories or 33 m)	134 to 268	2.5	1.25
Residential (Detached, Not higher than 10 stories or 33 m)	Over 268	3.0	1.25
Residential(Detached, Higher than 10 stories or 33 m)	Over 268	3.0	3.0
Institution for care	As permitted for this occupancy	3.0	3.0
Educational	As permitted for this occupancy	3.0	3.0
Assembly	Any	3.0	3.0
Business and Mercantile[d] (Not higher than 10 stories or 33 m) semi-detached	Any	1.5	PG[c], 3.0
Business and Mercantile (Not higher than 10 stories or 33 m) Detached	Any	1.5	1.25, 2.5

Occupancy	Plot Size* (m^2)	Rear Separation Distance (m)	Side Separation Distance [a](m)
Business and Mercantile[d] (Higher than 10 stories or 33 m) semi-detached	Over 536	3.0	PG[c], 6.0
Business and Mercantile (Higher than 10 stories or 33 m) Detached	Over 536	3.0	3.0
Industrial	As permitted for this occupancy	As per provisions of this Code	As per provisions of this Code
Storage	As permitted for this occupancy	As per provisions of this Code	As per provisions of this Code
Hazardous	As permitted for this occupancy	As per provisions of this Code	As per provisions of this Code

Notes:

[a] The two dimensions separated by comma stands for each of side separation distances of a semi-detached development.

[b] No side separation distance is required between buildings up to 15 m or 4 stories even for independent plots.

[c] PG stands for 'Pounding Gap', which is a calculated gap for safe distance to avoid pounding due to lateral loads as per provisions of Part 6 of this Code. This gap is not required if the adjoining plots are consolidated and built monolithically. Where pounding gap do not comply with the minimum separation distance, all walls within the separation distance shall be barrier walls.

[d] Mercantile occupancies shared walls between adjacent plots shall only be allowed in accordance to the detail area plan (DAP) administered by the development authority.

* For narrow plots (with site frontage below 12 m) of size 268 m^2 or above in unplanned areas, the local regulatory authority may allow semi-detached typology with a minimum side separation distance of 3m on the unattached side.

1.8.10 Courtyard and Interior Courtyard

An area having proper dimensions as per provision of this Code and open to the sky from the formation level and surrounded by a building or a group of buildings or walls or combination thereof shall be designated as Courtyard. The minimum size of such courtyard shall be derived from Table 3.1.6 depending on the height of the highest building or highest wall abutting the courtyard. The shorter side dimension of such courtyard shall not be less than one-third of the longer side dimension. All such courtyards shall remain open to sky over its entire cross section.

When the sum of exposure area of a courtyard to outer air through its adjacent walls exceed more than thirty percent area of its total peripheral enclosure, it shall be designated as Open courtyard. All other courtyards shall be designated as Interior or Closed courtyard.

1.8.10.1 If any room depends entirely on an interior open space for its natural light and ventilation, such interior open space shall be in the form of an interior courtyard open to the sky over its entire cross-section. The interior courtyard shall have the minimum dimensions depending on the height of the building as specified in Table 3.1.6. The shorter side dimension of such interior courtyard shall not be less than one-third of the longer side dimension.

Table 3.1.6: Minimum Area of Interior Courtyard

No. of Stories	Maximum Height (m)	Minimum Net Area of the Interior Courtyard, m^2
Up to 3	11	9
4	14	16
5	17	25
6	20	36
7	23	49
8	26	64
9	29	81
10	32	100
11	36	121
12-13	42	144
14-15	48	196
16-17	54	256
18-20	63	361

Notes:

1. For buildings above 20 storeys height, the size of the interior courtyard shall not be less than the square of one-third the height of the tallest wall abutting the courtyard.
2. Enclosed open to sky spaces used to provide ventilation as inlet/outlet or daylight to adjacent interiors having dimensions less than that stipulated for internal courts of corresponding storey height given in this Table will be considered ventilation or lighting shafts and not interior courtyards and will follow minimum requirements stipulated in Table 3.1.11

1.8.10.2 The courtyard shall not be interrupted by any form of construction at the courtyard level, except landscaping, sculpture, walkways and water bodies.

1.8.10.3 If the courtyard is to serve as a component of the means of egress, it shall be accessible from all exit points at ground level.

1.8.11 Permitted Construction in the Mandatory Open Space

1.8.11.1 Landscaping, sculpture, walkways, water body shall be permitted in the open space. Any such construction shall comply with Sec 1.8.2 of this Chapter.

1.8.11.2 A maximum of 50 percent of the open space in a plot required by the provisions of Sec 1.8.8 and Sec 1.8.9 may be used for construction of garage, ramps, caretaker or guards quarter and other services auxiliary to and required for the main occupancy of the building, provided that the requirement of community open space in Occupancy A3 is attained, and building is not higher than 10 storey or 33 m, and provided further that conditions (a) to (g) below are satisfied:

(a) No such construction permitted in the open space shall be higher than 2.75 m from the formation level of the plot, except for the tops of inverted beams or intermittent parapets, which may rise up to 3.25 m.

(b) No window, door or ventilator shall be placed on any wall adjacent to the abutting plot or street.

(c) Entrance to the garage or sloping drive way shall not be directly from a public road or street. Distance between the plot line adjoining the road and the entrance to a garage or a sloping drive way shall be kept at least 1.5 m or 4.8 m respectively.

(d) Drainage from the roof or any other part of such construction shall not be allowed to discharge into the adjacent property. Drainage from any part shall not discharge directly into the street through spouts.

(e) No structure or room shall be constructed over the garage or any other permitted service structure within the limits of the mandatory open space.

(f) The roof of any such construction permitted in the mandatory open space shall not be used as a balcony or a terrace or in any such manner that would interfere with the privacy of the occupants of the adjacent property.

(g) No toilet, generator room or electrical substation shall be constructed adjoining the abutting property or street.

1.8.11.3 Edges of slope roof or cornice of the building may be projected into the mandatory open space for a maximum distance of 750 mm. Such extensions shall not be accessible from the building at any level. The construction of a roof or a cornice shall be as such that rain or other water shall not fall from there into the adjacent plot or street.

1.8.11.4 Sunshades over exterior doors or windows of the building may extend into the mandatory open space for a maximum distance of 750 mm, provided that such sunshades are at least 2.5 m above the formation level of the ground.

1.8.11.5 Cantilever canopy at a clear height of at least 2.5 m above the formation level may project into the mandatory open space provided that a horizontal clearance of at least 1.5 m is maintained between the edge of the canopy and the property line. The top surface of such canopy shall not be used as a balcony and shall not be accessible from the building.

1.8.11.6 Balconies at levels higher than 6 m may project into the mandatory open space by not more than 0.9 m provided that a clearance complying the separation distances required in Sec. 1.8.8 and Sec. 1.8.9 are maintained between the edge of the balcony and the property line. Balcony shall be constructed as per provisions of this Code.

1.8.11.7 Water reservoirs, septic tanks, inspection pits, sewer and other underground or above ground service lines shall be permitted in the open space provided that no part of such construction is elevated more than 150 mm above the formation level and the 50 percent mandatory open space shall be unpaved green area.

1.9 General Height and Area Limitations

1.9.1 Authorities having jurisdiction shall permit the built area and building height for an area in accordance to the proposed density of the detail area plan (DAP). Where no such guideline is available, the height of the building shall be determined by the guidelines of Sections 1.9.2.1 to 1.9.2.9 and the built area will be a resultant of open space requirement and permitted height.

1.9.2 Height Limitations Based on Road Width

1.9.2.1 The maximum height of any building of Type I-A and Type I-B construction shall not exceed the nominal value of two times the sum of the width of the front road and the front open space (distance between the front property line and the building).For the purpose of fulfilling this requirement, the height limitations specified in Table 3.1.7 shall apply.

1.9.2.2 For plots having front road width not less than 23 m in an approved residential or business and/or mercantile area, there shall be no restriction on height for residential, business and mercantile buildings of Type I-A and I-B construction provided the minimum open space requirements specified in Table 3.1.8 are satisfied.

1.9.2.3 For Type I-C construction, the maximum permissible height of the building shall be 4 storeys or 14 m for values of two times the sum of the width of the front road and the front open space not less than 13.6 m.

Table 3.1.7: Height Limitations Based on Road Width and Front Open Space

2 × (Front Road Width Plus Front Open Space)	Maximum Permissible Height in Terms of Construction Classification							
	Group- I*						Group-II*	
	Type I-A and Type I-B		Type I-C		Type I-D		Type II-A, II-B, II-D	
	No. of storeys	Height (m)	No. of storeys	Height (m)	No. of storeys	Height (m)	No. of storeys	Height (m)
Below 10.6 m	3	11	2	8	2	8	2	8
10.6 m to below 13.6 m	4	14	3	11	2	8	2	8
13.6 m to below 16.6 m	5	17	4	14	3	11	3	11
16.6 m to below 19.6 m	6	20	4	14	3	11	3	11
19.6 m to below 22.6 m	7	23	4	14	3	11	3	11
22.6 m to below 25.6 m	8	26	4	14	3	11	3	11
25.6 m to below 28.6 m	9	29	4	14	3	11	3	11
28.6 m to below 31.6 m	10	32	4	14	3	11	3	11
31.6 m to below 34.6 m	11	36	4	14	3	11	3	11
34.6 m to below 37.6 m	12	39	4	14	3	11	3	11
37.6 m to below 40.6 m	13	42	4	14	3	11	3	11
40.6 m to below 43.6 m	14	45	4	14	3	11	3	11
43.6 m to below 46.6 m	15	48	4	14	3	11	3	11
and so on in increments of 3 m								

Notes:

1. For plots with front road width (Sec 1.9.2.5) not less than 23 m, residential and business and mercantile buildings of Type I-A and I-B construction shall have no height restriction subject to additional open space requirements (Sec 1.9.2.2).
2. The maximum permissible height for Type I-C construction is 4 storeys or 14 m (Sec 1.9.2.3).
3. The maximum permissible height for Type I-D and I-E of Group I construction and all types of Group II construction is 3 storeys or 11 m (Sec 1.9.2.4).

* For all Unprotected Construction Types I-E of Group I, Type II-C and Type II-E of Group II the maximum allowable storey and height shall be one storey and 8 m respectively.

1.9.2.4 For Type I-D and I-E of Group I construction and all types of Group II construction, the maximum permissible height of the building shall be 3 storeys or 11 m for values of two times the sum of the width of the front road and the front open space not less than 13.6 m.

1.9.2.5 For applying the provisions of Sections 1.9.2.1 to 1.9.2.4, the width of the front road for the layouts shown in Figures 3.1.2(b), (c), (d), (e) and (f) where the plot abuts more than one road, shall be taken as the average of the widths of the abutting roads.

1.9.2.6 For buildings more than six storeys or 20 m high, the following arrangements shall be provided:

(a) Lifts of adequate size, capacity and number as per provisions of this Code.

(b) Adequate fire protection and firefighting arrangements shall be as per provisions of this Code.

(c) Adequate emergency fire escape stair depending upon the type of occupancy and occupancy load as per provisions of this Code.

(d) For buildings with unlimited height (UL) provisions of Table 3.1.8 shall be mandatory.

Table 3.1.8: Minimum Separation Distance for Buildings of Unlimited Height

Occupancy	**Minimum Separation Distance from Plot**		
	Frontage	**Rear**	**Side**
	(m)	**(m)**	**(m)**
Residential	4.0	6.0	4.0
Business, Mercantile.	6.0	6.0	6.0
Educational, Institutional for care, Medical facilities.	6.0	6.0	6.0
Others	As per provision of this Code		

1.9.2.7 For buildings in the vicinity of airports or aerodromes, the height shall be limited by the requirements of the civil aviation authority, city or area development authority or other concerned agencies of the Government.

1.9.2.8 Where more than one construction type is permitted within a building as per provision of this Code among them the lowest fire resistance rated construction type shall be applicable for FAR allotment, and lowest fire resistance rating shall be applicable for the whole structure.

1.9.2.9 For road width above 8.8 m, the building form shall be contained within the pyramid formed by the sky exposure planes on all four sides or as many sides it has, following the guidelines of Figure 3.1.4.

1.9.3 Area Limitations based on FAR

1.9.3.1 Fire separation distance in terms of building setback and building occupancy type and construction type shall govern the FAR to restrict fire hazard volume. FAR shall be decided by the development authorities having jurisdiction.

1.9.3.2 For Occupancy in which unlimited FAR is permitted, the minimum open space requirements specified in Table 3.1.8 shall be applicable.

1.9.3.3 For the purpose of calculating FAR, the area of any floor including basement, of which at least two-third is used exclusively for car parking and the remaining one-third is used for purposes such as mechanical plant room, electrical substation, security cabin, reception booth, water tank, pump house, stairs, lifts and which are accessory to the main occupancy, shall be excluded from the calculation of the total floor area of the building.

1.9.3.4 For area with high public transport accessibility and high FAR the requirement for residential private parking should not be more than one car for every four dwellings or as per guidelines of the authority having jurisdiction.

1.9.3.5 In specifying FAR for a zone or an area, the city or area development authority shall follow the guidelines of Appendix-A (Development Control) and shall take into consideration the following:

(a) Proximity to Public/Mass Transport network

(b) Availability of Urban social infrastructure including urban open spaces

(c) Environmental balance

(d) Adequacy of present and proposed Utility services

(e) Occupancy group and land-use permitted by master plan

(f) Type of construction

(g) Population density of the area

(h) Width of approach roads

(i) Traffic density in the approach roads

(j) Local fire-fighting facilities

(k) Parking facilities

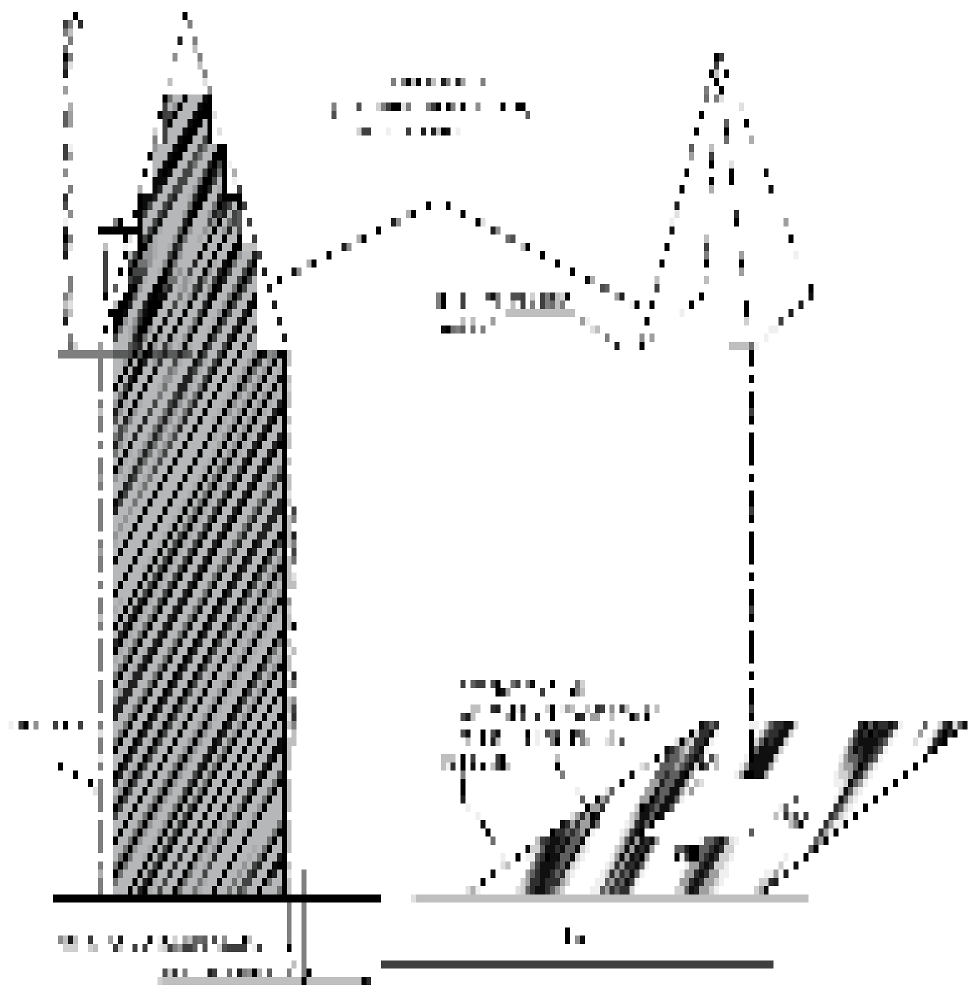


Figure 3.1.4 Limiting envelope for stepped tower structures

1.10 Off Street Parking Spaces

1.10.1 Off street parking requirement for a building or an area shall be decided by the development authority having jurisdiction. A suggestive guideline for off-street parking given in Appendix F might be followed.

1.10.2 Sloping drive way steeper than 1 vertical to 8 horizontal shall not be credited as a walking ramp. When a sloping surface used for both driveway and walking ramp shall be demarcated and the minimum width and sloping ratio of walkways shall be as per provisions of this Code. Sloping driveway entering below grade level shall be protected to prevent water flow into any level that they lead to.

1.11 Street Encroachment

No part of any building shall project beyond the property line or building line established by the provisions of this Code into the street, except the following:

(a) Below Grade: The footing of the boundary wall adjacent to the street may encroach on to the street land not more than 0.3 m and shall rest at least at a depth of 1.5 m below grade.

(b) Above Grade: Marquee, canopy or other temporary cantilever type projection from buildings of business and mercantile occupancy may project on the footpath of a road, provided that no part of such projection is below a height of 3 m from the footpath level and that the outer edge of the canopy is at a minimum clear horizontal distance of 0.25 m from the road side edge of the footpath. The canopy shall be so constructed as to be readily removable without endangering the building structure. No canopy shall project into a street without a footpath. Such canopies shall not project over Mandatory Open Space (MOS). Under no circumstances shall the top of the canopy be used by any floor of the building.

1.12 Community Open Space And Amenities

Community open space for an area or a building shall be decided by the development authority having jurisdiction. Where no such guide line exists or yet to be developed, the guidelines of Sections A.4 and A.5 of Appendix A and Sec B.3.2 of Appendix B shall be applicable.

1.13 Minimum Standard Of A Dwelling

Minimum standard of a dwelling shall be decided by the development authority having jurisdiction.

1.14 Requirements of Parts of Buildings

1.14.1 Plinth and Formation Levels

The plinth and formation levels of the building and the plot shall conform to the requirements of Sec 1.5.3.

1.14.2 Room Dimensions

1.14.2.1 Ceiling heights

(a) All habitable rooms in non-air-conditioned residential, business and mercantile buildings, apart from kitchen, store room, utility room, box room and garage, shall have a ceiling height not less than 2.75 m measured from the finished surface of the floor to the underside of the finished ceiling, or false ceiling. A maximum of one-third of the floor area of such habitable rooms may, however, have a minimum ceiling height of 2.44 m. For air-conditioned rooms in such buildings, the minimum ceiling height shall be 2.44 m.

In the case of pitched roof without a horizontal ceiling the lowest point of the finished ceiling shall be at least 2 m above the finished surface of the floor and the average height of the ceiling shall not be less than 2.44 m.

(b) The minimum clear head room under the ceiling, folded plate, shell etc. and under the false ceiling or duct in an air-conditioned room shall not be less than 2.44 m. The minimum clear distance between the floor below and the soffit of a beam shall not be less than 2.15 m.

(c) The requirements of ceiling height for buildings of occupancy other than residential and business and mercantile shall be as follows:

Table 3.1.9: Minimum Ceiling Heights for Different Occupancies

Occupancy	Minimum Ceiling Height
Educational, Institutional, Health Care, Assembly.	3 m for non-air-conditioned and 2.6 m for air-conditioned buildings.
Industrial, Storage, Hazardous.	3.5 m for non-air-conditioned and 3.0 m for air-conditioned buildings.

1.14.2.2 Room sizes

All habitable rooms used for sleeping and other purposes of a dwelling unit shall not be less than 9.5 m² of net floor area with a minimum width of 2.9 m and shall comply with indoor air quality requirement as per provisions of this Code. Other non-habitable rooms in the dwelling unit shall have a minimum area of 5 m² with a minimum width of 2 m.

1.14.3 Kitchen

1.14.3.1 The minimum clear height of kitchen measured from the finished surface of the floor to the finished ceiling shall be 2.75 m, except for any floor trap of the upper floor which shall have a minimum clearance of 2.15 m above the finished floor. The minimum clear height of kitchen shall be 2.15 m where mechanical exhaust is installed.

1.14.3.2 The minimum floor area of kitchen without provision for dining shall be 4 m^2 with a minimum width of 1.5 m. The minimum floor area of a kitchen which is intended to provide dining or occasional sleeping space shall be 7.5 m^2 with a minimum width of 2.2 m.

1.14.3.3 Every kitchen shall be provided with a kitchen sink or other means for washing utensils. The waste water shall be discharged into the waste water pipe or drain as per provisions of Part 8.

1.14.3.4 The floor of the kitchen shall be slip-resistant and water tight.

1.14.3.5 Every kitchen shall be provided with window having a minimum area of 1 m^2 which shall open to the exterior or to an interior open space of adequate dimensions complying with Sec 1.19.

1.14.3.6 It is recommended that all kitchens should be designed as accessible kitchens for people with disability considering the door width, accessible route, turning clearance within the kitchen, counter heights, placement of fixtures, knee and toe clearances under counters and other relevant criteria in compliance to the guidelines of Appendix D.

1.14.4 Bathroom and Toilets

1.14.4.1 The height of any bathroom, toilet or water closet shall not be less than 2.15 m measured from the finished floor surface to the finished ceiling or false ceiling or to the lowest point of any trap of the upper floor's plumbing system.

1.14.4.2 The minimum requirement of floor area and width of a bathroom with 3 fixtures, 2 fixtures or single fixture shall conform to the space standards of Table 3.1.10.

1.14.4.3 Details for requirement of adaptable or accessible toilets shall follow the guidelines of Appendix D.

Table 3.1.10: Bathroom Space Standards

Facility	Minimum Width (m)	Floor Area (m^2)
Water closet + bathing + hand washing	1.25	3.00
Water closet + bathing	1.00	2.80
Bathing only	1.00	1.50
Water closet only	1.00	1.20
Adaptable toilets	1.50	as per Appendix D

1.14.4.4 No bathroom or toilet containing water closet shall open directly into any kitchen or cooking space by a door, window, ventilator, fanlight or any other opening. Every such bathroom or toilet shall have a door completely shutting it off from the exterior.

1.14.4.5 Every bathroom, toilet and water closet shall be located against an exterior wall or wall on the interior open space (see Sec 1.8.10), except where they are ventilated through an interior lighting and ventilation shaft. Such interior lighting and ventilation shafts shall have the minimum dimensions specified in Table 3.1.11 for different heights of buildings. In addition, shafts for buildings exceeding 6 storeys or a height of 20 m shall be mechanically ventilated. All shafts must be accessible at the ground floor level for cleaning and servicing purposes.

Table 3.1.11: Minimum Dimensions of Lighting and Ventilation Shaft

Building Height		**Minimum Net Cross Sectional Area of Shaft**	**Minimum Width of Shaft**
No. of Stories	**Height (m)**	**(m^2)**	**(m)**
Up to 3	Up to 11	1.50	1.00
4	14	3.00	1.20
5	17	4.00	1.50
6	20	5.00	2.00
Over 6*	Over 20	6.50	2.50

* Mechanical ventilation of the shaft shall be provided for buildings over 6 stories high.
Shaft dimensions shall conform to mechanical design considerations.

1.14.4.6 Floors of bathrooms, toilets or water closets shall be treated with water repellent material and shall be water tight. All bathroom walls or partitions shall be treated with non-absorbent water repellent smooth impervious finish material to a height of not less than 1 m above the finished floor level. The floor shall be sloped gently towards gratings or openings of the floor traps.

1.14.4.7 All public buildings shall have adaptable toilet as per requirement of the development authorities having jurisdiction. Each dwelling unit shall have at least one adaptable toilet. The details of such toilet shall comply with requirements of Appendix D (Universal Accessibility).

1.14.5 Stairways

1.14.5.1 Limiting Dimensions

The minimum width of the staircase for various occupancies shall be as specified in Table 4.3.6 of Part 4.

1.14.5.2 Sum of two risers and one tread excluding nosing dimension shall not be less than 610 mm and not more than 648 mm. All Risers and Treads shall be identical in consecutive two flights starting from one floor to another floor. Difference between two consecutive risers or treads shall not be more than 5 mm. The combination of riser and treads shall comply with Table 4.3.4 Chapter 3, Part 4.

1.14.5.3 The maximum flight height between landings shall not be more than 3660 mm. For Assembly occupancy maximum flight height between landings shall not be more than 2440 mm.

1.14.5.4 The minimum clear head room between flights of a staircase shall be 2.15 m. The clear head room may be reduced to 2.03 m for not more than three flights in any staircase.

1.14.5.5 The minimum clear height of any passage below a landing providing access to non-habitable and service spaces shall be 2.03 m. The minimum clear height of all other passages and spaces below a landing shall be 2.15 m.

1.14.5.6 Handrails shall have a minimum height of 0.9 m measured from the nose of stair to the top of the handrail.

1.14.6 Mezzanine Floor

1.14.6.1 Each mezzanine floor area in a space shall not exceed one-third of the main floor area. The area of the mezzanine shall be included in calculating the FAR.

1.14.6.2 The clear headroom both over and under the mezzanine floor shall be at least 2.2 m.

1.14.6.3 The lighting and ventilation of the space both over and under the mezzanine floor shall not be obstructed in any way.

1.14.7 Lofts

1.14.7.1 Space under slope roof termed lofts shall not be used as a habitable space where minimum ceiling height is less than the requirement but more than 1.5 m.

1.14.7.2 The minimum ceiling height requirements for various rooms specified under Sections 1.14.2.1, 1.14.2.2, 1.14.3 and 1.14.4 shall be maintained under the loft.

1.14.7.3 A maximum of 25% of the floor area of any room may be covered by a loft, except bathrooms, toilets, water closets, store rooms and corridors where the whole area may have an overhead loft.

1.14.7.4 The loft shall not interfere with the lighting and ventilation of any room.

1.14.8 Cabins or Chambers

1.14.8.1 Cabins or Chambers created by removable partitions on open floor shall have a minimum area of $3m^2$.

1.14.8.2 Clear passages at least 0.75 m wide (or as stipulated in Part 4) shall be maintained between the cabins leading to a means of exit which shall in no case be further than 16 m from any cabin.

1.14.8.3 A clear gap of at least 300 mm shall be maintained between the top of the partition walls enclosing the cabin and the ceiling, unless the cabin is exposed to the exterior deriving natural light and ventilation or is artificially lighted and ventilated.

1.14.9 Store Room

A store room provided in a dwelling unit of a residential building shall have a minimum area of 1.5 m^2 with a minimum width of 1 m. The clear height of the store room shall not be less than 2.2 m.

1.14.10 Private Garage

Private garage in residential occupancy A1 and A2 building shall have a minimum clear height of 2.03 m. The minimum area of the parking stall in a garage shall be decided in accordance with the provision of Sec F.7.1 of Appendix F.

1.14.11 Basement

Any underground floor of a building wholly or partially below formation level shall be called a basement and shall satisfy the requirements of the following sections.

1.14.11.1 Subject to the provision of Sec 1.9.3.3, the area of the basement shall be included in the calculation of FAR.

1.14.11.2 The walls and floors of the basement shall be damp-proof and water-proof as per provision of this Code. The basement shall be protected against surface and sub-surface waste water intrusion.

1.14.11.3 The basement shall be lighted and ventilated as per provision of this Code.

1.14.11.4 The staircases of a building serving above grade level also entering into below street floor level shall be enclosed by barrier wall with two door smoke proof vestibule shall have minimum 2 hours fire resistance time.

1.14.11.5 Ramp provided as walkways shall not be steeper than 1 vertical in 8 horizontal.

1.14.11.6 The clear height of the basement below soffit of beams shall not be less than 2.03 m.

1.14.12 Entrance to the Building

All buildings shall have a covered entrance or other covered area for callers waiting at the door. The main entrance door to the building shall not open into an uncovered exterior. All public buildings shall have universal accessibility as per provisions of Appendix D of Part 3.

1.14.13 Roof Drainage

1.14.13.1 The roof of a building shall be constructed in such a manner that rain water is drained freely away from the building without causing dampness of the roof or the walls of the building or of an adjacent building.

1.14.13.2 Water from the roof shall not be discharged into the adjacent property or street.

1.14.13.3 For one or two storied buildings with flat or pitched roof, rain water may be discharged directly to the ground, in which case the roof shall have extended eaves or cornices to direct the water away from the walls.

1.14.13.4 For other buildings, gutters or parapets shall be provided to direct the water to the piping of an adequate rain water drainage system.

1.14.13.5 The roof shall be impermeable or shall be treated with an impervious material to make it effectively water tight. Flat concrete roofs shall be topped with an impervious layer of lime concrete or other effective means of waterproofing. All flat roofs shall be sloped gently towards gutters, gratings or mouths of the rain water drainage pipes.

1.14.13.6 For sustainable development, building may have rain water harvesting system as stipulated in Part 8, Chapter 7.

1.14.14 Parapet

All accessible flat roofs shall be enclosed by parapets or guardrails having a height of at least 1 m. All such parapets and guardrails shall be designed to withstand the lateral forces due to wind and occupancy in conformity with the provisions of Part 6 of this Code.

1.14.15 Septic Tank

A septic tank shall be provided within the premises for disposal of sewage, whether any public sewer is available or not. The location, design and construction of the septic tank shall conform to the requirements of this Code.

1.15 Landscaping

1.15.1 Plantation of trees and shrubs within the open spaces of a plot aimed at enhancing the environmental quality of the building shall comply with the requirements of this Section.

1.15.2 Trees and shrubs shall be planted judiciously to meet the requirements of shade and sunshine, to control noise and dust, to provide privacy and to improve visual quality, without jeopardizing natural ventilation and lighting of a building.

1.15.3 Species of trees shall be so chosen and planted that their roots do not endanger the building foundation and their branches do not interfere with the building superstructure. This shall be achieved by maintaining sufficient distance between the trees and the building depending on the species of the tree.

1.16 Damp-Proofing and Waterproofing

Foundation, floor slabs, walls and roof of a building shall be damp proof, water proof and weather proof in accordance with the provisions of Part 6 of this Code.

1.17 Existing Buildings

1.17.1 Existing buildings and structures in their present occupancy condition shall not be required to be in full compliance with all the requirements of this Part of this Code. Additions or alterations to such existing buildings or change of use thereof shall not be permitted if such addition, alteration or change of use or occupancy is likely to render the building more hazardous with respect to fire safety, life safety and sanitation than it was before.

1.17.2 Any horizontal or vertical extension of an existing building or any change of use thereof shall subject the altered building or occupancy to the provisions of this Code for a new building. The building together with the additions and changes shall not exceed the height, area and open space requirements for new buildings specified in this Code.

1.17.3 All buildings and structures, both new and existing shall be maintained in a safe and sanitary condition as provided for in this Code. To determine compliance with this requirement, the Authority may cause the building or structure to be periodically inspected.

1.17.4 Any proposed change in an existing building or structure shall have to satisfy the requirements set forth in Part 6 of this Code.

1.18 Buildings and Areas of Historical or Architectural Value

1.18.1 Buildings and areas of Historical value are part of our heritage and cultural inheritance and should therefore be protected. Similarly buildings and works under the jurisdiction of and identified by the Authority as having architectural value shall also be protected. The identification, listing and classification of all such buildings and places of historic or architectural values shall follow the guidelines of Chapter 3 of Part 9, Section 1.5 of Part 1 and Section 3.14 of Part 2.

1.18.2 Repairs, alterations and additions necessary for the preservation, restoration, rehabilitation, continued use or adaptive reuse of such historic buildings and structures, and of buildings and works of architectural value may be exempted by the Authority from having to be in full compliance with all the requirements of this Code, provided that the restored building or structure will be no more hazardous, if any, than the existing conditions in terms of life safety, fire protection and sanitation. All such buildings and places shall comply with the provisions for conservation of heritage buildings or area of Part 9.

1.19 Ventilation, Lighting and Sanitation

1.19.1 All rooms and interior spaces designated for human occupancy shall be provided with means of natural or artificial lighting and natural or mechanical ventilation as per provisions of this Code. At least one side of all habitable rooms shall be exposed to an exterior or an interior open space or to a balcony or verandah exposed to an open space.

1.19.2 All buildings shall have water and sanitation facilities as per provisions of this Code.

1.19.3 Every kitchen shall have facility for washing of utensils.

1.19.4 Every building or independent unit thereof shall be provided with at least one water closet.

1.19.5 All naturally ventilated and illuminated interior spaces, staircases and other areas of human occupancy in a building shall have windows or ventilators opening directly to the exterior or an interior open space or to a verandah. Ventilation of bathrooms may also be achieved through ventilation shafts as provided for in Sec 1.14.4.5.

1.19.6 All habitable and non-habitable spaces within a building shall have the following minimum aggregate area of openings in the exterior wall, excluding doors, expressed as percentage of the net floor area:

Table 3.1.12: Dimension of Openings for Different Uses

Space	Percent of Net Floor Area
Habitable rooms such as those used for sleeping, living, study, dining etc.	15
Kitchens*	18
Non-habitable spaces such as bathrooms, store, staircase and other utility	10

* Minimum height from the window sill of a kitchen shall be 450mm above cooking range. Air flow on cooking range shall be restricted.

1.19.6.1 An enclosed staircase shall have windows not less than 1 m^2 in area on exterior walls of every landings as per provisions of this Code.

1.19.6.2 Toilet and bathroom windows shall open to the exterior or an approved ventilation shaft and the operable area shall not be less than 1 m^2.

1.19.7 The required minimum average intensity of illumination in a habitable space at a height of 750 mm above the floor level shall be 65 lux. Any point in a room more than 7 m away from an exterior window shall be considered to be not illuminated by daylight unless measurement of illumination gives an intensity of 65 lux or more.

1.19.7.1 The required intensity of illumination for various tasks in a building shall be as specified in Chapter 1 of Part 8.

1.19.7.2 Whenever the illumination achieved by daylight is not sufficient or occupancy at night is necessary, artificial lighting shall be installed to supplement daylight, or to provide the required night lighting, in accordance with the provisions of Chapter 1 of Part 8.

1.19.8 Protected openings, when and where are installed shall not be normally operable form the inside of a building. Such openings however, shall not be credited towards meeting any ventilation requirements.

1.19.9 The requirements of opening areas specified in Sec 1.19.6 shall suffice for ventilation provided that the windows or ventilators forming the opening are operable. When part of a window area is made of fixed glazing, only the operable portion shall be counted in aggregating the opening area.

1.19.9.1 To achieve the desired indoor air quality by natural means, an interior space shall preferably have minimum two openings on two different walls where the opening acting as inlet must be an exterior wall and the summation of the net opening area on walls shall not be less than 5% of the net floor area thereof.

1.19.9.2 Mechanical ventilation, when provided, shall conform to the requirements of Chapter 3 of Part 8.

1.20 Air-Conditioning and Heating

When air-conditioning and heating system are installed, an indoor air quality shall be maintained as per provisions of Chapter 3 Part 8.

1.21 Provision of Lifts and Escalators

Wherever required by this Code or desired by the owner for comfort, lifts and escalator facilities shall be planned, designed and installed in accordance with the provisions of Part 4 and Part 8 of this Code. The minimum size of a lift lobby shall be 1.5 m x 1.5 m. For accessible lift guidelines of Appendix D shall be applicable.

1.22 Sound Insulation

Acoustical design of a building to attain the desired noise levels shall be performed in accordance with the provisions of Chapter 4 Part 8.

1.23 Thermal Insulation

Thermal comfort in a building shall be achieved through adequate ventilation and thermal insulation of walls and roof.

1.24 Lightning Protection of Buildings

Lightning protection measures shall be installed on all buildings whose exposure conditions indicate the likelihood of lightning strike and consequential hazard to life and property. The requirement of lightning protection systems shall be assessed and they shall be designed and installed in accordance with the provisions of Chapter 2 Part 8.

1.25 Rat Proofing and Termite Proofing Of Buildings

Rat proofing and termite proofing measures shall be undertaken on the basis of the degree of protection desired from rats and termites. Any chemical used for the control of rats and termite shall be free from environmental hazards. Periodic inspections shall be undertaken for effective protection against rats and termites.

1.26 Requirements For Buildings In Flood Prone and Coastal Regions of Bangladesh

The specifications of this Section shall be applicable to all buildings located in the flood or surge prone areas in addition to other requirements of this Code.

(a) The planning and development control authority of the city, township, municipality or region where this Code is intended to be applied shall delineate any area having a potential for being flooded under at least 1 m deep water due to flooding as Flood Prone Area (FPA). The provisions of Sec 1.26.1 shall be applicable to areas designated as FPA. There shall be a design flood level in the FPAs which shall be recommended by the Authority to be used in interpreting the provisions of this Section.

(b) Similar delineation shall be made in the coastal regions on the basis of expected occurrence of a surge or wave run-up of 1 m or higher. Such areas shall be designated as Surge Prone Area (SPA). The provisions of Sec 1.26.2 shall be applicable to buildings located in the SPAs. There shall be a design surge height in the SPAs which shall be recommended by the Authority to be used in interpreting the provisions of this Section.

1.26.1 Flood Prone Areas

1.26.1.1 Elevation

The habitable floors of a building located in the flood prone area shall be elevated above the design flood level. Buildings up to two storeys high shall have accessible roof with an exterior stair. Buildings having three storeys or more height, the floor immediately above the design flood level shall be accessible with an exterior stair.

Exceptions:

(a) Except for Occupancy A (Residential), any occupancy may have floors below the design flood level in accordance with the provisions of Sec 1.26.1.3.

(b) Floors which are used only for building access, exits, foyers, storages or parking garages may be located below the design flood level in accordance with the provisions of Sec 1.26.1.2.

1.26.1.2 Enclosures below design flood level

There shall be no enclosed space below the design flood level except for building access, exits, foyers, storage and parking garages. There shall be vents, valves or other openings in the walls of the enclosed spaces which shall equalize the lateral pressure of the water. The bottom of such openings shall not be higher than 300 mm above the finished grade. There shall be at least two openings for each enclosure in a building. The total net area of openings for an enclosure shall be at least 0.4 m^2 or 7 percent of the floor area of the enclosure, whichever is greater.

1.26.1.3 Flood-resistant Construction

Floors constructed below the design flood level under the provisions of Exceptions in Sec 1.26.1.1 shall comply with the following requirements:

(a) Floors and exterior walls of such floors shall have a construction impermeable to the passage of water.

(b) Structural components of such floors shall be capable of resisting the hydraulic and buoyant forces resulting from the occurrence of floods at the design flood level. Design requirements in such cases are specified in Chapter 1, Part 6.

(c) Vents, openings and valves provided below the design level shall have water-tight closures capable of resisting any structural forces resulting from the occurrence of the design flood.

(d) Penetrations made for electrical, mechanical or plumbing installations shall be made water-tight to prevent any penetration of flood water. Sewerage systems having opening below the design flood level shall have a closure device to prevent backwater flow during the occurrence of floods.

1.26.2 Surge Prone Areas

1.26.2.1 Elevation

The habitable floor of any building in a surge prone area shall not be located below the design surge height. For buildings of height two storeys or less the roof shall be accessible with an exterior stair. For buildings having three storeys or higher, the floor immediately above the design surge level shall be accessible with an exterior stair.

Exception:

Footing, mat or raft foundations, piles, pile caps, columns, grade beams and bracings may be constructed below the design surge height.

1.26.2.2 Enclosures below Design Surge Height

Spaces of a building in the SPAs below the design surge height shall not obstruct any flow of water during the occurrence of surge.

Exception:

Structural or non-structural members serving as entries or exits may be constructed below design surge height.

1.26.2.3 Foundations

Foundations of the buildings erected in the SPA'$_s$ shall be located well below the ground level so that they are protected from erosion or scour during the occurrence of surge. If piled foundations are used, they shall be designed to withstand with adequate factor of safety and the loss of support due to scour. Design of the foundations shall conform to the requirements of Chapter 3 Part 6.

1.27 Requirements for Buildings In Other Disaster Prone Areas

In hilly region, authority shall ask for a special site drainage plan conforming to the area drainage network before approval of any building work. This shall apply for all buildings to be constructed in hilly areas where there is the danger of failure of slopes, including mudslides, flash floods and soil erosion. Such failures may occur in hilly areas, where the angle of slope is greater than 30°. Prevention of failure of slopes shall be achieved by the following measures:

(a) Retaining walls to prevent soil erosion as per provisions of Part 6 of this Code.

(b) Weep holes to allow water pressure balancing from the water logged soil on the retaining wall.

(c) Adequate site drainage respecting the natural topography of the site and surrounds.

(d) Use of vegetation to retain the top soil and bonding quality of the soil.

(e) Protection of soil by catchment pools to prevent soil erosion due to discharge from elevated level onto the ground.

1.28 Special Provision For Storage of Dangerous Goods and Their Classification

1.28.1 Any substance including mixtures and solutions shall be assigned to one of the following Classes for any Occupancy if it crosses the limits of exempted quantities as per Table 3.2.5 of Part 3, Section 2. Some of these classes are subdivided into divisions also. The numerical order of the classes or divisions is not the representative of the degree of danger. These classes including their divisions are listed below:

Class 1: Explosives

Division 1.1: Substances and articles which have a mass explosion hazard.

Division 1.2: Substances and articles which have a projection hazard but not a mass explosion hazard.

Division 1.3: Substances and articles which have a fire hazard and either a minor blast hazards or a minor projection hazards, but not a mass explosion hazard.

Division 1.4: Substances and articles which present no significant hazard.

Division 1.5: Very insensitive substances which have a mass explosion hazard.

Division 1.6: Very insensitive substances which do not have a mass explosion hazard.

Class 2: Gases

Division 2.1: Flammable gases

Division 2.2: Non-flammable, non-toxic gases

Division 2.3: Toxic gases

Class 3: Flammable Liquids

Class 4: Flammable Solids; Substances Liable to Spontaneous Combustion; Substances which, in contact with Water, Emit Flammable Gases:

Division 4.1: Flammable solids, self-reactive substances and solid

Division 4.2: Substances liable to spontaneous combustion

Division 4.3: Substances which, in contact with water, emit flammable gases

Class 5: Oxidizing Substances and Organic Peroxides

Division 5.1 Oxidizing substances

Division 5.2 Organic peroxides

Class 6: Toxic and Infectious Substances

Division 6.1: Toxic substances

Division 6.2: Infectious substances

Class 7: Radioactive Material

Class 8: Corrosive Substances

Class 9: Miscellaneous Dangerous Substances and Articles

The quantity and procedure for storage, merchandising, handling, processing, packaging, transportation, shipment and uses of all dangerous goods of above classification shall be regulated as per guidelines of Explosive Act and other relevant Acts and as per rules of Bangladesh Shipping Corporation for safe handling of container for dangerous goods. The signs and symbols for all such goods shall comply with the requirements of Bangladesh Shipping Corporation's guidelines.

1.28.2 HS Code, Proper Shipping Names and UN Numbers

First Schedule of Bangladesh customs tariff that is Harmonized System code shall be used for the description of any substances and its corresponding UN number shall be used for proper shipping name and for the classifications of dangerous goods. The storage and use of all such substances and goods shall be controlled as per provision of this Code and explosive control act.

1.29 List of Related Appendices

Appendix A	Development Control and Planning
Appendix B	Minimum Standard Housing
Appendix C	Cluster Planning
Appendix D	Universal Accessibility
Appendix E	Building Types
Appendix F	Road Hierarchy, On-street and Off-street Parking

PART III

Chapter 2

Classification of Buildings Based on Occupancy

2.1 Occupancy Classification

2.1.1 Every building or portion thereof and land-use shall be classified according to its use or the character of its occupancy as a building of Occupancy A, B, C, D, E, F, G, H, I, J, K, L or M as defined below:

Occupancy A:	Residential
Occupancy B:	Educational
Occupancy C:	Institution for care
Occupancy D:	Health Care
Occupancy E:	Business
Occupancy F:	Mercantile
Occupancy G:	Industrial
Occupancy H:	Storage
Occupancy I:	Assembly
Occupancy J:	Hazardous
Occupancy K:	Garages
Occupancy L:	Utilities
Occupancy M:	Miscellaneous

2.1.2 Utilities under Occupancy L is incidental to operation in all other type of occupancy except Occupancy J shall be considered as non-separated use of the main occupancy but shall be taken special safety measure as per provision of this Code.

2.1.3 Any occupancy or use type not mentioned specifically in Table 3.2.6 (A-Z list) or elsewhere in this Code shall be classified by the Board of Appeals under the occupancy group to which its use most closely resembles, considering the life safety and fire hazard.

2.1.4 Each occupancy group shall be subdivided as detailed in the following sections. The detail classification including mixed occupancy provided in the Table 3.2.6 (A-Z list) is non-exhaustive. If there is any use or character of occupancy in a building which is not mentioned here, it shall be classified as per provision of Sec 2.1.3 of this Chapter.

2.1.5 Occupancy A: Residential Buildings

This occupancy type shall include any building or portion thereof providing sleeping and living accommodations to related or unrelated groups of people, with or without independent bathroom, cooking or dining facilities, except any building classified under Occupancy C or D. This Occupancy shall be subdivided as follows:

2.1.5.1 Single Family Dwelling (A1)

These shall include any building, row type or semi-detached or detached from neighboring buildings by distances required by this Code and having independent access to the plot, which is used as private dwelling by members of a single family.

2.1.5.2 Two Family Dwelling (A2)

These shall include any building, row type or semi-detached or detached from neighboring buildings by distances required by this Code and having shared or independent access for two families and having facilities for living, cooking and bathroom facilities independent of each other.

2.1.5.3 Flats or Apartments (A3)

These shall include any building or portion thereof which is provided for more than two families, having facilities for living, cooking and bathroom facilities independent of each other.

2.1.5.4 Mess, Boarding Houses, Dormitories and Hostels (A4)

These shall include any building or portion thereof in which sleeping, living accommodations and bathroom are provided for groups of related or unrelated persons, with or without common dining and facilities, and with common cooking under single management control or with individual or group cooking facilities.

2.1.5.5 Hotels and Lodging Houses (A5)

These shall include any building, a portion thereof or group of buildings under single management, in which sleeping, living accommodation and bathroom facilities are provided with or without dining facilities but without cooking facilities for adult individuals, is provided for hire on transient or permanent basis.

2.1.6 Occupancy B: Educational Facilities

This occupancy type shall include any building or portion thereof in which education, training and care are provided to children or adults. This Occupancy shall be subdivided as follows:

2.1.6.1 Educational Facilities up to Higher Secondary Level (B1)

These shall include any building or portion thereof used for purposes involving assembly for instruction, education and recreation of more than six persons on regular basis to fulfil the requirement of an academic curriculum approved by the Government up to Higher Secondary (12^{th} Grade), and which is not covered by occupancy I.

2.1.6.2 Facilities for Training and for Above-Secondary Level (B2)

These shall include any building or portion thereof used for purposes involving assembly for instruction, education, training and recreation of more than six persons, and which is not covered by occupancy I and B1.

2.1.6.3 Pre-School Facilities (B3)

These shall include any building or portion thereof used for purposes involving care, recreation and education of children more than six in number, who have not yet reached the age to attend the school.

2.1.7 Occupancy C: Institution for Care

Buildings classified under this occupancy shall include those used for purposes of institutional care of the occupants, such as detention for correctional or penal purposes, medical or nursing care of persons suffering from illness or infirmity due to mental condition, or accommodation of children or minor, where the personal liberty of the inmate is restricted. These buildings shall ordinarily provide accommodation for sleeping, dining and other provisions approved by the authority for the occupants. This occupancy shall be subdivided as follows:

2.1.7.1 Institution for Care of Children (C1)

These shall include any building or portion thereof or group of buildings under single management used as an institution for the full time care of children or minor, each providing accommodation for sleeping, dining and other provisions approved by the authority for more than six children.

2.1.7.2 Custodial Institution for Physically Capable Adults (C2)

These shall include any building or portion thereof or group of buildings under single management used for purposes of full time care and custody of adult or mentally disabled persons but physically capable of responding to emergency.

2.1.7.3 Custodial Institution for the Incapable Adults (C3)

These shall include any building or portion thereof or group of buildings under single management used for purposes of full time care and custody of persons physically or mentally incapable of responding to emergency.

2.1.7.4 Penal and Mental Institution for Children (C4)

These shall include any building or portion thereof or group of buildings under single management used for housing children under restraint, or who are detained for penal and corrective purposes, in which personal liberty of the inmates is restricted.

2.1.7.5 Penal and Mental Institution for Adults (C5)

These shall include any building or portion thereof or group of buildings under single management used for housing persons under restraint, or who are detained for penal and corrective purposes, in which personal liberty of the inmates is restricted.

2.1.8 Occupancy D: Health Care Facilities

Buildings under this Occupancy group shall include those used for purposes of providing medical care, diagnostic facilities and treatment to persons suffering from physical discomfort, in which sleeping accommodation may or may not be provided. This Occupancy shall be subdivided as follows:

2.1.8.1 Normal Medical Facilities (D1)

These shall include any building or portion thereof or group of buildings under single management in which essential medical facilities having surgery, emergency and casualty treatment facilities, general or specialized medical and other treatment are provided to persons suffering from physical discomfort.

2.1.8.2 Emergency Medical Facilities (D2)

These shall include any building or portion thereof used for purposes of providing essential medical facilities having surgery, emergency, casualty treatment facilities, general or specialized medical and other treatment is provided to persons suffering from physical discomfort. This Type shall be equipped and designated to handle post disaster emergency, by construction it is required to remain operational during and after disasters, built as a part of disaster preparedness program.

2.1.9 Occupancy E: Business

These shall include any building or portion thereof which is used for any business transaction other than mercantile. This Occupancy shall be subdivided as follows:

2.1.9.1 Office (E1)

These shall include any building or part thereof which is used for paper works, documentations, only display of samples of Products but not for direct sale, maintaining accounts and records for administrative or consulting services, banking or activities for business purposes and professional training.

2.1.9.2 Research and Testing Laboratories (E2)

These shall include any building or portion thereof which is used as research establishment and/or test laboratory involving hazardous materials within the limit of exempted quantity permitted in this Code.

2.1.9.3 Essential Services (E3)

These shall include any building or portion thereof used for purposes of providing emergency services and utilities which are required to remain operational during and after a disaster or other emergency situations.

2.1.10 Occupancy F: Mercantile

This occupancy type shall include any building or portion thereof or group of buildings which is used for display and sale of merchandises. This Occupancy shall be subdivided as follows:

2.1.10.1 Small Shops and Market (F1)

These shall include any building or portion thereof with an area divided or undivided not exceeding 300 m^2, used for purposes of display and sale of merchandise, either wholesale or retail, with or without incidental storage and service facilities.

2.1.10.2 Large Shops and Market (F2)

These shall include any building or portion thereof with an area divided or undivided more than 300 m^2 used for purposes of display and sale of merchandise, either wholesale or retail, with or without incidental storage and service facilities.

2.1.10.3 Refueling Station (F3)

These shall include any building or portion thereof used for providing refueling and maintenance without repair services for automobiles which is moderately hazardous in nature.

2.1.11 Occupancy G: Industrial Buildings

Buildings under this Occupancy shall be subdivided on the basis of hazard potential of the contents and the processes of the industry. The hazard shall generally mean the relative danger of the start of fire and the rapidity of its spread, the danger of smoke and gases generated that pose a potential threat to the safety of the occupants of the building. Unless areas with different degrees of hazard are effectively segregated and separated in accordance with the provisions of this Code, the most hazardous area in a building shall govern its classification. This occupancy shall also include facilities for public utility services at the producer or distributor’s end that deals with generation and distribution of utility facilities. Any such building or portion thereof, which is not using hazardous material quantified and categorized in occupancy group J, shall be subdivided as follows:

2.1.11.1 Low Hazard Industry (G1)

These shall include any industrial building in which the contents are of such low combustibility and the processes conducted therein are of such low hazardous nature that danger of self-ignition and self-propagation of fire is nonexistent, the only danger being an onset of fire from external sources with the resulting danger to life and property.

2.1.11.2 Moderate Hazard Industry (G2)

These shall include any industrial building in which the contents are moderately combustible and the industrial processes conducted therein are liable to give rise to a fire which will spread with moderate rapidity, giving off considerable smoke.

2.1.12 Occupancy H: Storage Buildings

Buildings under this Occupancy group shall include any building or portion thereof used primarily for storage or sheltering of goods, wares, merchandises, vehicles or animals. Any such building or portion thereof, which is not used for storing hazardous material quantified and categorized in occupancy group J, shall be subdivided as follows:

2.1.12.1 Low Fire-risk Storage (H1)

These shall include any building or portion thereof which is used for storage of materials or other contents which do not constitute the danger of self-ignition, and in the event of fire the rate of burning shall be less than moderate rapidity.

2.1.12.2 Moderate Fire-risk Storage (H2)

These shall include any building or portion thereof which is used for storage of materials which do not constitute the danger of self-ignition but which in the event of fire will burn with moderate rapidity.

Items which shall be deemed to render a building hazardous are specified in Sec 2.14.3 along with the exempted amount for each item.

2.1.13 Occupancy I: Assembly

Buildings under this Occupancy group shall include any building or portion thereof in which groups of people congregate or assemble for recreation, amusement, social, religious, political, cultural, travel and similar purposes. This Occupancy shall be subdivided as follows:

2.1.13.1 Large Assembly with Fixed Seats (I1)

This occupancy shall include a building or a portion thereof for assembly in a space provided with fixed seats for 1000 or more persons. Assembly buildings under this subdivision may be for theatrical, operatic performances or cinema projection having or not a raised stage, proscenium curtains, scenery loft or projection screen, lighting equipment, projection booth and necessary theatrical and mechanical equipment.

2.1.13.2 Small Assembly with Fixed Seats (I2)

This occupancy type shall include any building or portion thereof primarily meant for use as described for buildings under Occupancy I1, but with fixed seats for less than 1000 persons in a space. These assembly buildings may or may not be provided with a legitimate theatrical stage or related accessories or equipment.

2.1.13.3 Large Assembly without Fixed Seats (I3)

This occupancy type shall include any building or portion thereof for assembly in a space, in which there are no fixed seats, which may or may not be provided with a legitimate stage or theatrical accessories, and which has accommodation for 300 or more persons.

2.1.13.4 Small Assembly without Fixed Seats (I4)

This occupancy type shall include any building or portion thereof primarily intended for use as described in Occupancy I3, but with accommodation for less than 300 persons in a space.

2.1.13.5 Sports Facilities (I5)

This occupancy type shall include any building or portion thereof meant for assembly of spectators for recreational and amusement purpose mainly related to sports.

2.1.14 Occupancy J: Hazardous Buildings

Any Building or portion thereof used as storage, industrial, research and other facilities dealing with hazardous material in excess of exempted quantity defined in the Table 3.2.5 or any micro-biological facilities shall be categorized in this Occupancy group.

Definition of hazard and the amount of such materials which shall be deemed to render a building hazardous are set forth in Sec 2.14.3. This Occupancy shall be subdivided as follows:

2.1.14.1 Explosion Hazard Buildings (J1)

These shall include any building or portion thereof which is used for storage, handling, processing or manufacture of explosive materials and products that have explosion hazard.

2.1.14.2 Chemical Hazard Buildings (J2)

These shall include any building or portion thereof which is used for storage, handling, processing or manufacture of materials and products that are highly corrosive, toxic, poisonous and physically harmful including corrosive and toxic alkalis, acid or other liquids or chemicals, producing flame, fumes, radiation, and explosive, poisonous, irritant and corrosive gases.

2.1.14.3 Biological Hazard Buildings (J3)

These shall include any building or portion thereof which is used for storage, handling, processing or manufacture of materials and products that use biological processes and in which the risk of harmful biological threat to the occupants exist.

2.1.14.4 Radiation Hazard Buildings (J4)

These shall include any building or portion thereof which is used for storage, handling, processing or manufacture of materials and products that use nuclear and radioactive processes and in which the risk of radioactive contamination exists.

2.1.15 Occupancy K: Garage

These occupancy types shall include any building or portion thereof used one or more vehicles having containers of flammable liquid or compressed gas or carrying power or combination of any of these as a supply source for self-propelling are kept for use, sale, rental purpose, storage, repair, exhibition and all those floors of a building or portion thereof in which such vehicles are not separated by suitable cutoff to prevent fire spreading.

2.1.15.1 Parking Garage (K1)

This occupancy type shall include any building or portion thereof used solely for parking Motor Vehicles for a limited period of time.

2.1.15.2 Private Garage (K2)

This occupancy type shall include any building or portion thereof used as store of owner's or tenant's Motor Vehicles for private use for unlimited period of time.

2.1.15.3 Repair Garage and Showrooms (K3)

This occupancy type shall include any building or portion thereof wherein repair of electrical or mechanical system or denting or painting works of body is performed on any type of vehicles and includes associated floor spaces used as office, showrooms, incidental store and parking.

2.1.16 Occupancy L: Utility

This occupancy type shall include any building or portion thereof used to install any type of equipment to provide support service to any building or portion thereof or group of buildings of all occupancy groups and with special provisions for occupancy J.

This shall also include all public and private utility facilities of the consumer's end that are located within the consumer's site and all installations are required special care to ensure life and property safety as per provisions of this Code.

2.1.17 Occupancy M: Miscellaneous

Buildings under this Occupancy group shall include special buildings not covered in other Occupancy groups. These Occupancies shall be subdivided as follows:

2.1.17.1 Special Structure (M1)

Any building or structure which is neither listed in the A-Z list nor covered in any occupancy group provided in this Code but unique in character may be categorized in this occupancy by the Board of Appeals. Each and every individual M1 Structure shall be complied with NFPA or equivalent standards for the life and fire safety.

2.1.17.2 Fences, Tanks and Towers (M2)

These shall include fences and boundary walls over 1.5 m high, standalone structures for gravity water tank and towers for telecommunication, power distribution, air-traffic control terminal or observation towers.

2.2 Change of Use

2.2.1 Without prior permission from the Authorities having jurisdiction no change shall be made in the type of occupancy or use of any building that would place it in a different occupancy group or in a different subdivision of the same occupancy group. Such changes shall be permitted only when the land use and the building complied with the provisions of this Code and the laws of the land for such group of Occupancy.

2.3 Mixed Occupancy

2.3.1 The following occupancies shall not be required to designate as a separated occupancy classification from uses to which they are accessory any occupancy Group other than Occupancy Group J

(a) Assembly rooms having a floor area not more than 75 m^2.

(b) The administrative and clerical offices and similar offices not exceeding 25 Percent of the floor area of the major occupancy and not related to Hazardous Buildings as defined in Occupancy J.

(c) Administrative offices, gift shops and other similar uses in Occupancy A provided the uses do not exceed 10 Percent of the floor area of the major occupancy.

(d) Kitchens associated with a dining area.

(e) Carports having at least two sides entirely open associated with Occupancy A.

2.3.2 Forms of Occupancy Separations

A building is permitted to have multiple occupancy type, each type of occupancy shall be in groups, which may have combination of different occupancies and shall be separated horizontally or vertically or both accordingly as specified in the Table 3.2.1.

2.3.3 Types of Occupancy Separation

The occupancy separations shall be classified as follows:

(a) Four Hour Fire Resistive: The four hour fire resistive separation wall or slab shall have no unprotected openings therein and shall provide a fire resistance for at least four hour.

(b) Three Hour Fire Resistive: The three hour fire resistive separation wall or slab shall provide a fire resistance of not less than three hour. The total width of all openings in separation wall of any one storey shall not exceed 25 Percent of the length of that wall in that storey and no single opening shall have an area greater than 12 m^2. The openings shall be protected with a fire resistance assembly doors or windows providing fire resistance of at least three hour.

(c) In case of a floor slab having three hour fire resistance rating, the openings on floor slab shall be protected by vertical enclosures extended above and below such floor openings. The walls of such vertical enclosures shall be at least two hour of fire resistance. All openings in such enclosures shall be protected with fire assembly door or window having fire resistance rating of at least one and one-half hour.

(d) Two Hour Fire Resistive: The two hour fire resistive separation shall be of a construction having a fire resistance rating of not less than two hour. All openings in such separations shall be protected with a fire assembly door or window of a fire protection rating of at least one and one-half hour.

(e) One Hour Fire Resistive: The one hour fire resistive separation shall be of at least one hour fire protection construction. All openings in such separations shall be protected with a fire protection assembly door or window of at least one-half hour fire resistance.

Table 3.2.1: Fire Resistance Rating Requirements for Barrier Walls and Floor/Ceiling Assemblies between Separated Occupancies (hours)

[illegible]

2.4 GENERAL REQUIREMENTS OF ALL OCCUPANCIES

2.4.1 Location on Property

2.4.1.1 All plots for building construction shall have access to a public road from at least one side.

2.4.1.2 Fire separation distance shall be measured from the face of peripheral wall of a building to the adjacent property line. For the purpose of this Section, if a public road adjoining all along a property line shall get the benefit of half of Road width as a part of Fire separation distance. For two or more buildings on the same plot, distances of imaginary lines equidistant from all side of buildings shall be considered as the required fire separation distances.

2.4.1.3 The exterior walls of a building shall have a fire resistance and opening protection as specified in Tables 3.3.1 (a), 3.3.1 (b) and 3.2.3.

2.4.1.4 Any outward projected elements from the peripheral wall of a building line shall be limited to the sunshade line.

2.4.1.5 When openings in exterior walls are required to be protected due to distance from the property line, the aggregate area of such openings shall not exceed 50 Percent of the total area of the wall in each storey.

2.4.1.6 Dwellings separation walls in semi-detached or row type development shall comply with Sec 2.4.3.

2.4.2 Allowable Floor Areas

2.4.2.1 The total area of the building shall comply with Sec 1.8.3 Chapter 1 of this Part.

2.4.2.2 The floor area of the mezzanines shall be included in the area of the respective main floor.

2.4.2.3 Floor area calculation shall be divided in to two: (a) All Floor areas at and above the formation level which shall be generally included in the FAR calculation. (b) Floor areas below the formation level shall generally be excluded in FAR calculation provided the Occupancy classifications remain within Utility or Private Garages.

Table 3.2.2: Fire Resistance Ratings in Hours of Exterior Walls for Various Occupancy Groups

Fire Separation Distance	**Occupancy**			
	A1, A2, K2 , M2	**A3, A4, A5, B,C, D, E1, F1, F2, G1, I**	**E2, F3, F4, E3, G2, H1**	**H2, J**
Up to 1.5 m	1	2	3	4
Greater than 1.5 m and up to 3 m	N	1	2	3
Greater than 3 m and up to 4.5 m	N	N	1	2
Greater than 4.5 m and up to 9 m	N	N	N	1
Greater than 9 m	N	N	N	N

N= No requirements

Table 3.2.3: Requirements for Opening Protection Assembly Based on Fire Resistance Rating of Exterior Walls

Fire Resistance Ratings of Exterior Walls (in hours)	Fire Resistance Ratings for Opening Assembly (in hours)
4	Not permitted
3	3.0
2	1.5
1	0.5
N	No requirements

2.4.3 Permitted Types of Construction

2.4.3.1 The types of construction for any occupancy shall conform to the specifications set in Table 3.2.4.

2.4.3.2 Common walls in semi-detached or row type development shall not have any unprotected openings and shall be Type I-A construction and all such wall shall comply with requirements of Party wall or Fire wall or Separation wall.

2.4.3.3 Ground floor or basement of a building used for car parking and utilities within the barriers by at least three hour fire resistive construction shall be considered as non-separated occupancy provided the building accommodates one or more of the following occupancies:

(i) A3, A5

(ii) E1, F1, F2

(iii) I2, I3, I4

2.4.3.4 Entry lobbies, mechanical and electrical rooms and other similar uses incidental to the operation of the building may be provided in the car parking floors provided that the total area of such uses remains within ⅓ (one third) of the parking floor area.

Table 3.2.4: Permitted Types of Construction and Fire Zones for Various Occupancy Groups

Occupancy	Permitted Types of Construction	Fire Zones
A B C D E1 F1,F2 I K1, K2, M2	Group I and Group II*	1
E2, E3, F3, K3, M1 G H	Group I or Group II*	2
J	Group I	3

*Fire resistance rating of a building shall be credited in case of the mixed type of construction on the basis of lower rated construction elements among the same group or same type used thereof.

2.4.4 General Provision for High-Rise Buildings

For the purpose of this Code, a building of any class of Occupancy will be considered as high-rise when it has floors used for human occupancy located more than 33 m from ground level or the lowest level of fire department vehicle access. The provisions of Sec 2.9.6 shall be applicable to all such buildings.

2.4.4.1 Maintenance and inspection

All fire protection systems shall be maintained and inspected on a regular basis to keep them in operative condition. The maintenance inspection shall be performed quarterly.

All plumbing installations shall be maintained and inspected periodically to keep them in operative conditions.

2.4.4.2 Type of construction

All high-rise buildings shall be of Type I-A or I-B construction.

2.4.4.3 Fire detection, alarm, evacuation and extinguishment system

All high-rise buildings shall conform to regulations set forth in Part 4 of this Code

2.4.5 Helipads

2.4.5.1 General

Helipads on the roof top of a building or other locations shall be constructed in accordance with this Section.

2.4.5.2 Size

The minimum dimension of the landing area for helicopters weighing less than 1600 kg shall be 6 m × 6 m. There shall be an average clearance of 4 m surrounding and at the level of the landing area which shall not be less than 2 m at any point.

2.4.5.3 Construction

Helicopter landing areas and supports shall be constructed with non-combustible material.

2.4.5.4 Aviation approval

Before helipads start operating, formal approval shall be obtained from the civil aviation authority.

2.4.6 Universal Accessibility

2.4.6.1 All Building (except Occupancies G, H, M and J) shall have universal accessibility as per provisions of this Code.

2.4.6.2 Buildings have universal accessibility shall have accessible egress system.

2.5 Requirements For Occupancy A- Residential Buildings

Buildings shall be classified as Occupancy A in accordance with Sec 2.1.5.

2.5.1 Construction, Height and Allowable Area

2.5.1.1 Buildings or parts thereof classified as Occupancy A shall be limited to the type of construction set forth in Table 3.2.4 and shall not exceed in area or height as specified in Sections 1.8 and 2.4.2 of this Part.

2.5.1.2 Walls and floors separating dwelling units in the same building shall not be less than Type I-D construction.

2.5.1.3 Storage or laundry rooms in Occupancy A2, A3, A4 or A5 that are used in common by the occupants shall be at least Type I-D construction.

2.5.1.4 When a basement or a ground floor of a building of Occupancy A3 or A5 is used for parking or storage of private cars of the occupants, the parking floor shall be of at least Type I-B construction.

2.5.1.5 When the basement or ground floor of a building of Occupancy A is used wholly or partly for generator or electrical substation, the walls and floors surrounding such use shall be of at least Type I-B construction.

2.5.2 Location on Property

Buildings of Occupancy A shall comply with the requirements for location on property and fire resistive exterior walls and openings as specified in this Code.

2.5.3 Access and Exit Facilities and Egress System

2.5.3.1 Facilities for access and exit and egress or escape shall comply with the provisions set forth in this Code.

2.5.3.2 Every sleeping room in ground, first and second floors shall have at least one operable window or door for emergency escape which shall open directly into the exterior or an interior courtyard. The units shall be operable from the inside without the use of any tool to provide a minimum clear opening of 500 mm width by 600 mm height with a maximum sill height of 1 m above the floor.

2.5.4 Lighting and Ventilation

All buildings or part of a building classified as Occupancy A shall conform to the provisions of Part 3, and Chapters 1 and 3 of Part 8.

2.5.5 Sanitation

Sanitation facilities provided in all Occupancy A buildings shall conform to this Part and Chapter 7 Part 8.

2.5.6 Minimum Dimension of Habitable and Non-habitable Rooms

The minimum dimensions of habitable and non-habitable rooms are specified in Sec 1.12.2 Chapter 1 Part 3.

2.5.7 Fire detection, Alarm, Evacuation and Extinguishment

All buildings shall conform to regulations set forth in Part 4 of this Code.

2.5.8 Shaft and Exit Enclosure

Elevator shafts, vent shafts and other vertical openings shall be enclosed conforming to the provisions of Tables 3.3.1 (a) and (b). Exit requirements shall comply with Part 4 of this Code.

2.6 Requirements For Occupancy B - Educational Buildings

Buildings shall be classified as Occupancy B in accordance with Sec 2.1.6.

2.6.1 Construction, Height and Allowable Area

Buildings or parts of buildings classified as Occupancy B shall be limited to type of construction set forth in Table 3.2.4 and comply with the provisions of Sections 1.8 and 2.4.2 of this Part to meet the requirements of height and area limitations.

2.6.1.1 Rooms or groups of rooms sharing a common space where flammable liquids, combustible dust or hazardous materials are used, stored, developed or handled in an amount exceeding that specified in Sec 2.14.3 shall be classified as Occupancy J. Such rooms or groups of rooms shall comply with the requirements of fire protection as specified in Part 4, Chapters 4 and 5.

2.6.1.2 Rooms or groups of rooms, sharing a common space or having separate spaces, served by a common corridor or passage with less than 20 percent outdoor opening of wall in a building of height 11 m or less, or three storeys or less, need not be provided with smoke detectors and standpipe or sprinkler system for fire protection provided it conforms with the access and exit requirements specified in Part 3, Chapter 1, Sec 1.6 and Part 4, Chapters 4 and 5.

2.6.1.3 Buildings of Occupancy B situated outside the jurisdiction of any municipality shall have a construction of at least two hours fire resistance.

2.6.2 Location on Property

Buildings of Occupancy B shall comply with the requirements for location on property and fire resistive exterior walls and openings as specified in Sec 2.4.1.

2.6.3 Access and Exit Facilities and Egress System

Facilities for access and exit and Egress system shall comply with the provisions set forth in Sec 1.6, Chapter 1 Part 3 and Chapter 3 Part 4.

2.6.4 Lighting, Ventilation and Sanitation

Lighting, ventilation and sanitation facilities provided in Occupancy Group B buildings shall conform to Sec 1.16, Chapter 1 Part 3 and Chapters 1 and 3 Part 8.

2.6.5 Minimum Dimensions of Class Rooms, Common Toilets and Staircases

The dimension of a class room shall be not less than 4 m on any side and shall have an area of not less than $0.75 m^2$ per student. Other provisions for minimum dimensions shall comply with the requirements set forth in Sec 1.8 of Chapter 1 Part 3.

2.6.6 Shaft and Exit Enclosure

Elevator shafts, vent shafts and other vertical openings shall be enclosed conforming to the provisions of Tables 3.3.1 (a) and (b). Exit requirements shall comply with Chapter 3 Part 4.

2.6.7 Fire Detection, Alarm, Evacuation and Extinguishment System

All buildings shall conform to regulations set forth in Part 4 of this Code.

2.7 Requirements For Occupancy C- Institutional Buildings

Buildings shall be classified as Occupancy C in accordance with Sec 2.1.7.

2.7.1 Construction, Height and Allowable Area

The buildings or parts thereof classified as Occupancy C shall be limited to the type of construction set forth in Table 3.2.4 and shall comply with the provisions of Sec 1.8 Chapter 1 Part 3 and Sec 2.4.2 to meet the requirements of height and area limitations.

2.7.2 Location on Property

Buildings of Occupancy C shall comply with the requirements for location on property and fire resistive exterior walls and openings as specified in Sec 2.4.1.

2.7.3 Access and Exit Facilities and Egress System

Facilities for access and exit and egress system shall comply with the provisions set forth in Sec 1.6, Chapter 1 Part 3 and Chapter 3 Part 4.

2.7.4 Lighting, Ventilation and Sanitation

All buildings or part of a building classified as Occupancy C shall conform to the provisions of Sec 1.16, Chapter 1 Part 3 and Chapters 1 and 3, Part 8.

2.7.5 Shaft and Enclosure

Elevator shafts, vent shafts and other vertical openings shall be enclosed conforming to the provisions of Tables 3.3.1 (a) and (b). Exit requirements shall comply with Chapter 3, Part 4.

2.7.6 Fire Detection, Alarm, Evacuation and Extinguishment System

All buildings shall conform to regulations set forth in Part 4 of this Code.

2.8 Requirements For Occupancy D–Health Care Facilities

Buildings shall be classified as Occupancy D in accordance with Sec 2.1.8.

2.8.1 Construction, Height and Allowable Area

The buildings or parts thereof classified as Occupancy D shall be limited to the type of construction set forth in Table 3.2.4 and shall comply with the provisions of Sec 1.8 Chapter 1 Part 3 and Sec 2.4.2 to meet the requirements of height and area limitations.

2.8.2 Location on Property

Buildings of Occupancy D shall comply with the requirements for location on property and fire resistive exterior walls and openings as specified in Sec 2.4.1.

2.8.3 Access and Exit Facilities and Egress System

Facilities for access and exit and egress system shall comply with the provisions set forth in Sec 1.6 Chapter 1, Part 3 and Chapter 3 of Part 4.

2.8.4 Lighting, Ventilation and Sanitation

All buildings or part of a building classified as Occupancy D shall conform to the provisions of Sec 1.16 Chapter 1 Part 3, Chapters 1 and 3 of Part 8.

2.8.5 Shaft and Enclosure

Elevator shafts, vent shafts and other vertical openings shall be enclosed conforming to the provisions of Tables 3.3.1 (a) and (b). Exit requirements shall comply with Chapter 3 of Part 4.

2.8.6 Fire Detection, Alarm, Evacuation and Extinguishment System

All buildings shall conform to regulations set forth in Part 4 of this Code.

2.9 Requirements For Occupancy E–Business

Buildings shall be classified as Occupancy E in accordance with Sec 2.1.9.

2.9.1 Construction, Height and Allowable Area

The buildings or parts thereof classified as Occupancy E shall be limited to the type of construction set forth in Table 3.2.4 and shall comply with the provisions of Sec 1.8 Chapter 1 Part 3 and Sec 2.4.2 to meet the requirements of height and area limitations.

2.9.2 Location on Property

Buildings of Occupancy E shall comply with the requirements for location on property and fire resistive exterior walls and openings as specified in Sec 2.4.1.

2.9.3 Access and Exit Facilities and Egress System

Facilities for access and exit and egress system shall comply with the provisions set forth in Sec 1.6 Chapter 1 Part 3, Chapter 3 of Part 4.

2.9.4 Lighting, Ventilation and Sanitation

All buildings or part of a building classified as Occupancy E shall conform to the provisions of Sec 1.16 Chapter 1 Part 3, Chapters 1 and 3 of Part 8.

2.9.5 Shaft and Enclosure

Elevator shafts, vent shafts and other vertical openings shall be enclosed conforming to the provisions of Tables 3.3.1 (a) and (b). Exit requirements shall comply with Chapter 3 of Part 4.

2.9.6 Fire Detection, Alarm, Evacuation and Extinguishment System

All buildings shall conform to regulations set forth in Part 4 of this Code.

2.10 Requirements For Occupancy F–Mercantile Buildings

Buildings shall be classified as Occupancy F in accordance with Sec 2.1.10.

2.10.1 Construction, Height and Allowable Area

The buildings or parts thereof classified as Occupancy F shall be limited to the type of construction set forth in Table 3.2.4 and shall comply with the provisions of Sec 1.8, Chapter 1 of Part 3 and Sec 2.4.2 to meet the requirements and limitations of height and area.

2.10.2 Location on Property

Buildings of Occupancy F shall comply with the requirements for location on property and fire resistive exterior walls and openings as specified in Sec 2.4.1.

2.10.3 Access and Exit Facilities and Emergency Escapes

Facilities for access and exit and emergency escape shall comply with the provisions set forth in Sec 1.6 Chapter 1 Part 3 and Chapter 3 Part 4.

2.10.4 Lighting, Ventilation and Sanitation

All buildings or part of a building classified as Occupancy F shall conform to the provisions of Sec 1.16 Chapter 1 Part 3, Chapters 1 and 3, Part 8.

2.10.5 Shaft and Enclosure

Elevator shafts, vent shafts and other vertical openings shall be enclosed conforming to the provisions of Tables 3.3.1 (a) and (b). Exit requirements shall comply with Chapter 3, Part 4.

2.10.6 Fire Detection, Alarm, Evacuation and Extinguishment System

All buildings shall conform to regulations set forth in Part 4 of this Code.

2.10.7 Special Hazards

Installations which are discharging exhaust, heating apparatus, boiler and central heating/air-conditioning plant shall conform to the provisions of this Code as specified in this Code.

2.11 Requirements For Occupancy G–Industrial Buildings

Buildings shall be classified as Occupancy G in accordance with Sec 2.1.11. A non-exhaustive and indicative list of low hazard and moderate hazard industrial uses are listed in A to Z list. Storage and use of hazardous materials shall not exceed the exempt amount specified in Sec 2.14.3.

2.11.1 Construction, Height and Allowable Area

The buildings or parts thereof classified as Occupancy G shall be limited to the type of construction set forth in Table 3.2.4 and shall comply with the provisions of Sec 1.8 of Chapter 1, Part 3 and Sec 2.4.2 to meet the requirements and limitations of height and floor area.

The ceiling height of the production area, shall confirm to the minimum volume required per workers as specified by the Bangladesh Labor Act, 2006 and other laws of the land. In any case the ceiling height and the head room clearance of a production floor shall not be less than 3.3 meter and 2.286 meter respectively.

2.11.2 Location on Property

Buildings of Occupancy G shall comply with the requirements for location on property and fire resistive exterior walls and openings as specified in Sec 2.4.1.

2.11.3 Access and Exit Facilities and Egress System

Facilities for access and exit and emergency escape shall comply with the provisions set forth in Sec 1.6 Chapter 1, Part 3 and Chapter 3, Part 4.

2.11.4 Lighting, Ventilation and Sanitation

All buildings or part of a building classified as Occupancy G shall conform to the provisions of Sec 1.16 Chapter 1, Part 3 and Chapters 1 and 3, Part 8. Industrial buildings having roof opening for day lighting and natural ventilation shall comply with the following requirements:

(a) The aggregate opening in roof and external windows shall not be less than 10 Percent of the floor area.

(b) For natural ventilation by means of exterior window openings, the operable window area shall not be less than 5 Percent of the total floor area.

Exception:

Industrial buildings wherein artificial lighting and mechanically operated ventilation systems of approved quality are installed, need not be provided with natural ventilation or natural lighting.

2.11.5 Shaft and Enclosure

Elevator shafts, vent shafts and other vertical openings shall be enclosed conforming to the provisions of Tables 3.3.1 (a) and (b). Exit requirements shall comply with Chapter 3, Part 4.

2.11.6 Fire Detection, Alarm, Evacuation and Extinguishment System

All buildings shall conform to regulations set forth in Part 4 of this Code.

2.11.7 Special Hazards

Chimneys, vents and ventilation ducts shall be constructed with noncombustible materials. Every bailer, central heating plants, electrical rooms, or hot water supply boiler shall be separated from the rest of the occupancy or use by not less than two hour fire resistive construction.

2.12 Requirements For Occupancy H–Storage Buildings

Buildings shall be classified as Occupancy H in accordance with Sec 2.1.12.

2.12.1 Construction, Height and Allowable Area

The buildings or parts thereof classified as Occupancy H shall be limited to the type of construction set forth in Table 3.2.4 and shall comply with the provisions of Sec 1.8 of Chapter 1, Part 3 and Sec 2.4.2 to meet the requirements of height and area limitations.

2.12.2 Location on Property

The location on property for Occupancy H shall conform to Sec 2.4.1.

2.12.3 Access and Exit Facilities and Egress System

Facilities for access and exit and egress system shall comply with the provisions set forth in Sec 1.6 of Chapter 1, Part 3 and Chapter 3, Part 4.

2.12.4 Lighting, Ventilation and Sanitation

All buildings or part of a building classified as Occupancy H shall conform to the provisions of Sec 1.16 of Chapter 1 Part 3, Chapters 1 and 3, Part 8.

2.12.4.1 Special provision

The provisions of Sec 1.16, does not apply to non-habitable spaces of H1 and H2 occupancies unless otherwise required by this Code. Ventilators of size not less than 0.25 m^2 shall be provided where suitable 0.30 m above the floor level for floor level ventilators and 0.30 m below the roof level for roof level ventilators. There shall be one floor level ventilator and one roof level ventilator for every 0.25 m^2 of the floor area. Mechanized ventilation system of approved quality shall be installed where required.

2.12.4.2 Though inhabitable, the minimum air quality of such indoor spaces shall be maintained in a way that it does not pose any health hazard to the occasional users of that space.

2.12.5 Shaft and Enclosure

Elevator shafts, vent shafts and other vertical openings shall be enclosed conforming to the provisions of Tables 3.3.1 (a) and (b). Exit requirements shall comply with Chapter 3, Part 4.

2.12.6 Fire Detection, Alarm, Evacuation and Extinguishment System

All buildings shall conform to regulations set forth in Part 4 of this Code.

2.12.7 Special Hazards

The storage of hazardous materials shall not exceed the exempt amount as specified in Table 3.2.5. The storage of moderate and low hazardous materials shall be separated at least by a two hour fire resistive construction.

2.13 Requirements For Occupancy I–Assembly Buildings

Buildings shall be classified as Occupancy I in accordance with Sec 2.1.13.

2.13.1 Construction, Height and Allowable Area

The buildings or parts thereof classified as Occupancy I shall be limited to the type of construction set forth in Table 3.2.4 and shall comply with the provisions of Sec 1.8 Chapter 1 Part 3 and Sec 2.4.2 to meet the requirements and limitations of height and area.

2.13.2 Location on Property

Buildings of Occupancy I shall comply with the requirements for location on property and fire resistive exterior walls and openings as specified in Sec 2.4.1.

2.13.3 Access and Exit Facilities and Egress System

Facilities for access and exit and Egress system shall comply with the provisions set forth in Sec 1.6 Chapter 1 of Part 3 and Chapter 3 of Part 4 and universally accessibility as per provisions of this Code.

2.13.4 Lighting, Ventilation and Sanitation

All buildings or part of a building classified as Occupancy I shall conform to the provisions of Sec 1.16 Chapter 1 Part 3, Part 3 and Chapters 1 and 3, Part 8.

2.13.5 Shaft and Enclosure

Elevator shafts, vent shafts and other vertical openings shall be enclosed conforming to the provisions of Tables 3.3.1 (a) and (b). Exit requirements shall comply with Chapter 3, Part 4.

2.13.6 Fire Detection, Alarm, Evacuation and Extinguishment System

All buildings shall conform to regulations set forth in Part 4 of this Code.

The specification of this Section shall apply to all parts of buildings and structures that contain stages or platforms and other similar appurtenances as herein defined.

(a) Stages: A stage is a three side enclosed or partially enclosed portion of a building which is designed or used for presentation of plays or lectures or other entertainment. A stage shall be further classified as legitimate stage, regular stage and thrust stage.

(b) Stage, Legitimate: A stage wherein curtains, drops, leg drops, scenery, lighting devices or other stage effects are adjustable horizontally or vertically or suspended overhead.

(c) Stage, Regular: A stage wherein curtains, fixed drops, valances, scenery and other stage effects are suspended and are not adjustable or retractable.

(d) Stage, Thrust: A stage or platform extended beyond the proscenium line and into the audience.

2.13.6.1 Legitimate Stage

Legitimate stage shall be constructed as specified in Part 4, specifying the type of construction but shall not be less than construction Type I-C. The position of the legitimate stage extending beyond the proscenium opening line shall be permitted to be constructed with two hour fire-resistive materials. The floor of the stage may be constructed with one hour fire rating materials. Thickness of a wooden floor shall not be less than 50 mm.

2.13.6.2 Regular and Thrust Stages

Regular stages and thrust stages shall be constructed by not less than two hour fire resistive materials. Wooden floor when required in a stage shall not be less than 50 mm in thickness with one hour fire resistive rating.

2.13.6.3 Trap doors

All trap doors and any other opening in stage floors shall be equipped with tight fitting solid wood trap doors with thickness not less than 50 mm.

2.13.6.4 Stage rigging loft

The grid iron frame in the loft, housing lighting and audio equipment, all the machinery for flying scenery and fly galleries, along with their installations, shall be constructed of approved noncombustible materials.

2.13.6.5 Foot lights and stage electrical equipment

Foot lights and border lights shall be installed in a protective cover constructed of noncombustible materials.

2.13.6.6 Trim, finish and decorative hangings

All materials used in moulding and decoration around the proscenium shall be of approved noncombustible materials.

2.13.6.7 Proscenium curtain

The proscenium curtain shall be of approved fire retardant material and shall protect against passage of flame and smoke for at least 30 minutes.

2.13.7 Motion Picture Projection Rooms

2.13.7.1 Every projection room shall be constructed in conformity with the construction requirements for the type of the building in which the projection room is located. The wall opening required for projection need not have a fire protection assembly but shall be closed with glass or other approved materials.

2.13.7.2 The floor area of a projection room shall not be less than 8 m^2 for a single machine. The working space between the machines when more than one machine is used shall not be less than 0.75 m.

2.13.7.3 The height of the projection room shall have a minimum clear space of 2.5 m.

2.13.8 Sports Facilities

2.13.8.1 Vomiters, aisles and exits of seating galleries

Tunnels, aisles and exits of galleries shall be constructed conforming to the following requirements.

(a) There shall be a minimum of two exits remotely located from each other immediately to the outside for each balcony or tier. There shall be at least three exits when seating capacity exceeds 1000 persons and four exits when it exceeds 4000 persons. For every additional 1000 persons the exit shall be designed to accommodate provision (f) given below.

(b) There shall be at least 0.6 m^2 of space per person in the gallery. Minimum width considered for a seat in the gallery shall be 0.45 m.

(c) There shall be a maximum of 33 seats on each side of any aisle. Minimum width of the main aisles and the secondary aisles shall be 1.0 m and 0.7 m respectively.

(d) Entrance and exits shall be protected by safety railings.

(e) Back to back space between two rows of seats shall not be less than 0.80 m.

(f) The evacuation time in the galleries shall not be more than 10 minutes.

(g) All tunnels, aisles and exits shall conform to safety guidelines for means of escape set forth in Part 4.

(h) One percent of the total seat capacity shall have provisions for accommodation with universal accessibility at the approach or exit level.

2.13.8.2 Swimming pools

Any swimming pool used or constructed for exclusive use by Occupancy A1 and is available only to the occupants and private guests shall be classified as a private swimming pool. Any swimming pool other than private swimming pool shall be classified as a public swimming pool. Swimming pools shall be constructed in conformity with the following requirements.

(a) There shall be at least 1.5 m space between any sides of a swimming pool and a rear or side property line. For street property lines, this distance shall be at least 2.0 m.

(b) Swimming pools shall be provided with overflow provision to remove scum and other materials from the surface of the water. When water skimmers are used for private pools there shall be one skimming device for each 50 m^2 of surface area or fraction thereof.

(c) The overflow gutters shall not be less than 75 mm deep and shall be pitched to slope of one unit vertical to 50 units horizontal (1:50) toward drains.

(d) Public swimming pools shall be so designed that the pool water turnover is at least once every 8 hours.

(e) Private swimming pools shall be designed so that there is a pool water turnover at least once every 18 hours.

(f) Public swimming pools shall be equipped with filters, the capacity of which shall be controlled to filter 140 liters per minute per m^2 of surface area. Private swimming pool filters shall not filter more than 230 liters per minute per m^2 of the surface area.

(g) The pH value of the pool water shall be between 7.0 and 7.5.

(h) All recirculation systems shall be equipped with an approved hair and lint strainer installed in the system ahead of the pump.

(i) All swimming pool and equipment shall be designed to be emptied completely of water and the discharged water shall be disposed in an approved manner and shall not create problems in the neighboring property.

(j) Pumps, filters and other mechanical and electrical equipment shall be placed in enclosed spaces and shall not be accessible to the bathers.

(k) Used water from the pool when being discarded shall be reused as grey water for the building and its premises as per provision of Appendix G.

2.13.9 Amusement Building Fire Protection System

The fire protection system shall be as per provisions of this Code.

2.14 Requirements For Occupancy J–Hazardous Buildings

Buildings shall be classified as Occupancy J in accordance with Sec 2.1.14.

2.14.1 General

The plans for buildings and structures accommodating Occupancy J shall clearly indicate the type and intended use of materials and its processing or handling methods so as to reflect the nature of use of each portion of such buildings.

2.14.1.1 Occupancy J1

Any building or portion thereof containing any of the following items more than exempted quantity shall be classified as Occupancy J1.

(a) Combustible dusts and any similar solid material sufficiently comminuted for suspension in still air which, when so suspended, is capable of self-sustained combustion.

(b) Combustible liquids - Any liquid having a flash point at or above 40°C shall be known as class II and class III liquids. Combustible liquids shall be classified as follows:

(i) Liquids having flash point at or above 40°C and below 60°C.

(ii) Liquids having flash points at or above 60°C and below 95°C.

(c) Cryogenic liquids (flammable or oxidizing): Any liquid that has a boiling point below -130°C.

(d) Flammable Gases: Any gas when mixed with air in a proportion of 13% (by volume) forms a flammable mixture under atmospheric temperature and pressure.

(e) Flammable Liquids: Any liquid that has a flash point below 40°C and has a net vapour pressure exceeding 275 kPa at 40°C. Flammable liquids shall be known as Class I liquid and shall be further classified as follows:

(i) Liquids having flash point below 25°C and having a boiling point below 40°C.

(ii) Liquids having flash point below 25°C and having a boiling point at or above 40°C.

(iii) Liquids having flash points at or above 25°C and below 40°C.

(f) Oxidizers class 3: As determined in accordance with NFPA 43A.

(g) Oxidizing gases: As determined in accordance with NFPA 43C.

(h) Pyrophoric liquids, solids and gases that will ignite spontaneously in air at a temperature of 55°C or below.

(i) Unstable (reactive) materials class 3, non-detonable as determined in accordance with NFPA 704.

(j) Combustible fibers: Includes readily ignitable fibers like cotton, sisal, jute hemp, tow, cocoa fiber, oakum, baled waste, baled waste paper, kapok, hay, straw, excelsior, Spanish moss and other similar materials.

(k) Flammable solid: Any solid including blasting agent or explosive that is liable to cause fire through absorption of moisture, spontaneous chemical change or retained heat from manufacturing or processing, or which when ignited burns so vigorously and persistently as to create a serious hazard.

(l) Organic peroxides, Class II and Class III as determined in accordance with NFPA 43B.

(m) Oxidizers Class I and Class II as determined in accordance with NFPA 43A.

(n) The bulk storage of unstable (reactive) materials Class 1 and Class 2 as determined in accordance with NFPA 704, water reactive materials, Class 2 and Class 3 which react with water to release a gas that is either flammable or present a health hazard as determined in accordance with NFPA 704.

2.14.1.2 Occupancy J2

Any building or portion thereof containing the following shall be classified as Occupancy J2:

(a) Corrosives: Any substance that causes visible destruction of or irreversible alteration in living tissues by chemical action at the site of contact.

(b) Highly toxic materials: The materials falling in this category are as follows:

(i) Oral Toxicity: A chemical that has a median lethal dose of 50 mg or less per kg of body weight when administered orally to albino rats weighing between 200 and 300 gm each.

(ii) Toxicity of Inhalation: A chemical that has a median lethal concentration in air of 200 ppm or less by volume of gas or vapors, or 2 mg per liter or less of mist, fume or dust, when administered by continuous inhalation for 1 hour (or less if death occurs within 1 hour) to albino rats weighing between 200 and 300 grams each.

(iii) Toxicity by Skin Absorption : A chemical that has median lethal dose of 200 mg or less per kg of body weight when administered by continuous contact for 24 hours (or less if death occurs within 24 hours) with the bare skin of albino rabbits weighing between 2 and 3 kg each.

(iv) Irritants: Any noncorrosive chemical or substance which causes a reversible inflammatory effect on living tissues by chemical action at the site of contact.

(v) Radioactive Material: Any material or combination of materials that spontaneously emit ionizing radiation.

(vi) Sensitizers: A chemical or substance that causes a substantial proportion of exposed people or animals to develop an allergic reaction in normal tissue after repeated exposure.

(c) The Occupancy J2 shall also include among others the followings:

(i) Dry cleaning establishments using flammable solvents.

(ii) Explosive manufacturing.

(iii) Paint or solvent manufacturing (flammable base).

(iv) Pyrexin plastic manufacturing.

(v) Sodium nitrate or ammonium nitrate.

(vi) Storage of combustible film.

2.14.1.3 Occupancy J3

Any building or portion thereof which is used for storage, handling, processing or manufacture of materials and products that use biological processes and in which the risk of harmful biological threat to the occupants exist, shall comply with the guidelines specified by the Department of Health.

2.14.1.4 Occupancy J4

Any building or portion thereof which is used for storage, handling, processing or manufacture of materials and products that use nuclear and radioactive processes and in which the risk of radioactive contamination exists, shall comply with the guidelines specified by Bangladesh Atomic Energy Commission.

2.14.2 Special Provisions

2.14.2.1 The following shall not be included in Occupancy J but shall be classified in the occupancy group which they most nearly resemble and such classification shall be approved by the Authority:

(a) All buildings and structures and parts thereof which contain less than the exempt quantities as specified in Table 3.2.5, when such buildings comply with the fire protection provisions of this Code.

(b) Rooms containing flammable liquid in lightly closed containers of 4 litre capacity or less for retail sales or private use on the premises and in quantities not exceeding 820 litres/m² of room area.

(c) Retail paint sales rooms with quantities not exceeding 820 litres/m² of room area.

(d) Closed systems housing flammable or combustible liquids or gases used for the operation of machinery or equipment.

(e) Cleaning establishments.

(f) Liquor stores and distributors without bulk storage.

(g) Tire storage containing less than 10,000 vehicle tires.

(h) The storage or use of materials for agricultural purposes for use on the premises.

(i) Pyrophoric solids or liquids not exceeding 3 m^3 in storage cabinet located in a building that is equipped throughout with an automatic sprinkler system provided in accordance with the fire protection provisions of this Code.

(j) Pyrophoric solids or liquids not exceeding 3 kg in storage cabinet located in a building that is provided with an automatic sprinkler system installed in accordance with the fire protection provisions in accordance to Part 4 of this Code.

(k) Class 2 water reactive materials not exceeding 100 kg in an approved storage cabinet located in a building that is provided with automatic sprinkler installed in accordance with the fire protection provisions in accordance to Part 4 of this Code.

2.14.3 Construction, Height and Allowable Area

2.14.3.1 The buildings or parts thereof classified as Occupancy J shall be limited to the type of construction set forth in Table 3.2.4 and shall comply with the provisions of Sec 1.8 of Chapter 1, Part 3 and Sec 2.4.2 of this Chapter to meet the requirements of height and area limitations.

2.14.3.2 Floors: The floors and spaces containing hazardous materials and in areas where motor vehicles, boats, helicopters or airplanes are stored, repaired or operated shall be of noncombustible, liquid-tight construction.

Exception: In floors and areas where no repair works are carried out may be surfaced or waterproofed with asphaltic paving materials.

2.14.3.3 Spill Control: The floors containing hazardous repair or other works shall be recessed a minimum of 100 mm so as to prevent flow of liquids to adjoining areas.

2.14.3.4 Drainage: The buildings and areas shall be provided with approved drainage system to direct the flow of liquids to an approved location or room or area designed to provide secondary containment of the hazardous materials and fire protection water.

Table 3.2.5(a): Exempted Amount of Hazardous Materials in Terms of Physical Hazard in a Control Area

Sl. No.	Material	Class/State	Maximum Quantities in		
			Storage Limit	Use Closed Systems	Use Open Systems
1	Flammable liquids	Class I-A Class I-B and Class I-C	115 liters * 454 liters *	115 liters * 454 liters *	38 liters 115 liters
2	Combustible liquids	Class-II Class-III-A Class-III-B	454 liters* 1249 liters* 49962 liters*	454 liters* 1249 liters* 49962 liters*	114 liters 320 liters 12490 liters
3	Combination of flammable liquids	Class I-A Class I-B Class I-C	454 liters*	454 liters*	113 liters*
4	Flammable gases	Gaseous Liquefied	28 m^3 at NTP (Natural Temperature and Pressure) 113 liters	28 m^3 at NTP (Natural Temperature and Pressure) 113 liters	Not applicable Not applicable
5	Liquefied flammable	Class I-A Class I-B and Class I-C	113 liters 454 liters	113 liters 454 liters	38 liters 113 liters
6	Combustible fibres	Loose Baled	2.832 m^3 28.32 m^3	2.832 m^3 28.32 m^3	0.57 m^3 5.7 m^3

Sl. No.	Material	Class/State	Maximum Quantities in		
			Storage Limit	Use Closed Systems	Use Open Systems
7	Flammable solids	Pigs, ingots, heavy castings	454 kg	454 kg	454 kg
		Light castings, light metallic products	57 kg	57 kg	57 kg
		Scraps, shavings, powders, dusts	0.454 kg	0.454 kg	0.454 kg
8	Unstable (reactive) detonable	Class 4	0.454kg or $0.28m^3$ (NTP)	0.113 kg or $0.057m^3$ (NTP)	0.454kg or $0.28m^3$ (NTP)
		Class 3	0.454kg or $0.28m^3$ (NTP)	0.113 kg or $0.057m^3$ (NTP)	0.454kg or $0.28m^3$ (NTP)
9	Unstable (reactive) detonable	Class 4	0.454kg or $0.28m^3$ (NTP)	0.113 kg or $0.057m^3$ (NTP)	0.454kg or $0.28m^3$ (NTP)
		Class 3	2.27 kg or $1.42m^3$ (NTP)	0.454kg or $0.2832m^3$ (NTP)	0.454kg
		Class 2	22.7kg or 70.8 m^3 (NTP)	22.7kg or $70.8m^3$ (NTP)	4.54 kg
		Class 1	Not limited or $21.24m^3$ (NTP)	Not limited	Not limited
10	Water-reactive detonable	3	0.454 kg	11.25 kg	11.25 kg
		2	0.454 kg	11.25 kg	11.25 kg
11	Water-reactive non-detonable	3	2.27 kg	2.27 kg	0.454 kg
		2	22.7 kg	22.7 kg	4.54 kg
		1	Not limited	Not limited	Not limited
12	Oxidizing Materials	Class 4	0.454 kg	0.1135kg	0.1135kg
		Class 3	4.54 kg	0.227kg	0.227kg
		Class 2	113 kg	113 kg	113 kg
		Class 1	1816 kg	1816 kg	1816 kg

Sl. No.	Material	Class/State	Maximum Quantities in		
			Storage Limit	Use Closed Systems	Use Open Systems
13	Oxidizing Gas	Gaseous	42.48 m^3 (NTP)	42.48 m^3 (NTP)	Not applicable
		Liquefied	56.78 liters	56.78 liters	Not applicable
14	Pyrophoric Material detonable	Not applicable	0.454 kg or 0.056 m^3 (NTP)	0.056 m^3 (NTP)	0
15	Pyrophoric Material non-detonable	Not applicable	1.8 kg. or 1.4 m^3(NTP)	0.28m^3 (NTP)	0
16	Explosives**	Division 1.1	0.454 kg	0.1135 kg	0.1135 kg
		Division 1.2	0.454 kg	0.1135 kg	0.1135 kg
		Division 1.3	2.27 kg	0.454 kg	0.454 kg
		Division 1.4	22.7 kg	22.7 kg	Not applicable
		Division 1.4G	56.75 kg	Not applicable	Not applicable
		Division 1.5	0.454 kg	0.1135 kg	0.1135 kg
		Division 1.6	0.454 kg	Not applicable	Not applicable

* The maximum quantities may be increased by 100 percent in areas not accessible to the public in buildings provided with automatic sprinkler system.

** see: Explosive control act.

Table 3.2.5(b): Exempted Amounts of Hazardous Materials in Terms Health Hazard in a Control Area

Material	Class/State	Maximum Quantities in		
		Single Storage	Closed Systems	Open Systems
Corrosive	Not applicable	2270 kg or 1892 liters or 23 m^3 NTP	227kg or 1892 liters or 23 m^3 NTP	454kg or 379 liters
Highly toxic	Not applicable	4.54 kg or 0.57 m^3 NTP	4.54 kg or 0.57 m^3 NTP	1.362 kg
Toxic	Not applicable	227 kg or 23 m^3 NTP	227 kg	56.75 kg

Table 3.2.5(c): Location and Number of Control Areas

Grade Level	Floor Level[1]	Number of Control Areas per Floor[2]	Fire Resistance Rating of Barriers in Hours		
			Walls	Floors	Floor Supporting Members
Above	Higher than 9	5	1	2	2
	7-9	5	2	2	2
	6	12.5	2	2	2
	5	12.5	2	2	2
	4	12.5	2	2	2
	3	50	2	1	2
	2	75	3	1	2
	1	100	4	1	2
Below	1	75	3	1	2
	2	50	2	1	2
	Lower than 2	Not Allowed	Not Allowed	Not Allowed	Not applicable
	The maximum allowable quantity per control area shown in Table 3.2.5				

2.14.3.5 The drains shall be designed with adequate slope and section to carry the design discharge of the sprinkler system. The material used in the drains shall be suitable for drainage of the storage materials.

2.14.3.6 Separate drainage system shall be designed for materials which react with each other producing undesirable results. They may be combined when they have been provided with approved means of discharge into the public sewer or natural stream or river.

2.14.3.7 Containment: The outflow from the drains shall be directed to a containment system or other area that provide a secondary storage for the hazardous materials and liquids and fire protection water. The containment capacity shall be capable of containing the outflow from the drains for a period of at least one hour.

2.14.3.8 The overflow from secondary containment system shall be directed to a safe location away from the building, adjoining properties and storm drain.

2.14.3.9 If the secondary containment storage area is open to rainfall it shall be designed to accommodate 24 hour rainfall or a continuous rainfall of 100 mm per day.

2.14.3.10 Smoke and Heat Vents: Smoke and heat vents shall be provided in areas or rooms containing hazardous materials exceeding the exempt amount of Table 3.2.5.

2.14.3.11 Standby Power: Standby power shall be provided in the occupancies where Class I, II or III organic peroxides are stored.

2.14.4 Location on Property

The location on property for Occupancy J shall conform to Sec 2.4.1 and Part 4.

2.14.5 Access and Exit Facilities and Emergency Escapes

Facilities for access and exit and emergency escape shall comply with the provisions set forth in Sec 1.6 of Chapter 1, Part 3, and Chapter 3, Part 4.

2.14.6 Lighting and Ventilation

2.14.6.1 All spaces and rooms customarily occupied by human beings shall be provided with natural light by means of exterior glazing with an area of not less than 10 Percent of the floor area. Such rooms and spaces shall be provided with natural ventilation by means of exterior openings with an open able area not less than 5 Percent of the total floor area or artificial light and mechanically operated ventilation system as per provisions of this Code.

2.14.6.2 Ventilation in Hazardous Locations: The rooms, spaces or areas where explosive, corrosive, combustible, flammable or highly toxic dust, mists, fumes, vapors or gases are stored or may be emitted due to the processing, use, handling or storage of materials shall be mechanically ventilated.

2.14.6.3 The mechanical ventilation of all hazardous uses shall be segregated or separated from the ventilation of other areas. The emissions generated at work areas shall be confined to the area in which they are generated and shall be removed or discharged outside the building and preventive measures against back flow of such hazardous fumes or gases inside the building shall be installed.

2.14.6.4 Ventilation of Toilets: Toilets shall be provided with fully openable exterior window of at least 0.3 m^2 in area or a vertical duct not less than 62500 mm^2 in cross-section for the first water closet, with additional 31250 mm^2 for each additional fixture or a mechanically operated exhaust system equipped to provide a complete change of air in every 15 minutes. Such system shall be connected to the outside air and the point of discharge shall be at least 1.0 m away from any other opening into the building.

2.14.6.5 Other requirements of water closets are specified in Sec 1.12.4 Chapter 1, Part 3.

2.14.7 Sanitation

All buildings or part of a building classified as Occupancy J shall conform to the provisions of Sec 1.16 of this Chapter and Part 8 of this Code.

2.14.8 Shaft and Exit Enclosures

Elevator shafts, vent shafts and other vertical openings shall be enclosed conforming to the provisions of Tables 3.3.1 (a) and (b). Exit requirements shall comply with Chapter 3, Part 4.

2.14.9 Fire Detection, Alarm, Evacuation and Extinguishment System

All buildings shall conform to regulations set forth in Part 4 of this Code.

2.14.10 Explosion Control

Explosion control, equivalent protective devices or suppression systems or barricades shall be installed to control or vent the gases resulting from deflagrations of dusts, gases or mists in a room or area, building or other enclosures to minimize structural or mechanical damage.

Walls, floors and roofs separating a use from explosion exposure shall be designed according to the provisions of Chapter 1, Part 6.

Explosion venting shall be designed in exterior walls or roof only. The venting shall be provided to prevent serious structural damage and production of lethal projectiles. The venting design shall recognize the natural characteristics and behaviors of building materials in an explosion. The vents shall be designed to relieve at a maximum internal pressure of 1.0 kPa but not less than the loads required by Chapter 2, Part 6. One or more of the following systems shall be installed to relieve explosion, where applicable:

(a) Lightweight materials in walls

(b) Light fastening devices with hatch covers

(c) Light fastening with outward opening swing doors in exterior walls

(d) Nonbearing walls with light ties

The venting devices shall discharge vertically or horizontally directly to an unoccupied yard having a width of not less than 16 m on the same plot.

The releasing devices shall be so located that the discharge end shall not be less than 3 m vertically and 6 m horizontally from window openings or exits in the same or adjoining buildings.

2.14.11 Special Hazard

Chimneys, vents and ventilation ducts shall be of noncombustible materials.

All boilers, central heating plants, electrical rooms or hot water supply boiler shall be separated from the rest of the occupancies or uses by not less than 2 hour fire resistive construction.

The devices that generate a spark, flame or glow capable of igniting gasoline shall not be installed or used within 0.5 m of the floor.

Equipment or machinery that produces or emits combustible or explosive dust or fibers shall be provided with an approved dust collecting and exhaust system.

The equipment or systems that are used to collect or process or convey combustible dust or fibers shall be installed with explosion venting or containment system.

2.15 Requirements For Occupancy K–Garage Buildings

Buildings shall be classified as Occupancy K in accordance with Sec 2.1.15.

Exception: Non-separated use mentioned in Sec 2.3.1.

2.15.1 Construction, Height and Allowable Area

The buildings or parts thereof classified as Occupancy K shall be limited to the type of construction set forth in Table 3.2.4 and Sec 2.4.4.2 and shall comply with the other provisions of Sec 1.8 Chapter 1 Part 3, Appendix F and Sec 2.4.2 to meet the requirements and limitations of height and area. With the exceptions mentioned in Sec 2.4.3, all garage floors shall be constructed with not less than 4 hour fire resistance materials.

2.15.1.1 Floors: The floors and spaces where motor vehicles are stored, repaired or operated shall be of noncombustible, liquid-tight construction.

Exception: In floors and areas where no repair works are carried out may be surfaced or waterproofed with asphaltic paving materials.

2.15.1.2 Spill Control: The floors containing hazardous repair or other works shall be recessed a minimum of 100 mm so as to prevent flow of liquids to adjoining areas.

2.15.1.3 Drainage: The buildings and areas shall be provided with approved drainage system to direct the flow of liquids to an approved location or room or area designed to provide secondary containment of the hazardous materials and fire protection water.

The drains shall be designed with adequate slope and section to carry the design discharge of the sprinkler system. The material used in the drains shall be suitable for drainage of the storage materials.

The quality of discharged liquids must attain approved level before discharging into the public sewer or natural stream or river.

2.15.1.4 Smoke and Heat Vents: Smoke and heat vents shall be provided in areas or rooms containing hazardous materials exceeding the exempt amount of Table 3.2.5.

2.15.2 Location on Property

Buildings of Occupancy K shall comply with the requirements for location on property and fire resistive exterior walls and openings as specified in Sec 2.4.1.

2.15.3 Access and Exit Facilities and Emergency Escapes

Facilities for access and exit and emergency escape shall comply with the provisions set forth in Sec 1.6 Chapter 1 Part 3, Chapter 3 Part 4 and Appendix F.

2.15.4 Lighting, Ventilation and Sanitation

All buildings or part of a building classified as Occupancy K shall conform to the provisions of Sec 1.16 Chapter 1 Part 3, Chapters 1 and 3, Part 8.

2.15.5 Shaft and Enclosure

Elevator shafts, vent shafts and other vertical openings shall be enclosed conforming to the provisions of Tables 3.3.1 (a) and (b). Exit requirements shall comply with Chapter 3 Part 4.

2.15.6 Fire Detection, Alarm, Evacuation and Extinguishment System

All buildings shall conform to regulations set forth in Part 4 of this Code.

2.16 Requirements For Occupancy L–Utility Buildings

Buildings shall be classified as Occupancy L in accordance with Sec 2.1.16.

2.16.1 Construction, Height and Allowable Area

The buildings or parts thereof classified as Occupancy L shall be limited to the type of construction set forth in Table 3.2.4 and Sec 2.4.3, and shall comply with the provisions of Sec 1.8 Chapter 1 Part 3, and Sec 2.4.2 to meet the requirements and limitations of height and area.

2.16.2 Location on Property

Buildings of Occupancy L shall comply with the requirements for location on property and fire resistive exterior walls and openings as specified in Sec 2.4.1.

2.16.3 Access and Exit Facilities and Egress System

Facilities for access and exit and egress system shall comply with the provisions set forth in Sec 1.6 Chapter 1 Part 3 and Chapter 3 Part 4.

2.16.4 Lighting, Ventilation and Sanitation

All buildings or part of a building classified as Occupancy L shall conform to the provisions of Sec 1.16 Chapter 1 Part 3, Chapters 1 and 3, Part 8.

2.16.5 Shaft and Enclosure

Elevator shafts, vent shafts and other vertical openings shall be enclosed conforming to the provisions of Tables 3.3.1 (a) and (b). Exit requirements shall comply with Chapter 3 Part 4.

2.16.6 Fire Detector, Alarm, Evacuation and Extinguishment System

All buildings shall conform to regulations set forth in Part 4 of this Code.

2.16.7 Special Hazard

2.16.7.1 Since the nature of use of this occupancy involves hazard, special consideration for maintenance and operational safety must be ensured. Depending upon the degree of hazard involved, this occupancy type may have separate and isolated structure.

2.16.7.2 Chimneys and vents and ventilation ducts shall be of noncombustible materials.

All boilers, central heating plants, electrical rooms or hot water supply boiler shall be separated from the rest of the occupancies or uses by not less than 2 hour fire resistive construction.

The devices that generate a spark, flame or glow capable of igniting gasoline shall not be installed or used within 0.5 m of the floor.

Equipment or machinery that produces or emits combustible or explosive dust or fibers shall be provided with an approved dust collecting and exhaust system.

The equipment or system that is used to collect or process or convey combustible dust or fibers shall be installed with explosion venting or containment system.

2.17 Requirements For Occupancy M–Miscellaneous Buildings

Buildings shall be classified as Occupancy M in accordance with Sec 2.1.17.

2.17.1 General

The buildings or parts thereof classified as Occupancy M shall be limited to the type of construction set forth in Table 3.2.4 and shall comply with the requirements of Sections 1.8 and 2.4.2 to meet the requirements of height and area limitations.

Any building or portion thereof that exceeds the limitations provided in this Chapter shall be classified in the occupancy group other than M that it most nearly resembles.

2.17.2 Location on Property

The location on property for Occupancy M shall conform to Sec 2.4.1.

2.17.3 Access and Exit Facilities and Emergency Escapes

Access and exit facilities for Occupancy M shall comply with the specification set in Sec 1.6 Chapter 3, Part 4.

2.17.4 Lighting, Ventilation and Sanitation

All buildings or part of a building classified as Occupancy M shall conform to the provisions of Sec 1.16 Chapters 1 and 3, Part 8.

2.17.5 Shaft and Exit Enclosures

Elevator shafts, vent shafts and other vertical openings shall be enclosed conforming to the provisions of Tables 3.3.1 (a) and (b). Exit requirements shall comply with the requirements of Chapter 3, Part 4.

2.17.6 Fire Detection, Alarm, Evacuation and Extinguishment System

All buildings shall conform to regulations set forth in Part 4 of this Code.

Table 3.2.6: A—Z List of Occupancy Classification

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
	A	
Adhesives manufacture	Excluding manufacture of basic components	G or J depending on nature of materials involved
Advertising displays manufacture		G
Agricultural machinery manufacture	Including repairs	G

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Agriculture	Without nuisance or sales limitation	H
Agricultural	Small farm house, (limited to storage quantity)	F
	Large farm house, storage quantity unlimited	H or J
	Small grain processing unit, (limited to quantity)	G
	Large grain processing unit, quantity unlimited	G or J
Aircraft manufacture (including parts)		G or J depending on nature of materials and process involved
Airports		MIXED USE (depending on detail requirement)
Amusement parks, children's	(See children's amusement parks)	-
Amusement park activities		I
Animal	Animal hospitals	F
	Animal pound (for stray and lost animal)	H
	Animal crematorium	G
	Killing establishments, for retail sales	F
	Slaughtering, processing and packing	G
Antique stores		F
Apartments	(see residential)	
	in walkup buildings	A
	In high rises	A
	in housing complex	A
Apartment hotels		A5
Apparel	(See clothing)	

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Appliances	Electrical appliance Manufacturing	G
	Television, radio, phonograph or household appliance stores, (Limited as to floor areas)	F
	Television, radio, phonograph or household appliance stores, (Unlimited)	F
	Household appliance repair shops	F
Arenas, auditoriums, or stadiums	See Assembly (Limited as to capacity)	I
	See Assembly (Unlimited)	I
Art Galleries	Commercial (sales included)	F
	With exhibition open to public viewing for limited period (sales included)	I
Art goods manufacture, religious temple or church, excluding foundry operations		G
Art metal craft shops		F
Art needle work	Six occupants or less	Non-separated use to A1 and A2 Occupancy
	More than six occupants (see industrial)	G
Artist's supply stores		F
Asphalt or asphalt products	Manufacture	J
Assembly	Large assembly with fixed seats	I1
	Small assembly with fixed seats	I2
	Large assembly without fixed seats	I3
	Small assembly without fixed seats	I4
	For sport facilities	I5
Athletic equipment manufacture		G

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
Athletic goods stores		F
Auctions rooms, open to public		I
Auditoriums	See assembly	I
Automatic laundries		G
Automobiles	Dead Storage	H
	Driving Schools	E
	Glass or mirror shops	F
	Washing	K
	Manufacture, including parts, or engine rebuilding	J
	Rental establishments	K
	Repairs, body	K
	Repairs, without body repairs	K
	Sales open or enclosed	K
	Seat cover or convertible top establishments, selling or installation	F
	Showrooms, no repair services	K
	Supply stores, no repair services	F
	Tire sales establishments, limited to quantity	F
	Tire sales establishments, unlimited	J
	Wrecking establishments	G
Automotive service stations	Limited as to total area	K
	Unlimited...	K
Awnings	Custom shops	H
	Manufacture, with no limitation on production or on floor area	G

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
B		
Bakeries	Home-made, six or less occupants (baking included)	non-separated use to main occupancy
	Large scale, more than six occupants (baking included)	G
	Sales only	F
Banks,	Including drive-in banks	E
Banquet halls		I
Bar, alcoholic		I
Barber shops		F
Barns		H
Barracks	(See residential)	A4
Baths, steam		I
Beaches, commercial		Not applicable
Beauty parlors		F
Beverages	Bottling works	G
	Manufacture, Alcoholic	J
	Non-alcoholic	G
Bicycle	Manufacture	G
	Rental or repair shops	F
	Sales	F
Billiard parlors		I
Blacksmith shops	small scale (limited to six occupants), repair or making	F
	Unlimited	G
Blueprinting establishments	drawing printing	G
Boarding houses	(See residential)	A
Borstals		C
Boatels		A

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Boats or ships	Bailer works at port or dock	J
	Breaking	J
	Building or repair, for boats less than 200 ft. in length	J
	Building or repair, for boats 200 ft. or more in length	J
	Docks, for small pleasure boats	Not applicable
	Fuel sales, open or enclosed	
	Un- restricted as to location	F
	Restricted as to location	J
	Rentals opened or enclosed	F
	Sales opened or enclosed	F
	Showrooms, with no repair services..	F
	Storage, repair, or painting, including the incidental sales of boats, boat parts, or accessories, with restrictions on boat size and setbacks	G
Bone distillation		G or J depending on process or material used
Botanical garden structures		M
Book	Binding (see printing)	
	Hand binding or tooling	G
	Store	F
Bottling works, for all beverages		G
Bowling alleys	Limited as to number of lanes	I
	Unlimited	I
Breweries		G
Brick manufacture		J
Brush or broom manufacture		G

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Building materials sales	open or enclosed, limited as to lot area	F
	Yards, for sales, storage, or handling, open or enclosed, unlimited as to lot area except in the case of lumber yards	F
Bungalow	(See residential)	A
Business	Offices	B1
	Research and testing laboratories	B2
	Essential services	B3
Bus stations	With less than 10 berths	K (bus area) and I (passenger area)
	With 10 or more berths	MIXED (as per detail requirement including K and I)
Bus stops	see Bus stations	
Business machines	Manufacture	G
	Small, repair shops	F
	Stores, sales, or rentals..	F
Business schools or colleges		B
Buying house (garments)	storage restricted to sample	E
C		
Café	Six persons or less	Non-separated use to main Occupancy
	More than six persons (see mercantile)	F
Cafeteria	With commercial kitchen	MIXED (G and I)
	Without commercial kitchen	I
Camera and photo equipment	Manufacture	G
Camps, overnight or outdoor day		MIXED (A, I and other depending on the nature of use)

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Candy stores		F
Canneries, including food products		J
Canteen	With or without cooking facility	I
Canvas or canvas products manufacture		G2
Cargo terminal	containing low fire-risk materials	H
	containing moderate fire-risk materials	H
	containing high fire-risk materials	J
Carnivals, temporary		I5
Carpentry shops		G
Carpet	Cleaning establishments...	J or G depending on the nature of materials involved
	Manufacture	G
	Carpet, rug, linoleum or other floor covering stores Unlimited	F
Carport	Roofed wall less shelter for car	K or H depending on the nature of use
	Automated mechanical parking	K or H depending on the nature of use
Catering establishments	Commercial kitchen	G
	Office	E
	Storage, open or enclosed	H
	Storage for temporary structure's fabrication material	J
Cattle shed, stables		H
Cement manufacture		G2
Cemeteries		H
Ceramic products	Manufacture	G or J based on nature of material used
	Display and sales	F

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Chamber, doctors' or dentists', (outpatient only)	50 or less occupants above 50 occupants	E D
Charcoal manufacture		G
Chemicals	Compounding or packaging Manufacture	G or J depending on nature of materials involved G or J depending on nature of materials involved
Child care home		C
Child care institution		C
Children's amusement parks	Small Medium size Large size Unlimited as to size	I I I I
Churches, with fixed pews	(See Assembly with fixed seats)	I
Cigar stores		F
Cinema hall	(See Assembly with fixed seats)	I
Cineplex	(See Assembly with fixed seats)	I
Circuses, temporary	(See Assembly)	I
Class room	School, college or university	B
Clay manufacture		G
Clay pits		Not applicable
Cleaning or cleaning and dyeing establishments	(See dry cleaning)....	
Clinics	With inpatient Only outpatient, limited to quantity (see chambers, doctors' or dentists') Only outpatient, unlimited With diagnostic facilities (see diagnostic facilities) Government community clinic	D D E

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
Coaching centre	(See educational facilities)	B
Cold storage		H
Composite textile mill		G or J depending on nature of material and process used
Cottage industries	Small, fifty or less workers (see industrial facilities)	G1
	Large, more than fifty workers… (see industrial facilities)	G1 or G2 depending on the nature of material and process used
Clock	Manufacture	G
	Stores or repair shops	F
Clothing	Accessory stores	F
	Custom manufacture or altering for retail	F
	Manufacture	G or J depending on nature of the material involved
	Rental establishments	F
	Store, Limited as to floor area	F
	Store, Unlimited	F
Clubs Non-commercial (members only)	Including accommodation	MIXED (A and I)
	Night-club	I
	All types except those with outdoor swimming pools	
Clubs, for public use	Excluding accommodation	I
	Including accommodation	MIXED (I and A or other occupancies depending upon nature of use)
Clubs, Sporting		MIXED (I and A or other occupancies depending upon nature of use)

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Coal	Products manufacture	J
	Sales, open or enclosed, Limited as to plot area	J
	Unlimited (see coal storage)	J
	Storage, open or enclosed	J
Coin stores		F
Condensed and powdered milk	Manufacture	J
Coke products	Manufacture	J
Colleges or universities	See educational facilities	B
Colony, government or non-government		MIXED (A and other occupancies depending on use)
Commercial building	(see business and/or mercantile)	
Commercial parking garages or plots	(See garages)	K
Community centers	With commercial kitchen	MIXED (G and I)
	Without commercial kitchen	I
Concrete batching		G
Concrete products manufacture		G
Construction machinery	Manufacture, including repairs	G
Container terminal		H or J (According to the hazard classification regulation of the port authority)
Contractors' establishments	Electrical, glazing, heating, painting, paper hanging, plumbing, roofing, or ventilating	F
	Contractors' yards	Not applicable
Convalescent homes	(See nursing homes)	

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
Convents		MIXED (A, B and I)
Cork products	Manufacture	G
Cosmetics or toiletries	Manufacture..	J
Costume rental establishments		F
Cottage, tourist	(See residential)	A5
Cotton ginning or cotton wadding or liner manufacture		J
Court houses		I
Crate manufacture		G or J depending on the material and process involved
Crematoriums	Animals. Human.	J MIXED (J and I)
Cultural center		Mixed (depending on detail requirement)
D		
Dance halls	Public	I
Dance School		A
Dance studios	(see studios)	
Day camps, outdoor		I
Day care Centre	With six or less children More than six children	Non-separated use to Residential Occupancy C
Decorator's establishment	Office Storage, separated	E H or J depending upon the material involved
Defense Buildings, for critical national defense capabilities		Not Applicable
Delicatessen stores	(See food stores)	F

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Dental	Instruments manufacture Laboratories (See laboratories, medical or dental)	G
Department stores	not exceeding 300 m² more than 300 m²	F1 F2
Diagnostic facilities, medical	Outpatients only	D
Diaper supply establishments		H
Disinfectants manufacture		G
Dispensaries	Attached to hospital See drug store	L F
Dormitories	Universities or colleges (above 12 grade)	A
	Schools (12 grade or below)	C
Drafting instruments	Manufacture	G
Dressmaking shops, custom		F
Drinking places, non-alcoholic	(See cafe)	
Drive-in theaters		I
Drug stores		F
Dry cleaning or clothes pressing establishments	Limited as to floor area, solvents and machine capacity	G or J depending on the process and quantity of material used
Dry cleaning or cleaning and dyeing establishments	Without restrictions	G or J depending on the process and quantity of material used
Dry Cleaning, using other than flammable liquids in cleaning or dyeing operations		G

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
Dry goods stores	Limited as to floor area	F
	Unlimited	F
Dumps		Not applicable
Dyeing facilities/ industries		J
	E	
Eating or drinking places	With restrictions on entertainment (see Assembly)	I
	Without restrictions on entertainment or dancing but limited to location in hotels (see Assembly)	I
	Without restrictions (See assembly)	I
Eco park structures		MIXED (depending upon the nature of use)
Educational facilities	Up to higher secondary level	B1
	Training and above-higher-secondary education	B2
	Pre-school facilities	B3
Electric	Power or steam generating plants	G
	Substations, Public transit or railroad	G
	Substations, as part of public distribution system	G
	Substations, low to medium voltage step down, at consumers' end	L
Electrical Appliance	Manufacture	G or J depending upon the process or material to be used
	Stores (including television, radio, phonograph or household appliances)	F
	Contractors (See contractors' establishments)	
	Equipment assembly, not including electrical machinery	G
	Supplies, manufacturing	G

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Electronics manufacturing		J
Electrolysis works		J
Electrotyping or stereotyping	Limited to quantity	F
	Unlimited (see printing)	G
Embassy or High-commission or Consulate		MIXED (depending on detail requirement)
Engine	including rebuilding or reconditioning	J
Engraving or photo-engraving	Limited to quantity	F
	Unlimited (see printing)	G
Excelsior manufacture		J
Exhibition hall	See assembly	I
Exterminators	See pest control	F
F		
Fabric stores		F
Factory		G or J (depending on process and material involved)
Fairs, temporary		MIXED (I and F)
Feathers	Bulk processing, washing, curing, or dyeing	J
	Products manufacture, except washing , curing or dyeing	J
Felt	Bulk processing, washing, curing, or dyeing	G
	Products manufacture, except washing, curing or dyeing	G
Fertilizer manufacture		J
Field hospital, temporary	With provision for ambulance access (to parks and play grounds)	E
Filling stations	(See refueling station)	B

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
Film, photographic	Manufacture	G
Fire Stations		E
Fish products, packing or processing		G
Fishing tackle or equipment rental or sales		F
Flats	(see residential)	A
	In walkup buildings	A
	In high rises	A
	in housing complex	MIXED (A and other occupancies)
Florist shops		F
Food	Products processing, except meat slaughtering or preparation of fish for packing	G
	Stores, including supermarkets, grocery stores, meat markets, or delicatessen stores	F
Foundries	Ferrous or non-ferrous	G or J (depending on process and material involved)
Fraternity houses	(See colleges or universities)	
Freight depot	See storage and hazardous buildings	H and/or J
Frozen food lockers		J
Fuel briquettes manufacture		G
Fuel sales, open or enclosed	Limited up to exempted quantity	F
	Unlimited, See coal storage or petroleum storage	J
Funeral establishments		I
Fungicides manufacture		G

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Fur	Goods manufacture, not including tanning or dyeing	G
	Tanning, curing, finishing, or dyeing	J
Furniture	Custom shop, floor area of 100 m^2 or less	F
	Custom shop, floor area over 100 m^2	G
	Manufacture	J or G depending upon nature of materials involved
	Store, Limited as to floor area	F
	Store, Unlimited	F
Furriers shops, custom		F
Freight depot		H or J depending on the nature of material involved
G		
Garages	Parking garage	K1
	Private garage	K2
	Repair garage and show-rooms	K3
Garbage incineration or reduction		G
Garden shed		M
Garden supply stores		F
Gardens, truck	(See agriculture)	
Garments industries		G
Gas, fuel	Manufacture	J
	Distribution regulatory system (DRS)	G
Gas manufacture for	Medical purpose	J
	Hot-works (welding)	J
Gasoline service stations	(See refueling stations)	
Gelatin manufacture		G

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
Generating plants, electric or steam		G
Gift stores		F
Glass	Cutting shops	F
	Manufacture	G
	Products manufacture from previously manufactured glass	G
Glazing contractor's establishment	(See contractors' establishments)	F
Glue manufacture		G
Godown	See storage buildings	
Golf	Courses	Not applicable
	Courses, miniature	I
	Driving ranges	I
Grain	Milling or processing	J
	Storage	J
Graphite or graphite products	Manufacture	G
Gravel pits		Not applicable
Grocery stores		F
Group homes	Segregation of occupants on the basis of age group and disabilities (See institutional)	C
Gypsum production industry		J
Gymnasiums	Less than 300 occupants	I
	300 or more occupants	I
	Commercial without spectator gallery (max 50 occupants)	

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
H		
Hair	Bulk processing, washing, curing, or dyeing	G
	Products manufacture (except washing, curing, or dyeing)	G
	Products manufacture, custom	G
Hall, for incidental show (picture, drama, theatre)	(See assembly)	I
Hardware	Manufacture	G
	Stores	F
	Bodies manufacture	G
	Repair shops	F
Hazardous buildings	Explosion-hazard building	J1
	Chemical-hazard building	J2
	Biological-hazard building	J3
	Nuclear-hazard building	J4
Health centers	With inpatient	D
	Without inpatient (not more than 50 occupants)	E
	Government operated health centers	E or D (depending upon the facilities)
Healthcare facilities	Normal medical facilities	D1
	Emergency medical facilities	D2
Health club		I
Heating contractor's establishment	(See contractors' establishments)	
Heat, ventilation and air-conditioning equipment showrooms	Without repair facilities	F
Heliports		G
Hemp products manufacture		G

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
High Commission	See embassy	
Home for care	of the old and infirm (see institution)	
	of mentally disabled (see institution)	
Home office	Not more than 6 occupants	Non-separated use of Occupancy A
Hosiery manufacture		G
Hospital, except animal hospital	As part of disaster preparedness program	D
	Casualty unit	D
	Emergency unit	D
	Non-profit or voluntary, and related facilities	D
	Proprietary and related facilities	D
Hostels	For adults	A
	For children	C
Hotels	Transient	A
	Apartment hotel	A
	Starred hotel	MIXED
Household	Appliance repair shops	F
	Appliance stores (See appliances television, radio, phonograph, or household appliance stores)	F
Housing, complex multi-storied		MIXED (see appendix)
Housing, cluster		MIXED (see appendix)
Housing, low-income		MIXED (see appendix)
Housing, minimum standard		MIXED (see appendix)
Housing, rehabilitation		MIXED (see appendix)

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
	I	
Ice cream stores		F
Ice	Manufacture, dry or natural	G or J (depending on the process or material used)
	Sales, open or enclosed Limited as to lot area	F
	Unlimited	F
Incineration or reduction of garbage, offal, or dead animals		G
Indoor facility, for amusement park		I
Industrial buildings	Low-hazard Industries	G1
	Moderate-hazard Industries	G2
Infirmaries		C
Ink or inked ribbon manufacture		G or J depending on nature of materials involved
Inns	See residential	A
Insecticides manufacture		G or J depending on nature of materials involved
Institution	For care of children	C1
	Custodial, for physically capable adults	C2
	Custodial, for physically incapable adults	C3
	Penal or mental, for children	C4
	Penal or mental, for adults	C5
Institutions, philanthropic or non-profit	With sleeping accommodations	A
	Without sleeping accommodations	

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Interior decorating establishments	Limited as to floor area for processing, servicing, or repairs	F
	Unlimited, see furniture, textiles or upholstering	F
Irradiation plant		J
J		
Jail	see prisons	
Jewelry	Manufacture	G
	Costume	G
	From precious metals	G
	Shops	F
Junk Yards		Not applicable
Jute products manufacture		G or J (depending on quantity or process)
Juvenile correctional center	For children (see assembly)	
K		
Kennels		H
Kindergarten	See educational facilities	B
Knitwear industries		G2
L		
Laboratories	Medical or dental, for research or testing, with limitations on objectionable effects	E
	Research, experimental, or testing, unlimited	(G or J) and H depending on process or material used in compliance with safety standards
	Radiological laboratory, see radiological facilities	
	Pathological laboratory	G (in compliance with safety standards)

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
	Microbiological laboratory, for diagnostic facility	G or J depending on process or material used in compliance with safety standards
	Microbiological laboratory, for research	G or J depending on process or material used in compliance with safety standards
	Microbiological laboratory, for academic facility	G or J depending on process or material used in compliance with safety standards
Lampblack manufacture		G
Laundries, with no limitations on type of operation		G
Laundry establishments, hand or automatic self-service		G
Lavatory, public	see public toilet	
Leather	Tanning, curing, finishing or dyeing	J
	Goods stores	F
	Products manufacture	G
Libraries	Reading area (see assembly)	I
	Stack area (see storage)	H
	Reading and stack area combined	MIXED (I and H)
Lillah boarding	For children (see institutional)	C
	For adults (see residential)	A
Linen supply establishments		
Linoleum	Manufacture	J
	Stores (See carpet stores)	

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Liquor stores, package		F
Livestock	Storage, more than six castles	H
	Slaughtering or preparation for packing	G
Loan offices		E
Locksmith shops		F
Lodging	See residential	A
Luggage	Manufacture	G
	Stores	F
Lumber	Processing or woodwork, bulk	G
	Sales, Limited as to lot area	G
	Sales, Unlimited	G
	Yard, Limited as to lot area	G
	Yard, Unlimited	G
	M	
Machine	Shops including tool, die or pattern making	G
	Tools manufacture	G
Machinery	Manufacture or repair, Heavy	G or J depending on material and process
	Miscellaneous or electrical equipment	G or J depending on material and process
	Rental or sales establishments	F
	Repair shops	F
Machines, business	(See business machines)	
Madrasa	(See institution)	
Manure storage		H
Markets	Retail, including meat (See mercantile)	F
	Wholesale, produce or meat (See mercantile)	F
Masseurs		F

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
Matches manufacture		J
Mattress manufacture, rebuilding or renovating		J
Meat	Markets, Retail (See food stores)	F
	Markets, Wholesale	F
	Slaughtering or preparation for packing	G
Medical	Appliances, Custom manufacture	G
	Appliances, Manufacture	G
	Stores	F
	Instruments, manufacture	G
	Laboratories (See laboratories, medical)	
	Offices or group medical centers, Limited as to location within building	E
	Offices or group medical centers, Unlimited	E
Meeting halls	See Assembly	I
Mess houses	(See residential)	
Metal Fabrication industry		J
Metal Assembly industry		J
Metals manufacture	Alloys or foil, miscellaneous	G
	Casting or foundry products, heavy	G
	Finishing, plating, grinding, sharpening, polishing, cleaning, rust proofing, heat treatment, or similar processes	G
	Ores reduction or refining	G
	Products treatment or processing	G
	Reduction, refining, smelting, or alloying	G
	Stamping or extrusion	G
	Treatment or processing	G

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
Mental institution	Without detention facilities	D
Mental hospitals	(See institution)	C
Mercantile	Small shops and markets	F1
	Large shops and markets	F2
	Refueling station	F3
Mill	(See industrial and/ or hazardous buildings)	G or J (depending on material or process)
Mill works, and woodworking, wood distillation and particle boards manufacturing		J
Millinery shops		F
Mining machinery manufacture	Including repairs	G
Mirror silvering shops		G
Miscellaneous buildings	Special structures	M1
	Fences, tanks and towers	M2
Monasteries		MIXED
Monument	Sales establishments, with incidental processing to order	F
	Works, with no limitations on processing	G
Mosque	(See assembly)	I
Motels	(See residential)	A
Motion picture production and filming facilities		MIXED (G and other Occupancies as required)
Motorcycles	Manufacture	G
	Repairs, body	G
	Repairs, except body repairs	G
	Sales open or enclosed	F
	Showrooms, with no repair services (See garage)	K
Motor freight stations	See truck terminals	

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Motor vehicles	Dead storage	H
	Moving or storage offices, Limited as to storage	K
	Unlimited	K
Movie theatre	See assembly	I
Museums	See assembly	I
Music stores		F
Music studios	See studios	
Musical instruments	Manufacture, Excluding pianos and organs	G1
	Including pianos and organs…	G2
	Repair shops…	G1
N		
Newspaper publishing		MIXED (G and E)
	Printing	G
	Office	E
Newsstands, open or closed		F
Novelty products manufacture		G
Novitiates…	See institution	A
Nuclear medicine facilities	see radiological facilities	
Nuclear plant		J
Nurseries	See agriculture	
Nursing homes	Philanthropic or non-profit	C or D depending on the type of occupants and nature of use
	Private	C or D depending on the type of occupants and nature of use
Nursery schools	See pre-school	B

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
O		
Oakum products manufacture		G
Office equipment or machinery repair shops		F
Office or business machine stores	sales or rental	F
Offices	General	E
	Business, professional or Governmental (see business occupancy)	E
	Dental, medical, or osteopathic (See medical offices)	E
	Wholesale, with storage restricted to samples (see business occupancy)	E
Offices, small	Architect's/engineer's/ consultant's (Limited to six occupants)	Non-separated use of Occupancy A
	Architect's/engineer's/consultant's (more than six occupants)	E
Oil cloth manufacture		J
Oil sales, open and enclosed	Limited as to lot area	F
	Unlimited (See petroleum or petroleum products storage)	J
Old home	See institution	C
Optical	Equipment manufacture	G
	Goods manufacture	G
Orphanage	See institution	C
Optician or optometrist establishments		F
Orthopedic	Appliances, Custom manufacture	G
	Manufacture	G
	Stores	F
	Instruments, manufacture	G
Osteopathic offices	(See medical offices)	

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
P		
Packing or crating establishments		G2
Packing materials manufacture		G2
Pagoda	See Prayer hall	
Paint	Manufacture	J
	Stores, limited to quantity	F
	Stores, unlimited	H
Painting contractors	(See contractors' establishments)	
Paper	Mills (See wood pulp or fiber)	G
	Products manufacture	G
	Stock companies	H
Paper-hanging contractors	(See contractors' establishments)	
Parish houses		A
Parks, public or private	With provision for emergency vehicle access as part of disaster preparedness program	Not applicable
Park structures		M
Parking garages, public	See garage, parking	K
Parking lots, public	See garage, parking	K
Passenger stations and terminals	Small, passenger station	MIXED (depending on nature of use)
	Large, passenger station or terminal	MIXED (depending on nature of use)
	Passenger and freight terminal	MIXED (depending on nature of use)
Peat storage		H
Perfumed or perfumed soaps	compounding only, not including soap manufacture	J

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
Pest control	Exempted quantity only	F
Pet shops		F
Petrol pump	See refueling station	F
Petroleum or petroleum products	Refining	J
	Storage and handling	J
Pharmaceutical products manufacture		G or J depending on nature of materials used
Philanthropic, religious or non-profit activities		MIXED (depending on nature of use)
Phonograph	Repair shops	F
	Stores (See appliances)	F
Photocopying and book binding	Binding limited in quantity	F
Photographic	Developing or printing establishment, Retail	F
	Developing or printing establishment, Wholesale, Limited as to floor area	H
	Developing or printing establishment, Wholesale, Unlimited	H
	Equipment, Manufacture (film)	J
	Equipment, Manufacture (except film)	G
	Stores	F
	Studios	F
	Supply stores (limited to exempted quantity)	F
Photostatting establishments		F
Physical culture establishments		I
Picture framing stores		F
Plants, Industrial		G
Plants, Refrigeration		G

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Plastics	Products, manufacture	J
	Raw, manufacture	J
Plate making	(See printing)	
Playgrounds	With provision for emergency vehicle access as part of disaster preparedness program	I
Plots, parking	(See parking lots, public)	
Plumbing	Contractors' establishments	F
	Equipment manufacturer (See tools or hardware manufacturing)	
	Showrooms, without repair facilities	F
Police Stations		E
Pool halls		I
Porcelain products manufacture		G
Post offices		E
Poultry	Storage (live)	H
	Killing establishments, for retail sales on the same zoning lot only	G
	Packing or slaughtering	G
Power plant		G
Power stations	As part of national grid power distribution system	E
	At consumer's end	L
Prayer hall	See assembly	I
Precision instruments manufacture	Optical equipment, clocks, or similar products	G
	Medical, dental, or drafting instruments, optical goods, or similar products	G
Pre-school facilities	See educational	

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Press club, for journalist		I
Press, printing	See printing	
Primary schools	See educational	
Printing	Custom	G
	Limited as to floor area	G
	Unlimited	G
Printing, publishing, dyeing and printing industries		J
Prisons	See jail	C
Produce or meat markets, wholesale		F
Psychiatric sanatoria	With detention facilities (see institution)	
Public auction rooms		MIXED (F and/or I)
Public transit yards		Not applicable
Publishing	With printing	G
	Without printing	E
Pumping stations	Water or sewage (for city supply system)	G
	Dedicated to consumer	U
Q		
Quarter, Staff	Government or non-government	A or Mixed (See appendix)
R		
Racetracks		I
Radio	Appliance repair shops	F
	Stores	F
	Studios, with less than six occupants	Non-separated use to main Occupancy
	Studios, without transmission tower	E
	Studios, with transmission tower (see radio station)	
	Towers, non-accessory	M

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Radio station		Mixed (depending on the type of use)
Radiological facilities, medical	In compliance with the standard of atomic energy commission	D
Radioactive waste disposal services		J
Railroad	Equipment manufacture, including railroad cars or locomotives	G or J depending on the material and hot-work used
	Passenger stations	I
	Right-of-way	Not applicable
	Substations	
	Small or medium size	G
	Large	G
	Railroads, including rights-of-way, freight terminals, yards or appurtenances, or facilities or services used or required in railroad operations, but not including passenger stations	Not applicable
Rail station		Mixed (depending on the type of use)
Record stores		F
Recreation centers, non-commercial		I
Recreation piers	See assembly	I
Recreational vehicles manufacturing		J
Rectories		A
Reducing salons		I
Reformatories	See institutional facilities	
Refreshments stand, drive-in		I
Refrigerating plants		G

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Refueling station	Petroleum product storage within exempted quantity	E
Refuse incinerators		J
Religious or church art goods manufacture		G
Research establishment	dealing with non-hazard or low hazard materials only	E
Residences	Single-family detached	A
	One-family semi-detached or two-family detached or semi-detached	A
	Boarding or rooming houses	A
	Rest homes (See nursing homes)	
Residential	Single family dwelling	A1
	Two family dwelling	A2
	Flats or apartments	A3
	Mess, boarding house, dormitories and hostels	A4
	Hotels and lodging houses	A5
Rest Houses		
Restaurant	Dining area	I
	Performing area, limited	I
	Kitchen and storage	L
Reviewing stand		I
Riding academies, open or enclosed		E and H
Roofing contractors' establishments		F
Rooming houses	See residential	A
Rubber	Processing or manufacture, natural or synthetic	J
	Products manufacture (excluding all natural or synthetic rubber processing)	J
Rug stores	(See carpet stores)	

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
S		
Sail-making establishments		F
Salvage storage		H
Sand pits		Not applicable
Saloon, hair dressing		F
Sanatoriums	With detention facilities (see institution)	C
	Without detention facilities	D
Sawmills		G
Scenery construction		G
School (see educational)	Dormitories, for children	C
	Nursery, kindergarten, elementary or secondary schools	B
	Trade or other schools for adults, limited as to objectionable effects	B
	Trade schools for adults, unlimited	B
	For physically challenged, without accommodation	B
	For mentally challenged, without accommodation	B
Scrap metal, paper and rag storage		H
Secondary school	See educational	B
Seed stores		F
Seminar halls	For 50 or more occupants, See assembly	I
Seminaries		B
Settlement houses	(see housing)	MIXED (A and other Occupancy depending on the nature of use
Sewage	Disposal plants	G
	Pumping stations	G

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Sewing machine stores, selling household machines only		F
Ship chandlers, candle shops		F
Ship or boat building or repair yards	For ships 200 ft. in length or over	G
Shipping, waterfront		Not applicable
Shoes	Manufacture	G or J depending on the process and material involved
	Repair shops	F
	Stores	F
Shops	see definition	F or G (depending on the process and material involved)
Shop-house		mixed occupancy (A and F) or (A, F and G)
Sign painting shops	Limited as to floor area	G
	Unlimited	G
Silk processing and spinning		J
Silo, for storage of grain		H
Silver plating shops, custom		G
Silverware manufacture, plate or sterling		G
Sisal products manufacture		J
Skating rinks, roller	Indoor	I
	Outdoor	I
Slag piles		Not applicable

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
Slaughtering of animals or poultry		G
Soap or detergents	Manufacture, including fat rendering Packaging only	J G
Soldering shops		G
Solvent extracting		J
Sorority houses	(See hostel)	A
Sports centre		I
Sporting equipment manufacture.		G
Sporting goods stores		F
Stable for horses		H
Stadiums	Indoor or outdoor, with access for emergency vehicle as part of disaster preparedness program	I
Staff quarter	see quarter, staff	
Stamp stores		F
Station	Rail, bus, air and water way	MIXED (I and other Occupancy depending on the nature of use
Stationary stores		F
Statuary, mannequins, figurines, religious or church art goods manufacture, excluding foundry operations		G
Steel products	Miscellaneous fabrication or assembly (without hot-work) Structural products manufacture	G J
Stock yards or slaughtering of animals or poultry		G

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Stone processing or stone products		G
Storage buildings	Low-fire-risk storage Moderate-fire-risk storage	H1 H2
Storage facilities	Wholesale (see storage buildings) Offices, limited to quantity For cotton/jute/ paper/textile	H Non-separated use J
Stores	See definition	F
Students' halls of residence	For children For adults	C A
Studios	Music, dancing, or theatrical Radio (see radio studio) Television, with spectator Television, without spectator	I MIXED (I, E or G) MIXED (depending on nature of material and process involved)
Sugar	Production and Refining	J
Super market	See mercantile	F
Swimming pools	Commercial Non-Commercial (See clubs)	I I
T		
Table tennis halls	See assembly	I
Tailor shops, custom		F
Tanning (See leather or fur)		J
Tapestries manufacture		G
Tar products manufacture		G
Taxidermist shops		F
Telegraph offices		E

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Telephone exchanges or other communications equipment structures		E
Television	Repair shops	F
	Stores (See appliances)	F
	Studios (see television studios)	
	Towers, non-accessory	M
Television station	See business	MIXED (E3 with other Occupancies according to detail requirement)
Temple	See prayer hall	
Tennis courts, indoor		I
Terminal facilities at river crossings for access to electric, gas, or steam lines		G
Test laboratory	involving low hazard material	E
Textiles	Bleaching (see industrial)	G
	Products manufacture (see industrial)	G
	Spinning, weaving, manufacturing, dyeing, printing, knit goods, yarn, thread, or cordage (see industrial)	G
Textile industries and jute mills	including canvas, cotton cloth, bagging burlap, carpet and rags (see industrial)	J
Theater	See assembly	I
Theaters, drive-in	(See studios)	
Theatrical studios	without spectator	G
Tile	Manufacture	G
Tire sales establishments	Including installation services, Limited to quantity	F
	Including installation services, unlimited quantity	J

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Tobacco	Curing or manufacture, or tobacco products manufacture	J
	Stores (retail)	F
Toilet, public		L
Toiletries manufacture		G or J depending on the material and process involved
Tool or hardware manufacture	See industries	G
Topsoil storage	See storage	H
Tourist cabins	See residential	A
Towel supply establishments		F
Toys	Manufacture	G
	Stores	F
Trade or other schools for adults	Limited as to objectionable effects (see educational)	B
	Unlimited (see educational)	B
Trade expositions	Limited as to rated capacity	I
	Unlimited	I
Trailer, truck, bus	Manufacture, including parts	G or J depending on the material and process involved
	Repairs, body	G or J depending on the material and process involved
	Sales open or enclosed	F
	Showrooms, with no repair services	F
Training center	lecture based, limited to quantity (see educational facilities)	E1 or B2
	vocational or demonstrative (see educational facilities)	B2
Transit substations	Small or medium size	G
	Large	G

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Transport terminal	Small or medium size	MIXED depending on nature of use
	Large	MIXED depending on nature of use
Travel agency	(see business)	E
Travel bureaus	(see business)	E
Truck	Manufacture (including parts) or engine rebuilding	G or J depending on the material and process involved
	Repairs, body	G
	Repairs, except body repairs	G
	Sales open or enclosed	F
	Showrooms, with no repair services	F
	Trucking terminals or motor freight stations, Limited as to lot area	K1
	Trucking terminals or motor freight stations, Unlimited	K1
Tutorial homes	More than six occupants (see educational)	B
Turpentine manufacture		J
Typewriter stores		F
Typewriter or other small business machine repair shops		F
Typography	(See printing)	
U		
Umbrellas	Manufacture	G
	Repair shops	F
University	See educational facilities	B2
Upholstery	Manufacturing	J
	Bulk, including shops not dealing directly with consumers	J
	Shops dealing directly with consumers, retail	F
Utility		L

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
	V	
Variety stores	Limited as to floor area	F
	Unlimited	F
Varnish manufacture		J
Vehicles	Dead storage of motor	H
	Manufacture, children's	G
	Storage, commercial or public utility, open or enclosed	K
Venetian blind, window shade, or awning	Custom shops, limited as to floor area	F
	Manufacture, with no limitation on production or on floor area	J or G depending upon nature of materials involved
Ventilating contractors	(See contractors' establishments)	F
Ventilating equipment showrooms	Without repair facilities	F
Video games shop		F
Vihara, Buddhist	with occasional or regular assembly	mixed use
	W	
Wallpaper stores	Limited to quantity	H
Warehouses		H or J (depending on the nature of material stored)
Watch or clock stores or repair shops		F
Watch making		G
Waterfront shipping		Not applicable
Water pumping stations	At distributor's end	G
	At consumer's end	L
Water tank tower		M
Wax products manufacture		G

Use or Occupancy	Brief Description	Occupancy Class/Sub-class
Weaving, hand	Up to six hand-weaving machines	Non- separated use to main Occupancy
	More than six hand-weaving machines	G
Wedding chapels	See assembly	I
Welding shops	Arc welding only	G
	Gas welding within exempted quantity	G or J depending upon the quantity of material and process
Welfare centers		
Wholesale establishments		H or J depending upon the nature of material
Wholesale offices or showrooms, with storage restricted to samples		E
Window manufacture		G
Window shades	Custom shops, limited as to floor area	F
	Manufacture, without limitation on production or on floor area	G
Wood	Bulk processing or woodworking	G
	Distillation	G
	Products manufacture	G
	Pulp or fibre, reduction or processing, including paper mill operations	G
	Sales, open or enclosed, Limited as to lot area	F
	Unlimited (See lumber yards)	F
	Woodworking shops, custom	J or G depending upon nature of materials involved
Wool scouring or pulling		G
Workshops	With hot-works	J
	Without hot-works	G

Use or Occupancy	**Brief Description**	**Occupancy Class/Sub-class**
	X	
X-ray facilities	See radiological facilities	
	Y	
Yard		Not applicable
Yard, ship	See ship or boat building or repair yards	
Yarn, manufacturing		G or J depending on the quantity (see Table 3.2.5)
	Z	
Zoo structures		M

** The occupancy classification for any project, not included in this list, shall be determined through the following process:

i. The functional requirements of the unidentified occupancy shall be compared with the Occupancy use type, classification, sub-classification categories and descriptions to match with the given occupancies to find the most similar Occupancy,

ii. If process (i) fails to determine the Occupancy, the project will be referred to the Board of Appeal constituted as per directives of Part 2 Chapter 2. The Board of Appeal shall determine the Occupancy, and

iii. The decision of Board of Appeal shall be considered as an explanatory material of this Code and shall be added as addendum to this Code. For any future projects of similar nature this addendum will suffice and need not be referred to the Board of Appeal again.

PART III

Chapter 3

Classification of Building Construction Types Based on Fire Resistance

3.1 General

3.1.1 Classification by Type of Construction

For the purpose of this Code, every room or space of a building or a building itself hereafter altered or erected shall be classified in one specific type of construction as grouped as follows:

GROUP I: Noncombustible, subdivision

Type I-A: 4 hour fire protected

Type I-B: 3 hour fire protected

Type I-C: 2 hour fire protected

Type I-D: 1 hour fire protected

Type I-E: Unprotected

GROUP II: Combustible subdivision

Type II-A: Heavy timber

Type II-B: Protected wood joist

Type II-C: Unprotected wood joist

Type II-D: Protected wood frame

Type II-E: Unprotected wood frame

The fire resistance ratings of various types of construction for structural and non-structural members are specified in Tables 3.3.1 (a) and (b). For hazardous Occupancies involving an exceptionally high degree of fire risk or an exceptionally high concentration of combustible or flammable content, the Authority may increase the requirement of Table 3.3.1 (a).

Buildings having a height of more than 33 m shall be constructed with non-combustible materials.

The fire resistance ratings of various building components shall conform to ASTM standards.

No building or portion thereof shall be designated a given construction type unless it fully conforms to the minimum requirements for that Construction type.

When a type of construction is utilized which is superior than the type of construction required by this Code, there shall be no requirement to upgrade the rest of the construction to comply to that higher type of construction and the designated construction type shall be that of the lesser classification, unless all of the requirements for the higher classification are met.

3.1.2 Group I: Non-Combustible Construction

Buildings or portion thereof in Non-combustible Construction Group I are those in which the walls, exit-ways, shafts, structural members, floors, and roofs are constructed of non-combustible materials and assemblies having fire-resistance ratings specified in Table 3.3.1 (a). The Non-combustible group consists of Construction Type I-A, I-B, I-C, I-D and I-E.

3.1.2.1 Construction Type I-A

This construction type includes buildings in which the bearing walls and other major structural elements are generally of four-hour-fire-resistance rating.

3.1.2.2 Construction Type I-B

This construction type includes buildings in which the bearing walls and other major structural elements are generally of three-hour-fire-resistance rating.

3.1.2.3 Construction Type I-C

This construction type includes buildings in which the bearing walls and other major structural elements are generally of two-hour-fire-resistance rating.

3.1.2.4 Construction Type I-D

This construction type includes buildings in which the bearing walls and other major structural elements are generally of one-hour-fire-resistance rating.

3.1.2.5 Construction Type I-E

This construction type includes buildings in which the bearing walls and other major structural elements generally have no fire-resistance rating.

3.1.3 Group II: Combustible Construction

Buildings or portion thereof in Combustible Construction Group II are those in which the walls, exit-ways, shafts, structural members, floors, and roofs are constructed wholly or partly of combustible materials having fire-resistance ratings specified in Table 3.3.1 (b). The Non-combustible group consists of Construction Type II-A, II-B, II-C, II-D and II-E.

3.1.3.1 Construction Type II-A

This Construction type includes heavy timber construction in which fire-resistance is attained by—

(a) Limiting the minimum sizes of wood structural members and the minimum thickness and composition of wood floors and roofs;

(b) Avoiding concealed spaces under floors and roofs or by providing fire-stopping protection for these spaces; and

(c) Using fastening, construction details, and adhesives for structural members as required by this Chapter and Part 4.

(d) The minimum dimensions for framing members shall be prescribed in this Chapter and Part 4, except that members are protected to provide a fire-resistance rating of at least one hour need not comply with this requirement.

3.1.3.2 Construction Type II-B

This Construction type includes buildings and portion thereof in which

(a) Exterior walls, fire walls, exit-ways, and shaft enclosures are of non-combustible materials having the required fire-resistance ratings; and

(b) The floors, roofs and interior framing are wholly or partly of wood of smaller dimensions than required for type II-A construction, or are of other combustible or non-combustible materials, having the required fire-resistance rating.

Table 3.3.1 (a): Fire Rating for Construction Group I: Non-Combustible

Exterior wall with Fire Separation Distance of	Construction Element Exterior Wall	TYPE -I-A		TYPE -I-B		TYPE -I-C		TYPE -I-D		TYPE -I-E	
		Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]
0.9m or less	Bearing	4	N.P	3	N.P	2	N.P	2	N.P	2	N.P
	Non-bearing[f]	2		2		2		2		2	
More than 0.9m but less than 4.5m	Bearing	4	as per provisions of this Code	3	as per provisions of this Code	2	as per provisions of this Code	2	as per provisions of this Code	2	as per provisions of this Code
	Non-bearing[f]	2		2		2		2		2	
4.5m or more but less than 9.0m	Bearing	4		3		2		1		0	
	Non-bearing[f]	1½		1½		1		1		0	

Exterior wall with Fire Separation Distance of	Construction Element Exterior Wall	TYPE -I-A		TYPE -I-B		TYPE -I-C		TYPE -I-D		TYPE -I-E	
		Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]
9.0m or more	Bearing	4	N.L	3	N.L	2	N.L	1	N.L	0	N.L
	Non-bearing[f]	0		0		0		0		0	
Interior bearing walls and bearing partitions		4		3		2		1		0[g,i]	
Enclosure of vertical exits[e], exit passageways, hoistways and shafts		2		2		2		2		2	
Fire divisions and fire barrier Walls or partitions or ceiling slab		See Table 3.2.1 and provisions of this Code									
Columns[k], girders, trusses (other than roof trusses) and framing	Supporting one floor	3		2		1½		1		0[g,i]	
	Supporting more than one floor[l]	4		3		2		1		0[g,i]	
Structural members supporting walls		Structural members shall have the same fire resistance rating of wall to be supported, but not less than rating required by the construction classification.									
Floor construction including beams		3		2		1½		1		0[g,i]	
Roof construction, including beams, trusses and framing including arches, domes, shells, cable supported roofs and roof decks[h]	4.5m or less in height above floor to lowest member of ceiling	3		1½		1[i]		1[i]		0[g,i]	
	4.5m to 6m in height above floor to lowest member of ceiling	2[c,i] or 1[d,i]		1½[c,i] or 1[d,i]		1[i]		1[i]		0[g,i]	
	6m or more in height above floor to lowest member of ceiling	2[c,i] or 0[d,g,i]		1½[c,i] or 1[d,g,i]		1[c,i] or 0[d,g,i]		1[c,i] or 0[d,g,i]		0[g,i]	

Exterior wall with Fire Separation Distance of	Construction Element Exterior Wall	TYPE -I-A		TYPE -I-B		TYPE -I-C		TYPE -I-D		TYPE -I-E	
		Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]
Shafts (other than exits) and elevator hoistways		2		2		2		2		2	
Fire separation wall and party wall		4		2		2		2		2	
Access corridor leading to fire exits		1		1		1		1		1	

Non-combustible Material; N. P Not Permitted; N. L No Limit

Table 3.3.1 (b): Fire Rating for Construction Group II: Combustible

Exterior wall with Fire Separation Distance of	Construction Element Exterior Wall	TYPE -II-A		TYPE -II-B		TYPE -II-C		TYPE -II-D		TYPE -II-E	
		Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]
0.9m or less	Bearing	2	N.P	2	N.P	2	N.P	2	N.P	2	N.P
	Non-bearing[f]	2		2		2		2		2	
More than 0.9m but less than 4.5m	Bearing	2	as per provisions of this Code	2	as per provisions of this Code	2	as per provisions of this Code	1	as per provisions of this Code	1	as per provisions of this Code
	Non-bearing[f]	2		2		2		1		1	
4.5m or more but less than 9.0m	Bearing	2		2		2		1		0	
	Non-bearing[f]	2		2		2		1		0	
9.0m or more	Bearing	1	N.L	1½	N.L	1½	N.L	1	N.L	0	N.L
	Non-bearing[f]	0		0		0		0		0	
Interior bearing walls and bearing partitions		2		1		0		1		0	
Enclosure of vertical exits[e], exit passageways, hoistways and shafts		2		2		1[i]		1[i]		1	
Fire divisions and fire barrior Walls or partitions or ceiling slab		See Table 3.2.1 and provisions of this Code									
Columns[k], girders, trusses (other than roof trusses) and framing	Supporting one floor	as per provisions of this Code		1		0 or 1[j]		1		0	
	Supporting more than one floor	as per provisions of this Code		1		0 or 1[j]		1		0	

Exterior wall with Fire Separation Distance of	Construction Element Exterior Wall	TYPE -II-A		TYPE -II-B		TYPE -II-C		TYPE -II-D		TYPE -II-E	
		Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]	Ratings in Hours	Exterior Opening[a,b]
Structural members supporting walls		3		2½		2		1½		1	
Floor construction including beams		as per provisions of this Code		1		0 or 1[j]		1		0	
Roof construction, including beams, trusses and framing including arches, domes, shells, cable supported roofs and roof decks[h]	4.5m or less in height above floor to lowest member of ceiling	as per provisions of this Code		3/4		0		3/4		0	
	4.5m to 6m in height above floor to lowest member of ceiling	as per provisions of this Code		3/4		0		3/4		0	
	6m or more in height above floor to lowest member of ceiling	as per provisions of this Code		3/4		0		3/4		0	
Shafts (other than exits) and elevator hoistways		2		2		2		2		2	
Fire separation wall and party wall		4		2		2		2		2	
Access corridor leading to fire exits		1		1		1		1		1	

Non-combustible Material ; N. P Not Permitted ; N. L No Limit

Notes:

[a] Requirements of protected exterior openings shall not apply to religious assembly. [Protected openings within an exterior separation of 0.9m or less are permitted for buildings classified in Occupancy Groups A provided, however said openings do not exceed in total area of 25% of the façade of the storey in which they are located. The openings however, may not be credited towards meeting any of the mandatory natural light and ventilation as per provisions of this Code. Protection of openings with an exterior separation of 0.9 m to 9 m shall not be required for A-1, A-2 and A-3 Occupancy groups] or to buildings classified in Occupancy groups J, G and H additional requirements for exterior walls and exterior wall openings as per provisions of this Code.

[b] Upon special application, the area development authorities may permit exterior wall openings to be constructed in excess of the permitted area established by this Table if such openings at the time of their construction are located at least 18m in a direct line from any neighboring building except as otherwise permitted in footnote f. Such additional openings may not however be credited toward meeting any of the mandatory natural light and ventilation requirements of Sec 1.19 Part 3 of this Code. If any neighboring building is later altered or constructed to come within the above distance limitation, the affected exterior openings shall immediately be closed with construction meeting the fire-resistance rating requirements for exterior wall construction of the building in which they are located.

[c] Applies to occupancy groups J, G and H

[d] Applies to occupancy groups J, G and H

[e] See Provisions of this Code for additional impact resistance requirements applicable to certain stair enclosures and for certain exceptions to stair enclosure requirements.

[f] When two or more buildings are constructed on the Plot and the combined floor area of the buildings does not exceed the limits established by this Code for any of the buildings, not fire-resistance rating shall be required for non-bearing portions of the exterior walls of those buildings facing each other, and there shall be no limitation on the permitted amount of exterior openings.

[g] Fire retardant treated wood complying with the requirements of this Code may be used.

[h] Tabulated ratings apply to buildings over one storey in height. In one storey building, roof construction may be of material having zero hour fire-resistance rating.

[i] Materials which are not non-combustible as defined in this Code may be used in nonbearing construction elements as per provisions for this Code.

[1] Materials having a structural base of non-combustible materials as defined in this Code, and having a surface not over 3.2 mm thick which when tested in accordance with the provisions of this Code has a flame spread rating not higher than 50 (fifty).

[2] Materials which when tested in accordance with the provisions of this Code have a surface flame spread rating not higher than twenty five without evidence of continued progressive combustion, and which are of such composition that surface which would be exposed by cutting through the material in any way would not have a flame spread ratings higher than twenty-five without evidence of continued progressive combustion.

[j] Applies to the construction of the street floor and all construction below the level of the street floor in building or spaces classified in occupancy group A-3 except where the space below the street floor does not exceed five feet in height.

[k] Columns supproting the roof of a one-story building shall have the same fire-resistance rating as required for a column supporting one floor in a building of the same construction class.

[l] Members supporting loads of not more than two floors or one floor and a roof need not have a fire-resistance rating greater than the floor construction fire-resistance requirement in buildings classified in occupancy groups B, C and A-3, not including unsprinklered spaces of other occupancies, and in fully sprinklered buildings in occupancy groups E and A-5.

3.1.3.3 Construction Type II-C

This Construction type includes buildings and portion thereof in which

(a) Exterior walls, fire walls, exit ways, and shaft enclosures are of non-combustible materials having the required fire-resistance ratings; and

(b) The floors, roofs and interior framing are wholly or partly of wood of smaller dimensions than required for type II-A construction, or are of other combustible or non-combustible materials, having no required fire-resistance rating.

3.1.3.4 Construction Type II-D

This Construction type includes buildings and portion thereof in which exterior walls, bearing walls, floors, roofs, and interior framings are generally of wood or other combustible materials having the required fire-resistance ratings.

3.1.3.5 Construction Type II-E

This Construction type includes buildings and portion thereof in which

(a) The exterior walls are generally of wood or other combustible materials having the required fire-resistance ratings, and

(b) In which the bearing walls, floors, roofs, and interior framing are of wood or other combustible materials, generally having no fire-resistance ratings.

3.1.4 Separated Occupancy and Construction

When two or more occupancies accommodated in a building, each such occupancy shall be separated according to the provisions specified in Sec 2.3 Chapter 2 Part 3 and Table 3.2.1.

When two or more types of construction used within a building, the entire building shall be subject to the most restrictive construction type and shall comply with FAR restrictions as per provisions of this Code.

However if the Occupancies within the different Types of Construction are completely separated by construction that meets the fire-resistance rating requirements for fire separation listed in Table 3.2.1 of Chapter 1 Part 3 then each Occupancy so separated may, for the purpose of this Code, be considered as separate building section.

3.1.4.1 Restriction for mixed construction

In buildings of mixed construction, no structural element shall be supported by construction having a lower fire-resistance rating than that required for the element being supported.

3.1.5 Fire Zones

The planning and development authority of the city, township, municipality or region where this Code is intended to be implemented shall divide the area under their jurisdiction into distinct fire zones. The basis for this zoning shall be the fire hazard inherent in the buildings and the degree of safety desired for the occupancy accommodated therein. The number of zones in an area shall depend on its size and the strategies undertaken for its development.

3.1.5.1 Fire Zone 1

The following occupancy groups shall comprise this zone:

Occupancy A:	Residential	Occupancy F:	Mercantile
Occupancy B:	Educational	Occupancy H:	Livestock Storage Building
Occupancy C:	Institutional for Care	Occupancy I:	Assembly
Occupancy D:	Health Care	Occupancy K:	K1 and K2 Parking
Occupancy E:	Offices	Occupancy M:	Miscellaneous Buildings

3.1.5.2 Fire Zone 2

The following occupancy groups shall comprise this zone:

Occupancy G:	Industrial Buildings
Occupancy H:	Storage Buildings
Occupancy K:	K3 Parking

3.1.5.3 Fire Zone 3

The only occupancy falling in this zone shall be Occupancy J, Hazardous Buildings.

3.1.5.4 Change in Fire Zone Boundaries

The demarcations of fire zones may be changed or new occupancies may be included in any fire zone through the same procedure as for promulgating new rules or ordinances or both.

3.1.5.5 Buildings on overlapping fire zones

Buildings falling on more than one fire zones shall be considered to be situated on the zone in which the major portion of the building falls. If a building is divided equally between more than one fire zones, it shall be considered as falling in the fire zone having more hazardous occupancy buildings.

3.1.5.6 Restrictions on temporary constructions

Permission may be granted by the Authority for temporary constructions only in fire zones 1 and 2 and not in fire zone 3. Such temporary constructions shall adhere to the conditions of the permission and shall be demolished and removed completely after the expiry of the duration of the permission unless it is extended by the Authority or a new permission is obtained.

3.1.6 Permissible Types of Construction for Various Occupancies

3.1.6.1 New buildings

Types of constructions permitted for various buildings on the basis of fire zones are specified in Table 3.2.4.

3.1.6.2 Existing buildings

Existing buildings in any fire zone need not comply with the provision of this Code for type of construction unless they are altered or in the opinion of the Authority they constitute a hazard to the safety to the occupants of the buildings or the adjacent properties.

3.1.7 Exterior Walls

The fire resistance rating of the exterior walls shall conform to the provisions set forth in Table 3.2.2 and Sec 3.2.3.

3.1.8 Basement Floor

Basement floor of a building shall be enclosed with a one hour fire resistive construction. Doors in such constructions shall be made of noncombustible materials.

3.1.9 Restricting Horizontal and Vertical Spread of Fire

Generally walls restrict horizontal movement and slabs restrict vertical movement of fire.

3.1.9.1 Interior or barrier or enclosure wall

Propagation of fire, smoke, gas or fume through the openings or shafts or penetrations of fire resistive floors and walls shall be restricted by sealing with an approved material which shall have a fire resistance rating at least equal to that of the floor-wall assembly. The sealing material shall be capable of preventing passage of flame and hot gases sufficient to ignite cotton waste when tested in accordance with ASTM E119.

3.1.9.2 Exterior walls

Permitted unprotected openings in the exterior wall in two consecutive floors lying within 1.5 m laterally or vertically shall be separated with flame barriers as similar as sunshades or cornices or projected wall at least 750 mm from the external face of the exterior wall. The flame barrier shall have a fire resistance rating of not less than three-fourths hour.

3.1.10 Exceptions to Fire Resistance Requirements

The provisions of this Section are exceptions to the occupation separation requirements of Table 3.2.1.

3.1.10.1 Fixed partitions

(a) Stores and Offices: In such cases where offices, stores and similar places occupied by one tenant are separated by non-load bearing walls that do not form a corridor serving an occupant load, the partition walls may be constructed of any one of the following:

- (i) Noncombustible materials;
- (ii) Fire retardant treated wood;
- (iii) One hour fire resistive construction;
- (iv) Wood panels or similar light construction up to three fourths the height of the room in which placed; and
- (v) Wood panels or similar light construction more than three-fourths the height of the room in which placed with not less than upper one fourth of the partition constructed of glass.

(b) Hotels and Apartments: In such cases where non-load bearing walls act as interior partitions in individual dwelling units in apartment houses and guest rooms or suites in hotels when such dwelling units, guest rooms or suites are separated from each other and from corridors by not less than one-hour fire-resistive construction, the partition walls may be constructed of any one of the following:

(i) Noncombustible materials of fire retardant treated wood in buildings of any type of construction; or

(ii) Combustible framing with noncombustible materials applied to the framing in buildings of Type II construction.

(c) Folding, Portable or Movable Partitions: Folding, portable or movable partitions need not have a fire resistance rating if the following conditions are satisfied:

(i) Required exits are not blocked without providing alternative conforming exits;

(ii) Tracks, guides or other approved methods are used to restrict their locations; and

(iii) Flammability shall be limited to materials having a flame-spread classification as set forth in Table 3.3.2 for rooms or areas.

Table 3.3.2: Flame Spread Classification

Class	Flame Spread Index
I	0-25
II	26-75
III	76-200

(d) Walls Fronting on Streets or Yards: For walls fronting on a street or yard having a width of at least 12 m, certain elements of the wall may be constructed as follows regardless of their fire-resistive requirements:

(i) Bulkheads below show windows, show window frames, aprons and show-cases may be of combustible materials provided the height of such construction does not exceed 5 m above grade.

(ii) Wood veneer of boards not less than 25 mm in nominal thickness or exterior type panels not less than 10 mm in nominal thickness may be used in walls provided:

- the veneer does not extend beyond 5 m above grade; and
- The veneer is placed either directly against noncombustible surface or furred out from such surfaces not to exceed 40 mm with all concealed spaces fire blocked.

(e) Trim: Wood may be used to construct trim, picture moulds, chair rails, baseboards, handrails and show window backing. If there is no requirement for using fire protected construction, unprotected wood doors and windows may be used.

(f) Loading Platform: Noncombustible construction of heavy timber may be used for exterior loading platforms with wood floors not less than 50 mm in nominal thickness. Such wood construction shall not be carried through the exterior walls.

(g) Insulating Boards: Combustible finished boards may be used under finished flooring.

3.1.11 Shaft Enclosures

3.1.11.1 General

Construction requirement for shafts through floors shall conform to the provisions of Tables 3.3.1 (a) and (b).

3.1.11.2 Extent of enclosures

Shaft enclosures shall extend from the lowest floor opening through successive floor openings and shall be enclosed at the top and bottom.

Exceptions:

(a) Shafts need not be enclosed at the top if it extends through or to the underside of the roof sheathing, deck or slab.

(b) Noncombustible ducts carrying vapours, dusts or combustion products may penetrate the enclosure at the bottom.

(c) Shafts need not be enclosed at the bottom when protected by fire dampers conforming to "Test Methods for Fire Dampers and Ceiling Dampers", installed at the lowest floor level within the shaft enclosure.

3.1.11.3 Special provision

In groups other than Occupancies C and D, openings which penetrate only one floor and are not connected with any other floor or basement and which are not concealed within building construction assemblies need not be enclosed.

3.1.11.4 Protection of openings

Openings in shaft enclosures shall be protected with a self-closing or an automatic-closing fire assembly having a fire resistance rating of

(a) one hour for one hour fire resistive walls

(b) one and one-half hours for two hour fire resistive walls

3.1.11.5 Rubbish and linen chute termination rooms:

Rubbish and linen chute shall terminate in rooms separate from the remaining of the building having the same fire resistance as required for shafts in Table 3.3.1 (a) and (b) but not less than one hour.

3.1.12 Expansion and Contraction Joints

Expansion and contraction joints provided to accommodate expansion, contraction, wind or seismic movement shall be protected with an approved material having the same degree of fire resistance as that of the wall or floor in which it is installed.

3.1.13 Weather Protection

3.1.13.1 Weather resistive barrier

All weather exposed surfaces shall have a weather barrier to protect the interior wall from damping. Such weather barriers shall have a fire resistance rating of at least equal to that of the wall or floor on which it is applied. Weather resistive barrier need not be used in the following cases:

(i) When exterior covering is of approved waterproof panels

(ii) In back plastered construction

(iii) When there is no human occupancy

(iv) Over water repellent panel sheathing

(v) Under approved paper backed metal or wire fabric lath

(vi) Behind lath and Portland cement plaster applied to the underside of roof and eave projections

3.1.13.2 Flashing and counter flashing

Exterior openings exposed to the weather shall be flashed to make them weather proof. There shall be copings with all parapets. Corrosion resistant metals shall be used for flashing, counter flashing and coping.

3.1.13.3 Waterproofing weather-exposed areas

Waterproofing shall be applied to exposed surfaces like balconies, external stairways and landings.

3.1.13.4 Damp-proofing foundation walls

Outside of foundation walls enclosing a basement floor below finished grade shall be damp-proofed from outside.

3.1.14 Members Carrying Walls

All members carrying masonry or concrete walls shall be fire protected as specified in Table 3.3.1 (a) and (b).

3.1.15 Parapets

Parapets constructed on exterior wall of a building shall have the same degree of fire resistance required for the wall upon which they are erected and there shall be noncombustible faces on the side adjacent to the roof surface for the uppermost 405 mm including counter flashing and coping materials. The height of the parapet shall be at least 750 mm from the upper surface of the roof.

3.1.16 Projections

Sunshades, cornices, projected balconies and overhanging beyond walls of Type I construction shall be of noncombustible materials. Projections from walls of Type II may be of combustible or noncombustible materials.

3.1.17 Guards and Stoppers

3.1.17.1 Guards

Guards or Guardrails shall be provided to protect edges of floor, roof, roof openings, stairways, landings and ramps, balconies or terraces and certain wall, which are elevated more than 750 mm above the grade and as per provisions of this Code.

3.1.17.2 Stoppers

Stopper shall be provided in open parking garages located more than 450 mm above the adjacent grade or back to back parking stall. The height of the stopper shall be at least 300 mm and it shall be positioned at outer edges of a car parking stall.

3.1.18 Insulation

The provisions of this Section are applicable to thermal and acoustical insulations located on or within floor-ceiling and roof ceiling assemblies, crawl spaces, walls, partitions and insulation on pipes and tubing.

Materials used for such insulation and covering shall have a flame spread rating not more than 25 and a smoke density not more than 450.

3.1.19 Atrium

3.1.19.1 General

Atrium may be provided in all groups other than Occupancy J (Hazardous Buildings). Such atrium shall have a minimum opening and are as specified in Table 3.3.3. A vertical opening serving as other than an exit enclosure connecting only two adjacent stories shall be permitted to be open to one of the two stories.

Table 3.3.3: Atrium Opening and Area

Height in Stories	Minimum Clear Opening[1] (m)	Minimum Area (m^2)
2-4	6	40
5-7	9	90
8 or more	12	160

[1] The specified dimensions are the diameters of inscribed circles whose centers fall on a common axis for the full height of the atrium.

3.1.19.2 Smoke control system

A mechanically operated air-handling system shall be installed to exhaust the smoke either entering or developed within the atrium.

(a) Exhaust Openings: The location of the exhaust openings shall be in the ceiling or in a smoke trap area immediately adjacent to the ceiling of the atrium above the top of the highest portion of door openings into the atrium.

(b) Supply Openings: Supply openings designed for a minimum of 50 percent of the exhaust volume shall be located at the lowest level of the atrium. Supply air may be introduced by gravity provided the height of the atrium is not more than 18 m and smoke control is established. For atria having height greater than 18 m, supply air shall be introduced mechanically from the floor of the atrium and directed vertically toward the exhaust outlets. Supplemental air supply may be introduced at upper levels in atrium over six storeys in height or when tenant spaces above the second storey are open to the atrium.

(c) Automatic Operation: The smoke control system for the atrium shall be activated automatically by the automatic sprinkler system or smoke detectors installed within the atrium or areas open to the atrium.

(d) Manual Operation: The smoke control system shall also be manually operable for use by the fire department. The smoke control system may be separate from or integrated with other air handling systems. Air handling systems interfering with the smoke control system shall be shut down automatically when the smoke control system is activated.

(e) Smoke Detector Location: Smoke detectors which will automatically operate the smoke control system of the atrium shall be accessible for maintenance, testing and servicing. Their locations shall be as follows:

(i) At the atrium ceiling, spaced in accordance with the manufacturer's instructions.

(ii) On the underside of projections into the atrium, in accordance with the manufacturer's instructions.

(iii) Around the perimeter of the atrium opening on all floors open to the atrium. These detectors shall be spaced no more than 9 m on centre and shall be located within 5 m of the atrium opening.

(iv) If projected beam type smoke detectors are used, they shall be installed in accordance with manufacturer's instructions.

(f) Enclosure of Atrium: Atria shall be separated from the adjacent spaces with fire resistive separation of at least one hour.

Fire windows may be provided in fixed glazed openings when the window has a fire resistive rating of at least three-fourths hour and the area of the opening does not exceed 25 percent of the wall common to the atrium and the room into which the opening is provided.

3.1.20 Mezzanine Floors

Construction of a mezzanine floor shall conform to the requirements of the main floor in which it is constructed but the fire resistance rating need not exceed one hour for unenclosed mezzanines.

PART III
Chapter 4
Energy Efficiency and Sustainability

4.1 Scope

The purpose of including this Chapter in the Code is to enhance the design and construction of buildings through the use of building concepts having a positive environmental impact and encourage sustainable construction practices, allowing efficiency and conservation of energy, water and building materials, and to promote resource efficiency.

In addition to the clauses stipulated here, all Codes and standards relevant to a building occupancy as set forth in other Sections of this Code will be applicable during implementation.

Design and drawings will be submitted to indicate the location, nature and scope of the proposed energy efficient/sustainable feature. These shall indicate compliance to the provisions of this Code, and will be supplied by the relevant design professionals, e.g. electrical engineers, mechanical engineers, plumbing engineers, etc., supporting architectural drawings.

4.1.1 Rationale for Sustainable/Green Buildings

Climate change is an established phenomenon affecting the environment globally and it is recognized that buildings and the built environment play a vital role in the process, impacting on the natural environment and the quality of life. Sustainable development concepts and approaches applied to the design, construction and operation of buildings or to any built environment can enhance both the economic and environmental benefits of the community in Bangladesh and around the world. Energy efficiency and sustainability is not an individual issue rather an integrated and inseparable part of the building design and construction process. The benefits of sustainable design principles include resource and energy efficiency, healthy buildings and materials, ecologically and socially sensitive land use and strengthened local economics and the communities, objectives vital for future development of Bangladesh.

4.2 Definitions

DAYLIGHT ZONE	An area with a depth of 5m parallel to any glazed external wall.
EMERGENCY LIGHTING	Lighting used for emergency spaces and functions, e.g. in fire stairs, for egress path signage.
GREY WATER	Waste water generated from wash hand basins, showers and baths, Grey water often excludes discharge from laundry, dishwashers and kitchen sinks due to the high nutrient levels. It differs from the discharge of WC's which is designated sewage or black water to indicate it contains human waste.
REGULARLY OCCUPIED SPACE	All the main areas in the buildings that are used on a frequent basis, such as living rooms, bedrooms, classrooms, lobbies, meeting rooms, hall rooms and office spaces. Service spaces like toilets, bathrooms, corridors and stores will not be considered as frequently occupied areas.
WINDOW TO WALL RATIO OF BUILDING (WWRB)	The window-to-wall ratio of a building is the percentage of its facade taken up by light-transmitting glazing surfaces, including windows and translucent surfaces such as glass bricks. It does not include glass surfaces used ornamentally or as cladding, which do not provide transparency to the interior. Only facade surfaces are counted in the ratio, and not roof surfaces.
LIGHTING POWER DENSITY (LPD)	Average total lighting power installed divided by the total occupied area.
SHADING COEFFICIENT (SC)	The ratio of solar heat gain at normal incidence through glazing to that occurring through 1/8 inch thick clear, double-strength glass. Shading coefficient, as used herein, does not include interior, exterior, or integral shading devices.
SOLAR HEAT GAIN COEFFICIENT (SHGC)	An indicator of glazing performance is the amount of heat admitted through the glass vis-à-vis the total heat incident on the glass by virtue of direct solar radiation. The unit is a simple fraction or percentage.

U-VALUE (THERMAL TRANSMITTANCE)	Heat transmission in unit time through unit area of a material or construction and the boundary air films, induced by unit temperature difference between the environments on each side. Units of U-value are $W/m^2/^{o}k$
VISIBLE LIGHT TRANSMITTANCE (VLT)	Amount of light transmitted through glazing, expressed as a simple fraction or percentage

4.3 Site Sustainability

This Section deals with sites to ensure energy efficiency through passive and low energy architectural features and management of resources.

4.3.1 Mandatory Unpaved Area

Fifty (50) percent of mandatory open space shall be permeable on sites of all occupancy categories. The permeable area shall not remain bare generating dust, but will have green cover or be treated with perforated paving (≥ 50%), organic mulch, charcoal, etc.

4.3.2 Site Drainage and Run-Off Coefficient

Designs shall indicate site drainage considerations along with flash flooding and erosion prevention measures for sites above 1340 m^2 in area. As excessive paving is largely responsible for fast water run-off and flash flooding, design shall indicate measures taken to make paving permeable. The net run-off from a site shall be a maximum of sixty (60) percent. The following method will be used for the calculations, in conjunction with Table 3.4.1:

Total Perviousness on Open Area of Site $(A_p) = A_1 \times C_1 + A_2 \times C_2 + \ldots$ (3.4.1)

Where, A_1, A_2, etc., being the areas of various surfaces, e.g. Pavements, roads, vegetation, etc., with different run-off coefficients C_1, C_2, C_3 etc., shown in the Table 3.4.1.

4.3.3 Vegetation Plan

For sites above three (3) acres, it is mandatory for a vegetation plan to be submitted along with the site plan and Irrigation Plan, where priority shall be given to native plants in the selection for planting.

4.3.4 Irrigation Plan

4.3.4.1 For sites above ten (10) acres, an irrigation plan with construction details shall be submitted with the site plan, where considerations shall include for management of rainwater.

4.3.4.2 For these sites a retention pond of ≥ 3% of site area shall be provided. This shall include any existing natural water body within the site.

Table 3.4.1: Run-Off Coefficients of Various Surfaces

Surface Type	Run-Off Coefficient, C
Roofs, conventional	0.95
Green Roofs (soil/growing medium depth ≥ 300 mm)	0.45
Concrete paving	0.95
Gravel	0.75
Brick paving	0.85
Vegetation:	
1-3%	0.20
3-10%	0.25
>10%	0.30
Turf Slopes:	
0-1%	0.25
1-3%	0.35
3-10%	0.40
>10%	0.45

4.3.5 Rain Water Harvesting System

4.3.5.1 Buildings of total floor area ≥ 4000 m^2 shall have its own rain water harvesting system as discussed in Chapter 7 Part 8 and installed complying with Section 7.13 Part 8, of this Code. The reservoir capacity shall be a multiple of the area of Ground Coverage of the building and a rain collection coefficient of 0.073.

4.3.5.2 The rainwater reservoir may be placed under the roof or at lower levels, including underground.

4.4 BUILDING ENVELOPE

4.4.1 Window to Wall Ratio

4.4.1.1 For mechanically ventilated and cooled buildings of all occupancies, other than Hazardous and Storage, the Window to Wall ratio of building (WWRB), will be determined in conjunction with the glazing performance, as indicated by the Solar Heat Gain Coefficient (SHGC) or Shading Coefficient (SC) of the glass used. The relationship is given in Figure 3.4.1 and Table 3.4.2.

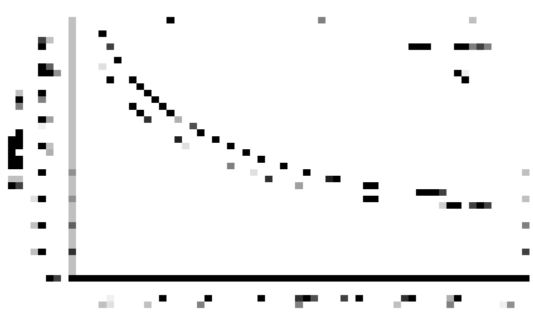


Figure 3.4.1 Selection of glazing SHGC based on WWR

Table 3.4.2 Selection of Glazing SHGC Based on WWR in Tabular Format

WWR	SHGC	SC
10	0.85	0.98
20	0.6	0.69
30	0.5	0.57
40	0.4	0.46
50	0.35	0.4
60	0.33	0.38
70	0.31	0.36
80	0.3	0.34
90	0.27	0.31

4.4.1.2 In all of the above cases, the Visible Light Transmittance (VLT) of the glazed element shall not be lower than thirty five (35) percent.

4.4.1.3 For Air-conditioned buildings with external shading, permitted SGHC limit may be adjusted, but the increase shall not exceed values determined by Eq. 3.4.2 below:

$$SHGC_{adj} = SHGC + A \qquad (3.4.2)$$

Where,

$SHGC_{adj}$ is the adjusted solar heat gain coefficient limit for windows with shading

SHGC is the solar heat gain coefficient from Table 3.4.2

A is the SHGC correction factor for the external shading as per Table 3.4.3 or Table 3.4.4: For a window with overhang and fin, the value of A can be only used either from overhang or from fin.

4.4.1.4 For naturally ventilated buildings, window size shall be based on Sec 4.4.2. Window Openings of this Code and shading shall be provided as per Sec 4.4.3.

4.4.1.5 Window size shall under no circumstances be less than as stipulated under Part 3: Chapter 1, Section 1.17 of this Code.

Table 3.4.3: Correction Factor against Overhang Shading Projection Factor

Overhang Projection Factor	SHGC Correction Factor(A)
0.0	0.00
0.1	0.05
0.2	0.09
0.3	0.14
0.4	0.19
0.5	0.24
0.6	0.28
0.7	0.33
0.8	0.38
0.9	0.43
1 or higher	0.47

Projection factor for overhang is the depth of the overhang divided by the height of the window

Table 3.4.4: Correction Factor against Vertical Shading (fins) Projection Factor

Vertical Shading (Fins) Projection Factor	SHGC Correction Factor (A)
0.0	0.00
0.1	0.04
0.2	0.08
0.3	0.12
0.4	0.16
0.5	0.20
0.6	0.24
0.7	0.28
0.8	0.32
0.9	0.36
1 or higher	0.40

Projection factor of fins is the depth/length of fin divided by the width of the window.

4.4.2 Window Openings

Mechanically ventilated and cooled buildings of all occupancies, other than hazardous, retail and storage, shall have the provision of using natural ventilation for cooling and fresh air, in frequently occupied areas , with a fraction $\geq$ 4% of the floor area being specified as openable windows. Openable balcony doors can be counted in this calculation. Note if the window area defined under Sec 4.4.1 is less than openable area, then fifty (50) percent of window area should be openable.

4.4.2.1 Naturally ventilated buildings of all occupancies, other than hazardous and storage, shall provide for fifty (50) percent of its window area to be openable.

4.4.2.2 All the openable windows above ground should be designed with safety measures in place such as protection hand rails for child safety.

4.4.2.3 Windows to any regularly occupied space on exterior walls in naturally ventilated buildings shall be shaded conforming to Sec 4.4.3.

4.4.3 Shading

4.4.3.1 For naturally ventilated buildings of all occupancies, horizontal sunshades shall be provided over windows on South, East and West, the depth of which shall be calculated by multiplying the window height with a factor of 0.234 (Figure 3.4.2). Horizontal louvers can be used instead of sunshades, in which case, depth of louver shall not be less than 0.234 times the gaps between the louvers (Figure 3.4.3).

4.4.3.2 Vertical Shading devices shall be provided on the West, depth of which shall be calculated, by multiplying the gaps between the vertical fins, or the window width if the shades border the window width, with a factor of 0.234 (Figure 3.4.4).

Exceptions:

(a) The above rule shall be relaxed if it can be demonstrated that shading is achieved by existing neighbouring structures.

(b) The north side of all buildings are exempt from the above rules.

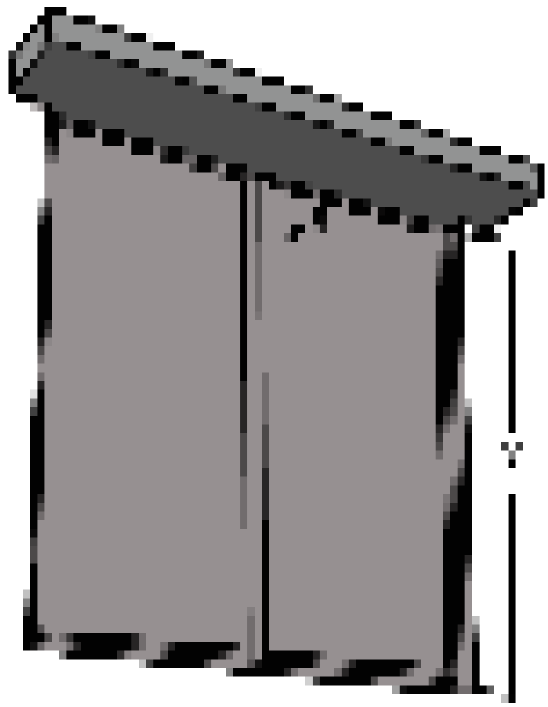


Figure 3.4.2 Horizontal shade: $x \geq 0.234y$

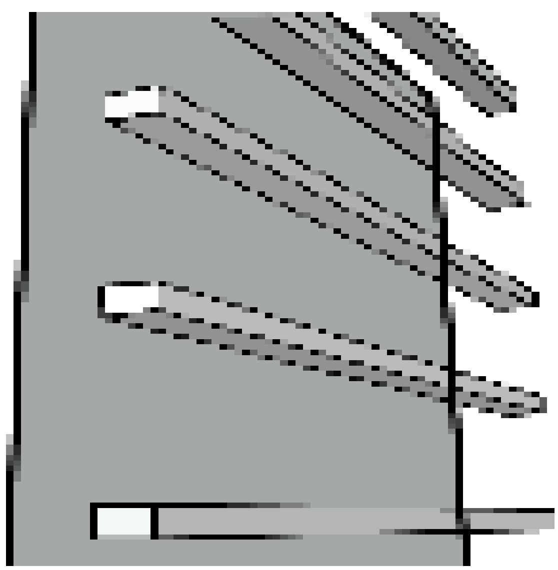


Figure 3.4.3 Horizontal Louvres: relationship between depth (x) and gap (y): $x \geq 0.234y$

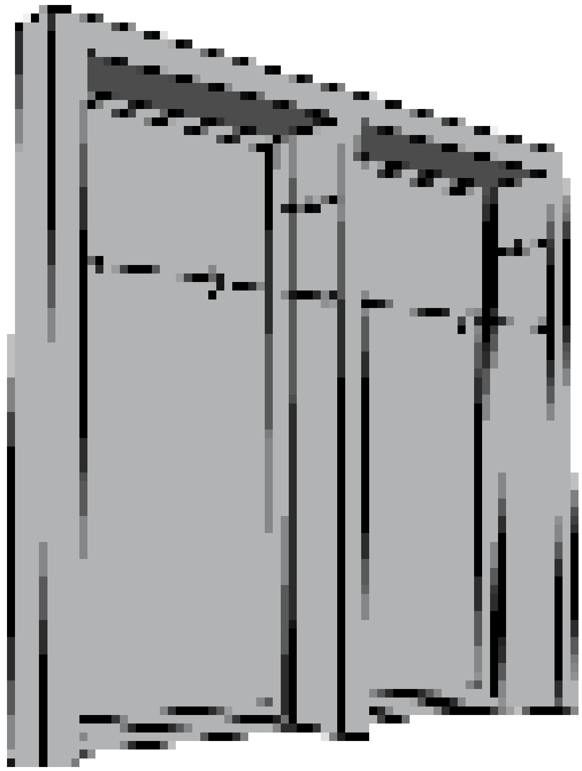


Figure 3.4.4 Vertical shading or louvres: relationship between depth (x) and gap (y): $x \geq 0.234y$

4.4.4 Roof Insulation and Green Roofing System

4.4.4.1 Fifty (50) percent of horizontal exposed roof slabs of Buildings of Occupancy B, C, D and E, shall have green roofing system, to manage water run-off from roof tops, to control internal temperatures within the top floors and to reduce the carbon footprint of the building. This shall not include any covered roof surface, e.g. solar panels, solar thermal heaters, machinery for mechanical or electrical systems, water tanks, etc. Stair loft or machine room tops will be exempt from this rule.

(a) The roof slab design shall consider structural support of the green roof system, with growing medium of minimum 300 mm.

(b) The design will indicate protection from dampness and provide a drainage system

4.4.4.2 Horizontal roof slabs, which are not covered by green roofing system, will have roof slabs with insulation, so that the time lag and decrement factor is greater than the other floor slabs of the building.

4.5 Energy Efficient Building Systems

4.5.1 Daylighting and Supplementary Lighting System

4.5.1.1 Window area shall not be less than 14 percent or 1/7th of the total floor area of the building.

4.5.1.2 Every regularly occupied space shall contain a minimum percentage of day-lit area along the building perimeter zones, with no window less than an area of 1 m^2 and will ensure the appropriate stipulations given below.

(a) for rooms that measure less than 8 m in depth, window area shall be at least 20 percent of the area of the external wall of the room.

(b) for rooms that measure between 8 to 14 m in depth, window area shall be at least 30 percent of the area of the external wall of the room and 35 percent of the external wall.

(c) for rooms that measure more than 14 m in depth, window area shall be at least 35 percent of the area of the external wall of the room.

4.5.1.3 For Buildings of Occupancy A5, B, C, E1 and E2, photoelectric sensors shall be connected to luminaires, to enable dimming or switching off lamps that do not require to be operated, due to the presence of adequate daylight. The photoelectric sensor shall be located approximately at half (½) the depth of day-lit zone.

4.5.1.4 If occupancy sensors are installed in the daylight area, the occupancy sensor shall override the daylight sensor during non-occupancy period.

Exceptions:

(a) Zones with special requirements are exempt from the stipulation of Sec 4.5.1.3. The designer shall justify the reason for exemption.

(b) Hotel guest rooms are exempt.

4.5.2 Lighting Power Density

4.5.2.1 Lighting Power Density (LPD) of the values set in Table 3.4.5 shall be provided for the respective functions within all building occupancies, or as specified.

4.5.2.2 In addition to Sec 4.5.2.1, Illumination values (Lux) as specified in Tables 8.1.5 to 8.1.14 of Part 8 of this Code shall be provided for buildings of the respective occupancies.

4.5.3 Occupancy Sensors

4.5.3.1 In order to limit the use of electricity in the unoccupied areas of buildings, occupancy sensors linked to lighting (except for emergency and security lighting) shall be installed in the public areas of buildings of occupancies specified in Table 3.4.6.

Table 3.4.5: Maximum Allowable Lighting Power Density for Different Occupancies

Occupancy		Maximum LPD (W/m^2)
E1 and E2	Offices	9
F1 and F2	Retail/Mercantile	13
A5	Hotels	9
D1	Hospitals	11
A1, A2 and A3	Apartments/residences	7
B	Educational	11
All occupancies	Covered parking*	3
All occupancies	Open/outdoor parking	1.6

* LPD for car parks shall calculated from the total lighting power divided by the total car park area

Table 3.4.6: Applicability of Occupancy Sensors

Occupancy		Applicability
E1 and E2	Offices	Meeting rooms and corridors
A5	Hotels	Meeting rooms and corridors
A3	Apartments	Covered car parks and corridors
B	Educational	Covered car parks and corridors

4.5.3.2 For car parks a minimum 2/3rd of the lighting shall be controlled by occupancy sensors.

4.5.3.3 Emergency lighting shall not be connected to occupancy sensors.

4.5.4 Ceiling/ Wall Mounted Fans

4.5.4.1 For naturally ventilated buildings of occupancy A, ceiling/wall mounted fans shall be provided in each regularly occupied space.

4.5.4.2 For buildings of occupancy B, C, D, E and I, ceiling/wall mounted fans shall be provided in each room larger than 25 m^2, with a minimum of one fan every 25 m^2.

Exceptions:

(a) Corridors of buildings of all occupancies

(b) ICU, CCU, operating theatres of Hospitals and Clinics

4.5.5 Lift and Escalator Efficiencies

4.5.5.1 Escalators, in buildings of all occupancies, shall be fitted with controls to reduce speed or to stop when no traffic is detected.

4.5.5.2 Such escalators shall be designed with one of the energy saving features as described in i or ii below:

(i) Reduced speed control: The escalator shall change to a slower speed when no activity has been detected for a period of a maximum of three (3) minutes. Detection shall be by photocell activation at the top and bottom landing areas.

(ii) Use on demand: The escalator shall shut down when no activity has been detected for a period of a maximum of fifteen (15) minutes, designed with energy efficient soft start technology. The escalator shall start automatically when required; activation shall be by photocells installed in the top and bottom landing areas.

4.5.5.3 Elevators (lift) in buildings of occupancy A5, D1, E1, E2, F1, F2, I1 and I3 occupancies shall be provided with controls to reduce the energy demand, using the following features in traction drive elevators:

(a) AC Variable-Voltage and Variable-Frequency (VVVF) drives on non-hydraulic elevators.

(b) An average lamp efficacy, across all fittings in the lift car, of >55 lamp lumens/circuit watt, with provision for switching off, when lift is inactive for a period of a maximum of five (5) minutes.

(c) The provision to operate in stand-by condition during off-peak periods, when the lift has been inactive for a period of a maximum of five (5) minutes.

4.5.6 Renewable Energy Options

4.5.6.1 Buildings of occupancy A shall use Solar or other renewable sources of energy to power 3% of the total electric load of the building, applicable to the uses in Sec 4.5.6.3.

4.5.6.2 Buildings of all occupancies other than A, shall use Solar or other renewable sources of energy to power 5% of the lighting and fan loads of the entire building, mandatory to uses in Sec 4.5.6.3.

4.5.6.3 For all occupancies, the solar or other renewable energy connection shall power spaces in the following order of priority: lighting in underground/basement spaces, dark corridors, supplementary lighting, fans, emergency lighting like fire stairs, emergency signage egress path lighting, etc.

4.5.7 Heating Ventilation and Air-conditioning (HVAC) System

For conditioned buildings, any Heating Ventilation and Air conditioning (HVAC) system planned for installation will meet energy efficiency standards specified in Part 8 of this Code.

4.6 Internal Water Management

4.6.1 Reuse of Grey Water

Buildings of occupancy A5, E1 and E2 and I shall reuse grey water for water efficiency and management.

Grey water from wash basin shall be reused in toilet flushing and/or irrigation after filtration to ensure a BOD (Biochemical Oxygen Demand) level <50. Such water shall not be considered potable.

4.6.2 Efficient Fittings in Toilets

Water efficient fittings, including faucets, showerheads and flushes, that use less water for the same function as effectively as standard models, shall be used in buildings of all occupancies. The low flow fixtures shown in Table 3.4.7 shall be used.

Table 3.4.7: Fixture Ratings

Type of Fixtures	Quantity (max)	Unit
Water closets	Dual Flush (6/4)	liters/flushing cycle (full/low)
Shower	9.5	liters/min at 551 kPa
Urinals	3	liters/flushing cycle
Hand wash taps	6	liters/min at 417.7 kPa
Kitchen/pantry faucets	6	liters/min at 417.7 kPa

4.6.3 Service Hot Water and Pumping

In order to reduce the energy used for water heating, buildings of occupancy A5 and D1 shall use solar hot water system to supply a minimum of thirty (30) percent of the total building hot water requirements. The solar hot water system can be flat plate solar collectors or vacuum tube solar system, this system must be designed and installed with the backup system or as a per heating for the main hot water system.

PART III
Appendix A
Planning and Development Control

A.1 Scope

In absence of planning and zoning code in the national level, this Appendix states certain planning guidelines for development control and for environmental and human safety. The guidelines formulated in this Appendix are suggestive.

A.2 Land Use Classification

Land is a finite resource. An integrated and hierarchical planning from national to local level of this resource is essential to identify and use its potential in compliance with the National Land Use Policy.

Every city, township, municipality or other settlements shall have planning and zoning regulations administered by the authority having jurisdiction to guide the existing and future developments of that settlement in compliance with the National Land Use Policy and with local or regional master planning. Any land-use planning shall clearly classify the following uses:

(a) Permitted land use

(b) Conditionally permitted land-use and

(c) Restricted land use

Any such plan shall also include the preservation of open spaces and water bodies as part of the land use planning.

A.3 General Guidelines For Residential Density Planning

A.3.1 Control of residential density is a fundamental component of effective land use planning, as the relative distribution of population has major implications for all other provisions. In determining Residential density, a coherent view should be considered to achieve integrated land-use, transport, environmental and infrastructural planning.

A.3.2 Along with this integrated approach, the following factors shall also guide the residential density:

(a) A hierarchy of residential densities should be maintained to ensure diversity of housing needs.

(b) Residential density of an area shall correspond to the capacity of existing and planned infrastructure and environmental features of that area.

(c) Densities should be planned in such a way that encourage public transport and reduce travel demand.

(d) For large cities higher densities around stations and interchanges of rail based public transport system may be encouraged to reduce reliance on road based public transport system.

(e) Since higher density residential development near high capacity transportation system creates pressure on urban land use planning, careful environmental planning with definitive environmental objectives shall be there for such instances.

(f) To protect environmentally sensitive areas or areas of historical importance, a low density residential development should be proposed where and as necessary.

(g) In heavily built up areas such as old towns, provision of Transfer of Development Rights (TDR) could be implemented to control over-crowding.

A.3.3 The maximum allowable density of a residential development shall be guided by the Planning Guidelines for a zone or area or locality or township which shall be prepared under and administered by the authority having jurisdiction. Floor Area Ratio (FAR) for any development shall comply with any such planning guidelines. Where such density guideline is not available, the maximum allowable density for a residential development shall be 175 units per hectare.

A.4 General Planning Guidelines For Open Space Requirements

In a high density context like ours, pressure on land is extreme. Preserving open spaces and maintaining environmental balance become a priority in this context. In any planning process, open space must be planned as a land use in its own right and not as the remainder after providing other land uses. The hierarchy of Recreational open spaces in planning settlements may be as following:

(a) Local Open Space

(b) Ward Open Space

(c) Regional Open Space

Authorities having jurisdiction shall determine the extent of the terms ‘local’, ‘ward’ and ‘region’ on the basis of governance structure and the master planning for any particular area. However, in developing any area layout as specified in Sec A.5.2 of this Appendix, provisions for Local open space should be applicable. All open spaces in this hierarchy should act as a connected component of an open space network. The following paragraphs indicate few guidelines on location and space requirements for such open spaces.

A.4.1 Every locality should have Local Open Space which is required to meet primarily the passive recreational needs (e.g. outdoor sitting, jogging/ walking tracks, playgrounds for children etc.) of the population. Such spaces should be located either within the residential neighborhood or somewhere centrally to serve a wider area of more than one neighborhood. Considering the projected future population of a locality, a minimum requirement of 1 m^2 of Local open space per locality occupant should be allotted. Where possible such open spaces should be at least 500 m^2.

A.4.2 Every ward in a city or town or union should have Ward Open Space which is required to meet the active recreational needs (e.g. standard facilities for sports) and passive recreational needs of the ward population. Considering the projected future population of a Ward, a minimum requirement of 1 m^2 Ward Open Space per Ward occupant should be allotted. Where possible such open spaces should be at least 10,000 m^2 (1 hectare).

A.4.3 Every region should have Regional Open Space which is required to meet wider recreational needs of the population that cannot be served by local or ward open spaces. In metropolitan areas, Ward Open Space may supplement for 50 percent of the Regional Open Space requirement. Conscious planning effort are needed to create/ designate and/ or preserve 'Regional Open Spaces' located close to major public transport routes, taking advantage of natural landscape, waterfront, hill views, forest areas and/or views to special features that may draw visitors from all around. Where possible such open spaces should be at least 50,000 m^2 (5 hectare).

A.5 Community Open Space And Amenities

Every plot as specified in Sections A.5.1 and A.5.2 shall have community open space which is required to meet primarily the passive recreational needs (e.g. outdoor sitting, jogging/walking tracks, playgrounds for children etc.) of the population. Such spaces shall be located within the plot boundary. Considering the types of development, the following minimum requirements of community open space and amenities shall be applicable as per guideline of Sections A.5.1 and A.5.2 of this Appendix.

A.5.1 Community Space for a Single Tall Building or Group of Buildings In a Plot

For all residential or residential-cum-business buildings having ten or more storeys, or for all plots on which more than one residential or residential-cum-business buildings are constructed, community built space at the rate of 5 percent of the total floor area shall be provided within the building for use of the occupants of the building solely. Roofs of such buildings shall not be considered as community open spaces.

For residential or residential-cum-business plots measuring more than 0.1 hectare, 10 percent of the area of land or 1 m^2 per occupant of the plot, whichever is larger, shall be left vacant to be used as children's playground. This playground shall be contiguous and shall have a length not exceeding 2.5 times its width. The playground may extend into the mandatory open space of the plot.

Open space and amenities for residential or residential-cum-business plots measuring more than 0.4 hectare shall be as per guidelines of Sec A.5.2 of this Chapter.

A.5.2 Community Open Space and Amenities in Area Layouts

(a) Residential or Business Areas

In dividing any land measuring a total of 0.4 hectares or more into residential or business plots, community open spaces and amenities shall be reserved for recreational, educational, health care and other purposes depending on the size of the population for which the layout is planned. For planning such open spaces and amenities the guidelines of Sec A.4 of this Appendix and Sec B.3 of Appendix B shall be applicable.

(b) Industrial Areas

In dividing any land measuring a total of 1 hectare or more into industrial plots, 5 percent of the total land area shall be reserved as amenity open space which shall be used as lawn, park or garden. The minimum size of such open space shall be 600 m^2. When the area of the open space exceeds 1000 m^2, the area of land in excess of 1000 m^2 can be used for the construction of buildings for banks, clinics, welfare centers and other common facilities for use of the persons working in the industries.

A.6 Plot Size

Plot divisions and plot sizes are part of integrated planning decision of detail area plan and shall be determined by the Area Development Authority having jurisdiction. Where no such guideline exists or yet to be undertaken, the following criteria mentioned in Sections A.6.1 to A.6.8 regarding plot size shall be applicable.

A.6.1 Residential Plots

(a) For any future development, the minimum size of the plot for Occupancy A1, A2 and A3 shall be 66 m^2, 133 m^2 and 200 m^2 respectively.

The sizes of the plots and the corresponding minimum widths of road frontage of the plots shall be as specified in Table 3.A.1 provided that:

(i) Plots accessible by link roads shall be considered to have a frontage equal to the width of the link road, and

(ii) Plots of irregular shape abutting the road shall be considered to have a frontage equal to their average width parallel to the road.

(b) The limitations of plot sizes and frontages imposed in Sec A.6.1 (a) above may be waived for approved affordable housing including site and service schemes. Guidelines governing the planning and design of such housing are given in Appendix B.

(c) The minimum size of the plot for a group housing development scheme and other special requirements for group housing developments shall be as specified or approved by the respective city development authority.

(d) For minimum standard transitional housing, government may allow smaller plots following the guidelines mentioned in Appendix B.

Table 3.A.1: Minimum Frontage of Residential Plot

Type of Residential Development	Plot Size (m^2)	Minimum Frontage (m)
Approved row type houses	66 (Minimum size)	4.5
	Over 66 to below 133	8
Semi-detached houses	133 (Minimum size)	8
	Over 133 to 200	8-10
	Over 200 to 267	10-12
	Over 267 to 334	12
Detached	Over 267 to 334	12
	Over 335 to 669	16
	Over 669	24

Note: For plot sizes larger than 133 m^2 detached house type may be allowed provided that the site frontage is 12 m or more

A.6.2 Plots for Educational Buildings

The minimum size of plot for educational buildings shall be based on occupant capacity and shall be at the rate of 4 m^2 per pupil or occupant. With exception for nursery school the minimum plot size required for educational purpose shall be 3950 m^2 (see Appendix B).

A.6.3 Plots for Assembly Halls, Theatres, Cinema

The minimum size of plot for assembly halls, theatres, cinema halls and other similar buildings where people gather for entertainment or other public functions shall be based on the seating capacity of the building and shall be at the rate of 3 m^2 per seat. Table 3.A.2 shows the minimum plot size for such function:

Table 3.A.2: Plot Sizes for Assembly Occupancy

Sub-Category	Nature of Use or Occupancy	Minimum Plot Size
I1	Large assembly with fixed seats (1000 seats or more)	3000 m^2
I2	Small assembly with fixed seats (less than 1000 seats)	3 m^2 per seated person
I3	Large assembly without fixed seats (300 or more occupants)	900 m^2
I4	Small assembly without fixed seats (less than 300 seats)	3 m^2 per seated person
I5	Sports facilities	Related to event and spectator capacity

A.6.4 Plots for Community Centers

The size of plot for rural or urban community centers shall be not less than 1300 m^2 and commensurate with the size of the community.

A.6.5 Business and Mercantile Plots

The minimum size of a business and mercantile plot shall be 200 m^2 and its road frontage width shall not be less than 10 m.

A.6.6 Industrial Plots

The minimum size of an industrial plot shall be 300 m^2 and its road frontage width shall not be less than 15 m.

A.6.7 Petrol Filling Stations

The minimum size of the plot for a petrol filling station without service bay or repair workshop shall be 500 m^2 and its road frontage width shall not be less than 30 m. The minimum size of the plot for a petrol filling station with service bay but without repair workshop shall be 1100 m^2 and its road frontage width shall not be less than 30 m.

A.6.8 Plots for Other Uses

The minimum sizes of plots for buildings for uses other than those mentioned in Sections A.6.1 to A.6.7 shall be as determined by the Authority having jurisdiction.

PART III
Appendix B
Guidelines for Minimum Standard Housing Development

B.1 General

B.1.1 Government bodies or public agencies may designate an area in the master plan for the development of Mass Housing Projects. Generally all such development shall be known as Minimum Standard Housing, which shall be broadly categorized into the following two categories: a) Minimum Standard Community Housing and b) Minimum Standard Transitional Housing. Requirements for master plan and dwelling units in such projects shall have special provisions depending upon the category of development mentioned above.

The guidelines of this Appendix cover the planning and the general building requirements of all such Minimum Standard Housing developments.

B.1.2 A minimum standard community housing is a housing that confirms to the basic minimum requirement regarding dwelling units, community and other facilities according to the provision of this Code. Requirements for community facilities in these housings shall depend upon the size and scale of the community (Sec B.4.2). Dwelling units in all such developments shall be classified as Occupancy A1, A2, A3 or Mixed Occupancy depending upon the type of development (Sec B.3) and degree of mixing with other occupancies within the same structure.

B.1.3 Minimum Standard Transitional Housings are housing facilities on a transient basis before providing its inhabitants with Minimum Standard Community Housing. Ensuring safety, health and sanitation requirements shall be the primary obligation for such housing. Since, it may not be convenient or practicable for the planning and dwelling units in such projects to be in full compliance with all the requirements of this Code, a few exemptions have been made for all such housings in the following Sections of this Appendix. All Transitional Housing, irrespective of its type of development shall be group housing. Dwelling units in a transitional housing shall be classified as Occupancy A3, A4 or Mixed Occupancy depending upon the type of development and degree of mixing with other occupancies within the same structure.

B.1.4 Only government bodies or public agencies should be responsible for planning the number and location of the settlements in an approved master plan following the density guidelines of Appendix A and the layout of units within the settlement. The guidelines of this Appendix regarding layout planning are applicable to government bodies or public agencies responsible for such planning.

B.1.5 The guidelines and requirements regarding design and construction of buildings for minimum standard housing in approved layouts are applicable to all government bodies, public agencies, private developers or individual owners who undertake such constructions.

B.2 Types Of Development

The development of minimum standard housing may be any one or a combination of the following types:

(a) Single unit plots of row type housing

(b) Multi-storied flats of row type housing

(c) Block development as a group housing

(d) Cluster housing, and

(e) Site and service schemes

The guidelines for planning and general building requirements shall be applicable to all types of development of minimum standard housing unless exempted as mentioned in this Appendix.

B.2.1 Single Unit Plots of Row Type Housing

Row Type in a housing development is characterized by independent plotted developments for single family dwelling (A1) and shared community facilities. Along with single family detached and semi-detached dwelling, this type of dwelling is used to achieve low-densities in a settlement.

This type of development should be located away from high capacity public transportation system and should be characterized by high car ownership rate.

All such development shall comply with the provisions for parti-wall and general criteria for natural lighting and ventilation as described in Part 3 Chapter 1, Part 6 and Appendix E.

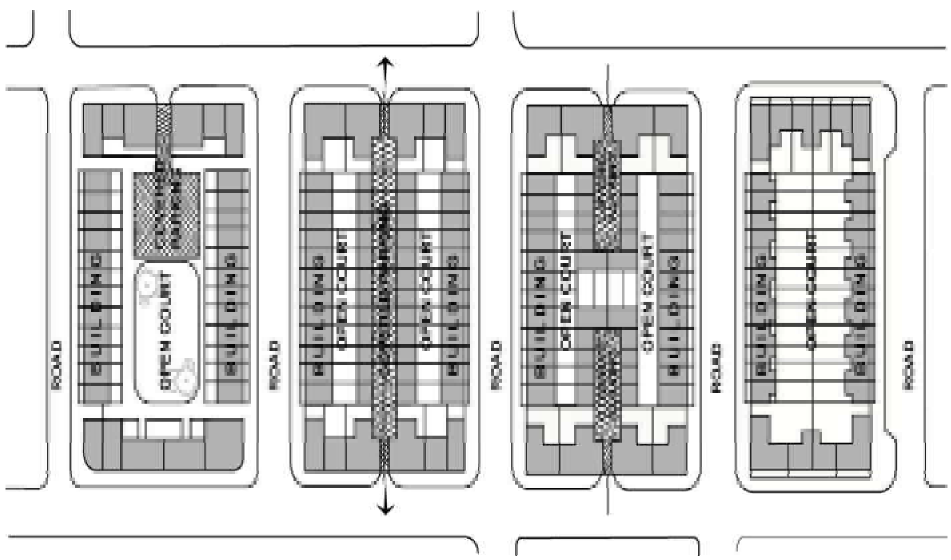


Figure 3.B.1 Row type development

B.2.2 Multi-Storied Flats of Row Type Housing

Unlike single unit plots of row type development, multistoried flats of row type housing are not plotted development. Here the un-demarcated un-divided land is shared by the flat owners within such complex.

Houses or flats or apartments in Multi-storied Row type developments shall share walls with adjacent flat or apartment, provided that they are designed and built at the same period as part of the same project and also that they fulfil the general criteria for natural lighting and ventilation as referred in Part 3, Chapter 1.

Multi-storied row type developments should be permitted for walkup apartments (A3) as well as two family dwelling (A2) with many of the community facilities at lower levels. Such developments are good for medium density settlements and may also be used in mixed use zones.

B.2.3 Block Development as a Group Housing

Block developments are characterized by high-density large scale developments within a block usually surrounded by roads all around and serviced by high capacity transport and utility network.

Large plots of 4048 m^2 or more, with road access on one side may also be considered as block development if provided with a peripheral access road along its perimeter and also if they are in close proximity to high capacity transport and utility infrastructure network. All such plots may be allowed to attain higher densities than other typologies. Such developments are more desirable in the central urban areas emphasizing more dependency on public transportation and less on private transportation. Parking requirements in such typologies shall be less than other typologies.

Block development for Group Housing should be permitted for large scale developments of Apartments (A3) or Mess, Boarding houses, Dormitories and Hostel (A4).

All such residential developments can be mixed with other permitted occupancies to encourage availability of all services within the close proximity and to support growth of local economy. All such developments can be subdivided into following two categories:

(a) Tower Block Development for Group Housing

This includes Flats or Apartments (A3) in a single or multiple high-rise towers within a block/large site with relatively low ground coverage and higher density along with open spaces and community facilities within and around the tower blocks.

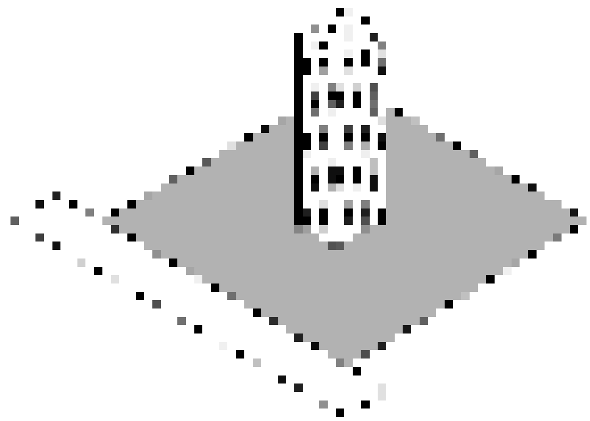


Figure 3.B.2 Tower block development

(b) Perimeter Block Development for Group Housing

Unlike Tower blocks, Perimeter block developments involves one single building or multiple buildings placed along the perimeter of the site to create an internal open space shielded from its surrounding, commonly used as community space. Each housing unit in such developments shall have at least two sides open—one at the internal open space and the other at the external road side.

Depending upon the plot size and population such development may also be permitted to become high-rise development, provided that the internal open space in such development confirms to the criteria for minimum requirements of a courtyard (Part 3, Chapter 1).

B.2.4 Cluster Housing

Cluster type development, as a housing form, is suitable for accommodating low to high density of population within walkup range. Cluster type development for Group Housing should be permitted for all ranges such as for low density development for single family dwelling (A1) and two family dwelling (A2) or for moderate to high density developments of Apartments (A3) or Mess, Boarding houses, Dormitories and Hostel (A4). Details of this typology have been discussed in Appendix D.

B.2.5 Site and Service Schemes

Site and service schemes shall delineate individual plots and provide for the infrastructural needs for the development of a permanent housing. Interim constructions by the allottees should also be permitted. Skeletal structures with a roof on columns and/or developed plinths may be provided if funds are available.

Sanitation and water supply must be provided in all site and service schemes. A sanitary service core or common water supply and sanitation facilities for planned groups of plots should normally suffice. The developing agency shall install the services before handing over the plots.

Site and service schemes for group housing should be permitted for low density development of single family dwelling (A1) and two family dwelling (A2) and ownership right of such housing shall be non-transferrable.

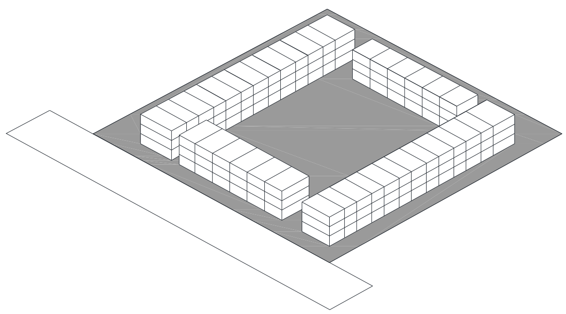


Figure 3.B.3 Perimeter block development

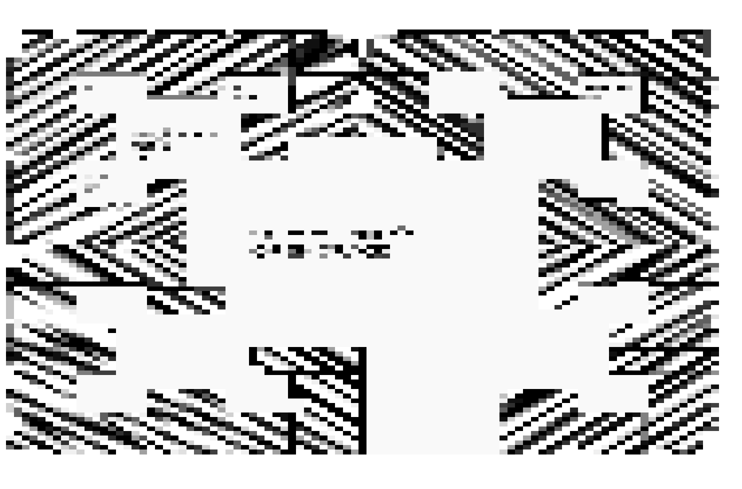


Figure 3.B.4 Cluster type development

B.3 Planning

B.3.1 A minimum standard housing shall ensure quality living for all its inhabitants. Providing proper environmental quality, social and utility infrastructure, educational facilities, health care and recreational facilities, connectivity to commerce and job locations in a comprehensive planning are pre-conditions to attain that. With increasing urban population and the shortage of developable land, high residential densities shall occur. This multiplies the necessity for the provision of community facilities to ensure the minimum standard of the housing.

B.3.2 Density

Housing density for an area or locality or settlement is a planning issue and shall be decided by the planning authority in accordance with the Detail Area Plan (DAP). Where such guidelines are unavailable, the gross density of a minimum standard housing shall not be more than 175 units per hectare, considering an average population of 5 persons per dwelling unit. The general distribution of built area and open space shall follow the provisions of Sec B.3.3.

B.3.3 Basic Requirements for Community Facilities and Facilities for Locality

Any minimum standard housing, irrespective of its type of development, should be planned and organized in groups or clusters, where each of the clusters should not exceed 400 dwelling units with an average population of 5 residents per dwelling. Where the number of dwellings is more than that, more than one clusters, each below 400 units, should be formed.

In any housing project, community and facilities for locality are essential. The facilities required for any such project shall depend on the size of the community or communities within a locality. Table 3.B.1 and Table 3.B.2 show the requirements of such facilities for any housing development in reference to their variation of net density and their relation to the threshold population.

B.3.3.1 Open space within a site area

Open space within a site area shall be as defined in Sec 1.8 Chapter 1, Part 3. However for a minimum standard housing, such open space shall also be equal or more than the open to sky space which is an outcome of addition of

(a) Mandatory setback area and

(b) Area requirement for Community Open Space (COS) as per Sec B.3.3.2.

For plots below 4048 m² community open space may overlap with mandatory setback area.

B.3.3.2 Community open space shall not be less than 1 m^2 per occupant.

B.3.3.3 Community facilities consist of two components :

(a) Community Open Space (COS) : Community open space is an undivided contiguous open space within a plot or block or cluster which along with mandatory setback area constitutes the open space of the site. Depending upon the size of population and land area this space may be used as any or combination of the following uses—

(i) Lawn

(ii) Garden

(iii) Play lot

All such uses shall be exclusive to the residents of that plot or block or cluster.

No paved area, other than the minimum required area for vehicular and pedestrian access to the site, is allowed within this community open space. All such road and walkways shall ensure pedestrian priority and safety, and shall not be used as a through circulation.

The following guidelines shall be maintained if the area of the community open space within the site fulfils the criteria mentioned in the first column of Table 3.B.1.

(b) Community Built Space (CBS) : Community built space is part of the community facilities dedicated to serve the population within a given site or block or cluster. Depending upon the threshold population, the CBS shall include facilities as per guideline of Table 3.B.2 for a threshold population up to 200 families.

Table 3.B.1: Guideline for Minimum Width for Community Open Space

Open Space Area (m^2)	Minimum Width (m)	May Accommodate
130 or more	9	Badminton, table tennis etc.
700 or more	21	Basketball, badminton etc.

B.3.3.4 Facilities for Locality (FL) : Facilities for localities are shared facilities consisting of both Local Open Spaces (LOS) and Local Built Spaces (LBS) located outside the plot boundary but within the locality. The total space requirement of the Facilities for locality shall vary considerably depending upon the requirements of net density, gross density and threshold population of Table 3.B.2.

Table 3.B.2 refers to list of such facilities for a plot/locality/ward/region in relation to population size and space requirement for a settlement.

Table 3.B.2: Threshold Population and Minimum Reference Standard of Facilities for Community and Locality

Facilities	Threshold Population to Start the Facilities	Minimum Reference Standard	Level Served
General*			
Community Open Space (COS)	3 families	1 m² per person	Plot
Management office	10 families	10 m² room	Plot
Meeting room/hall (multi-purpose with storage and washroom facilities)	20 families	2 m²/ family	Plot
Indoor games room	30 families	1 m² per family	Plot
Prayer hall	40 families	0.75 m² per family	Plot
Educational			
Day care center**	100-300 families/500-1500 persons	Minimum 6 children	Plot/ local
Nursery/kindergarten school (age group 3 to under 6 years)**	400-600 families/2000-3000 persons	Minimum 6 classrooms	Local
Primary school (age group 7 to under 11 years)**	1000-1600 families/ 5000-8000 persons	Minimum 3950 m² site area with minimum site width of 55m	Local
Secondary school (age group 12 to under 17 years)	1800-2400 families/ 9000-12000 persons	Minimum 6950 m² site area with minimum site width of 65m	Ward
Colleges/community colleges/vocational colleges	10,000 families/ 50000 persons	2000 m²-7000m² site area	Regional

Facilities	Threshold Population to Start the Facilities	Minimum Reference Standard	Level Served
Commercial			
Small store/s	50 families	4 stores per 200 families (20 m² per store, max)	Plot/ local
Medicine store/dispensary/ convenience grocery store/ bakery/beauty parlor	200 families	175 m² per 400 families minimum	Plot/ local
Super-market	2000 families/10,000 persons		Ward/ regional
Shopping center	4000 families/20,000 persons		Ward/ regional
Socio-Cultural			
Community welfare center	300 families	400 m² per 300 families	Local
Places for worship (Mosques, temples, churches etc.)	400 families	175 m² per 400 families	Local
Community Hall	1000 families	Plot size 1300 m² minimum.	Local/ ward
Library	1000 families	175 m² minimum	Ward
Healthcare			
Health center	400 families/2000 persons	175 m² per 400 families	Ward
Small clinic	2000 families/10,000 persons	Minimum 2200 m² plot per 10,000 persons	Ward
Services and Utilities			
Internal roads and walkways	Any	15 to 20 per cent of the site area	
Amenities (Garbage disposal, water pump, local electrical substation/generator etc.)	Any	5 per cent of the site area	Local

Facilities	Threshold Population to Start the Facilities	Minimum Reference Standard	Level Served
Office of local authority, community police etc.	400 families/ 2000 persons	175 m² per 400 families	Ward
Public transport stoppage/ station***	As per planning guideline	At least 1 bus bay with passenger shed	
Sports and Recreational			
Park, water front	2000 families/10,000 persons	1 hectare minimum per 10,000 persons	Ward/regional
Play ground	1000 families/ 5,000 persons	400 m² minimum (local level) per 5000 persons	Local/ward
Children and youth center	2400 families/12,000 persons	630 m² minimum per 12000 persons	Local/ward
Gymnasium, Indoor games	1000 families/5000 persons	175 m² minimum per 5000 persons	Local/ward
Swimming pool	6000 families/30,000 persons	420 m² of water surface/ 30,000 persons	Ward
Sports Complex with indoor and outdoor facilities	10,000families/ 50,000 persons	0.6 hectare (ward level) per 50000 persons	Ward

* All general facilities shall be accommodated within the plot or block or cluster area. For any housing having a population of 10 families or above, the total built area (excluding community open space) dedicated to such facility/ facilities (depending upon the population size) shall not be less than 1m² per person.

** All such facilities must be a part of integrated planning prioritizing close proximity to the housing units they serve. The distance of such facilities from the serving units should not be more than 0.4 kilometers, All such connecting pathways, street etc. shall be away from major roads and shall have pedestrian priority.

*** Public transport stoppages should be located nearer to health facilities, post-secondary educational institutions and other public buildings and should be supported by public parking facilities nearby.

B.3.4 Size of Plot

B.3.4.1 Minimum standard community housing

All minimum standard community housing shall follow the general guidelines of plot sizes mentioned in Appendix A.

B.3.4.2 Minimum standard transitional housing

All types of minimum standard transitional housing, except site and service type, shall be group housing (A3 or A4 occupancy) and shall abide by the plot size requirement of Appendix A. For Site and Service transitional housing, the authority may allow higher density for larger plot size with close proximity to public transport network provided that such decisions are in compliance with the planning guidelines for the area or locality.

Site and Service Scheme for Single Room Development : For site and service transitional housing development where a minimum standard house with single room, kitchen, bathing facilities and water closet are expected to develop, the minimum plot size may be reduced to 30 m^2 with a minimum frontage of 4.1 m. In areas other than metropolitan cities; with population less than 0.5 million, the minimum size of plot for such houses should be 40 m^2 with a plot frontage of minimum 4.8 m. In dense inner city areas of metropolitan cities with population more than 1.5 million, the Government may decide to have a minimum plot size of 25 m^2 with a minimum frontage of 4.1 m for such housing.

Site and Service Scheme for Two Room Development : For site and service transitional housing development where a minimum standard house with two rooms, kitchen, bathing facility and water closet are expected to develop, a minimum plot size of 40 m^2 shall be required. In areas other than metropolitan cities, having a population less than 0.5 million, the minimum size of the plot for such houses should be 60 m^2.

B.3.5 Internal Roads and Walkways

Pedestrian walkways when provided as means of access shall be at least 3 m wide. Such walkways shall not be longer than 60 m, nor serve more than 10 plots on each side of the path. Other internal roads shall be at least 6 m wide to allow emergency vehicles to enter. The paved portion of such roads, if used for pedestrian movement only, should be at least 2 m wide.

B.4 General Building Requirements

B.4.1 Plinth Coverage

The plinth area coverage of any plot of minimum standard housing shall not exceed 65 percent of the plot area. Plots with higher net density shall have lower plinth area and ground coverage.

Exception: For minimum standard transitional housing

The plinth area coverage of any plot of transitional housing shall not exceed 75 percent of the plot area. There shall be a setback of minimum 1.5 m on the rear side of a plot. There is no requirement for such set back on the sides and front of a plot if facing an internal road.

B.4.2 Height Limitation

The height limitation in such housing will vary according to typology of each development. However, maximum height for minimum standard housing for site and service scheme shall be 10 m. Minimum standard transitional housing of cluster type development and multistoried Row type development shall have a height limitation of maximum 20 m or 6 storied.

B.4.3 Plinth Level

The minimum height of the plinth shall be 300 mm from the surrounding ground level.

B.4.4 Habitable Room

B.4.4.1 All dimension stated in this Section do not include area or dimension required for partition or enclosure wall. Criteria for habitable room shall follow the basic guidelines of Part 3 Chapter 1.

B.4.4.2 Overcrowding

To avoid overcrowding in all habitable rooms in minimum standard housing including transitional housing, a minimum air volume of 9.5 m^3 (9.5 cubic meters) per occupant shall be allotted. To calculate such volume no account shall be taken of any space with a height higher than 4.25 m or less than 2.15 m from the floor level of the room.

Exception: For transitional housing

(a) In a flatted development, one roomed dwelling units shall have a multi-purpose room which may include an alcove or space for cooking (as specified in Sec B.4.5). The minimum area of the room shall be 12 m^2 with a minimum width of 2.5 m.

(b) For dwelling units with two habitable rooms, the minimum size of at least one room shall be 9.5 m^2 with a minimum width of 2.5 m. Other habitable room in the dwelling unit shall have a minimum area of 5 m^2 with a minimum width of 2 m.

(c) One-roomed dwelling with plan for future extension into a two-roomed house in a staged construction scheme shall satisfy the requirement of (a) and (b) above regarding room sizes. The first room to be built in this type of development shall have a minimum area of 9.5 m^2 with a minimum width of 2.5 m. The total area of the two rooms after future extension shall be a minimum of 15 m^2.

(d) All habitable rooms shall have a minimum clear height of 2.75 m. For sloped roofs, the average height shall not be less than 2.75 m with a minimum of 2 m at the lowest side.

B.4.5 Kitchen

B.4.5.1 Criteria for kitchen shall follow the basic guidelines of Part 3 Chapter 1.

B.4.5.2 Exception: For minimum standard transitional housing

(a) The size of the cooking alcove or cooking space provided in a multi-purpose room of a one-roomed house shall not be less than 2.25 m^2 with a minimum width of 1.2 m.

(b) Separate kitchen provided in a two-roomed house shall have a minimum area of 3.25 m^2 with a minimum width of 1.6 m.

(c) Minimum clear height of the kitchen or cooking space shall be 2.15 m.

B.4.6 Bathroom and Water Closet

B.4.6.1 Independent water closets shall have a minimum width of 0.9 m and a minimum length of 1.15 m. The water closet shall be fitted with a door.

B.4.6.2 Independent bathroom without water closet shall have a minimum width of 1 m and a minimum length of 1.4 m.

B.4.6.3 The minimum size of a combined bathroom and water closet shall be 1.8 m^2 with a minimum width of 1 m. The bathroom shall be fitted with a door.

B.4.6.4 The minimum clear height of bathrooms and water closets shall be 2.15 m.

B.4.7 Balcony and Corridor

B.4.7.1 The minimum width of individual balcony shall be 0.9 m. Corridors for use of more than one dwelling units shall have a minimum width of 1.2 m.

B.4.8 Stairs

B.4.8.1 Minimum Width: Criteria for minimum width of stairs shall follow basic guidelines of Chapter 1 Part 3.

B.4.8.2 Maximum Rise: Criteria for maximum riser shall follow the basic guidelines of Chapter 1 Part 3.

B.4.8.3 Minimum Tread Depth: Criteria for minimum tread depth shall follow the basic guidelines of Chapter 1 Part 3.

B.4.8.4 Minimum Head Room: Criteria for minimum clearance of head room shall follow the basic guidelines of Chapter 1 Part 3.

B.4.8.5 Landing: Criteria for minimum landing depth shall follow the basic guidelines of Chapter 1 Part 3.

B.4.9 Water Supply

One water tap or hand tube-well pump per dwelling unit should be provided, if feasible. Each unit of public water hydrants or community hand pumps, if provided in lieu of individual water supply, shall serve not more than 10 dwelling units and shall not be farther than 15 m from any dwelling unit served.

B.4.10 Lighting and Ventilation

Every room, bathroom and kitchen shall have windows in an external wall opening on a courtyard, a balcony not wider than 2.5 m, or the exterior. The aggregate area of openings in the exterior wall of a habitable room or kitchen shall not be less than 12 percent of the floor area and that for a non-habitable room such as bath room, water closet or stair shall be at least 8 percent of the floor area.

PART III
Appendix C
Special Requirements of Cluster Planning for Housing

C.1 General

These guidelines cover the planning and general building requirements of different housing developments in cluster typologies as referred in Appendix B. These requirements are applicable to all housing projects of this type taken up by public, private or co-operative agencies.

All cluster housing typologies shall fulfill the criteria regarding minimum area and width requirement of each habitable and non-habitable room as specified by this Code. The construction classification for Cluster Housing shall be of protected type.

C.2 Cluster Typology

C.2.1 Cluster Type Development

Cluster type development (Figure 3.C.1), as a housing form, is suitable for accommodating low to moderate density of population. However with smaller plot size, it can also attain high density situation as may happen for Transitional Housing.

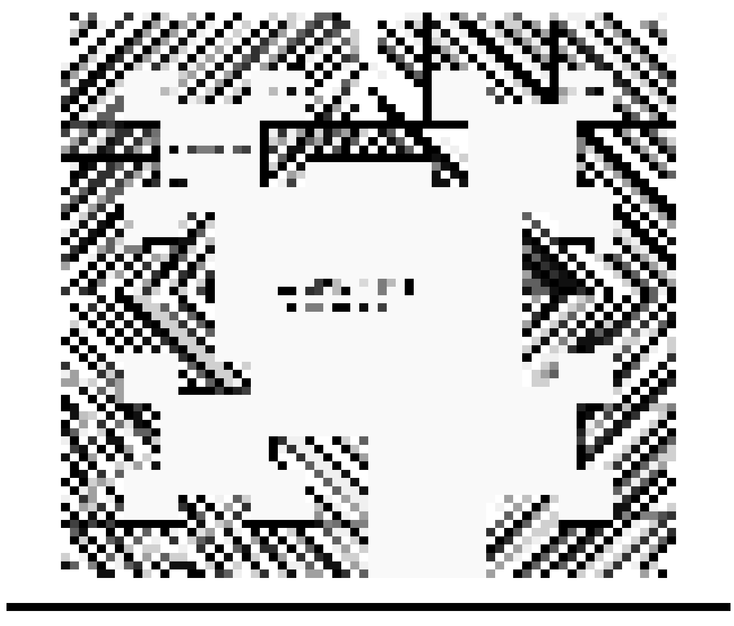


Figure 3.C.1 Cluster type Housing

C.2.2 Examples of Cluster Typologies

(a) Back-to-Back Cluster—Clusters when joined back to back and/or on sides (Figure 3.C.2).

(b) Closed Clusters—Clusters with only one common entry into cluster open space (Figure 3.C.3).

(c) ‘Cul-de-sac’ Cluster-Plots/dwelling units when located along a pedestrianized or vehicular ‘cul-de-sac’ road (Figure 3.C.4).

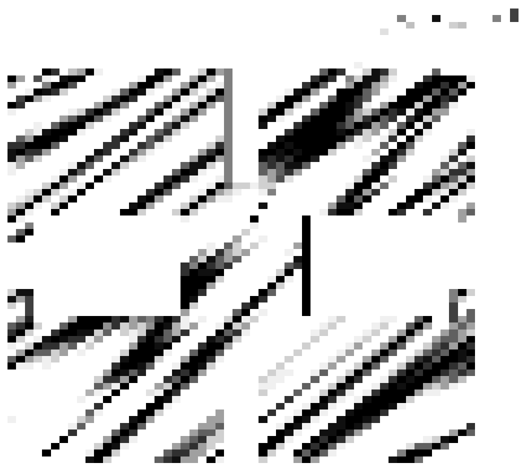


Figure 3.C.2 Back to back cluster

Figure 3.C.3 Closed cluster

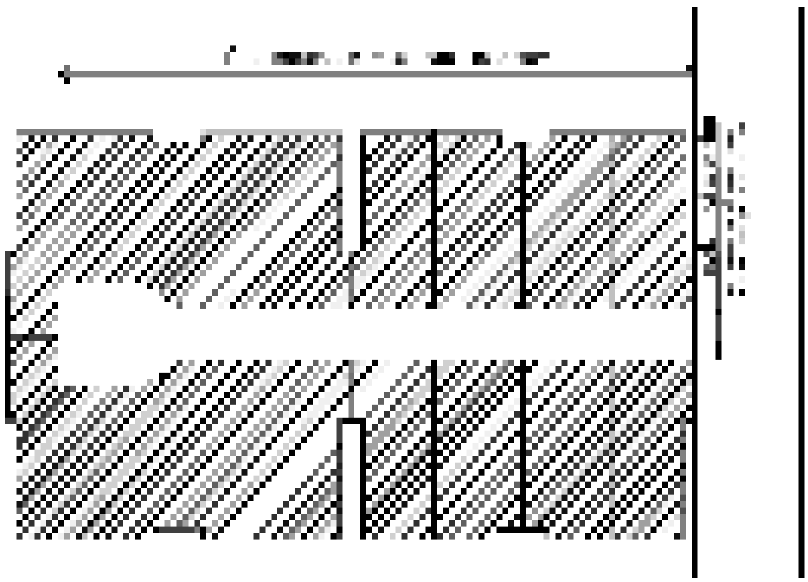


Figure 3.C.4 Cul-de-sac cluster

C.2.3 Plot Size

In an integrated cluster planning, the minimum plot size shall comply with the guidelines set in Appendix B (including transitional housing).

C.2.4 Community Facilities

All such housing shall be provided with the requirement of neighbourhood and community facilities required for the population. In such cases, the authority may allow a FAR of 2.00 with 100 percent ground coverage provided that the basic natural lighting and ventilation criteria is met through the two exterior sides, both having exposure to the adjacent neighbourhood spaces.

C.2.5 Group Housing

Group housing may be permitted in the form of cluster housing. However, dwelling units with plinth areas up to 20 m² should have scope for adding a habitable room. Group housing in a cluster should not be more than 15 m in height.

C.2.6 Size of Cluster

In single to two-storeyed structures not more than 20 houses should be grouped in a cluster. Clusters with more dwelling units may create problem relating to identity, encroachment and maintenance.

C.2.7 Size of Cluster Open Space

Minimum dimensions of width of open spaces shall be not less than 6 m or 3/4th of the height of buildings along the cluster open space, whichever is higher. The area of such cluster court shall not be less than 36 m^2. Group housing around a cluster open space should not be more than 15 m in height.

C.2.8 Setbacks

In any cluster type development at least two sides of each individual dwelling shall have exterior walls and opening. No setback is required in other two sides of such developments. However for compliance of natural lighting and ventilation with this code, light well and ventilation well may be used within a cluster plot.

C.2.9 Right to Build in Sky

Pedestrian paths and vehicular access roads to clusters separating two adjacent clusters may be bridged to provide additional dwelling units. While bridging the pedestrian path way minimum clearance should be one storey height; length of such bridging should be not more than two dwelling units. While bridging the vehicular access roads minimum clearance from ground level shall be 6 m with a vertical clearance of 5 m.

C.2.10 Vehicular Access

A right of way of at least 6 m width with a vertical clearance of 5 m shall be provided up to the entrance to the cluster to facilitate emergency vehicle movement up to cluster.

C.2.11 Pedestrian Paths

Minimum width of pedestrian paths shall be 3 m.

C.2.12 Width of Access between Two Clusters

Built area of dwelling unit within cluster shall have no setbacks from the path or road, space. Hence, the height of the building along the pathway or roads shall be not less than 60 percent of the height of the adjacent building subject to minimum of 3 m in case of pathway and 6 m in case of vehicular access.

C.2.13 Density

Transitional housing shall result in higher densities with low rise structures.

For a minimum standard housing with one habitable room, one kitchen and one two-fixture toilet and the required enclosure walls as per provision of this Code, a minimum dwelling unit of 23.81 m^2 is required. However for transitional housing, the size of dwelling units may be reduced up to 18.5 m^2 to increases density. In all such transitional housing a maximum allowable density shall follow the density guidelines of Appendix B.

C.2.14 Group Toilet for Transitional Housing

Transitional housing may have group toilets at the rate of one water closet, one bath and a washing place in three separate chambers per three families. These shall not be community toilets, as keys to these toilets shall be only with these three families, making them solely responsible for the maintenance and upkeep of these toilets.

C.3 Other Requirements

C.3.1 Requirements of Building Design

With the exception of clauses mentioned above, requirements of building will be governed by the provision of this Code and good practice. Requirements of fire safety, structural design, building services and plumbing services shall be as specified in this Code.

PART III
Appendix D
Universal Accessibility

D.1 Scope

The aim of this Appendix is to set out the fundamental design and construction requirements and guidelines for different occupancy types, accessible to persons with permanent or temporary disabilities. The requirements and guidelines should be applicable for all buildings and facilities as shown in Table 3.D.1 for emergency evacuation provisions of Part 4 shall be applicable.

D.2 Terminologies

D.2.1 Definitions

For the purpose of this Part of the Code, the definitions/terminologies below shall be applicable:

ACCESSIBLE/ ADAPTABLE WASHROOM	Refers to a compartment with a water closet, wash basin, grab bars and other essential washroom accessories and with clear floor spaces at fixtures as per provision of this Code which a wheel chair user or any other person with disability can avail with ease and safety.
ACCESSIBLE/ ADAPTABLE WATER CLOSET COMPARTMENT	Refers to a compartment with adequate maneuvering space as per provision of this Code having a single water closet with grab bars installed to assist persons with disabilities.
ACCESSIBILITY	See Part 3 Chapter 1 definition.
ACCESSIBLE RAMP	See Part 3 Chapter 1 definition.
ACCESSIBILITY ROUTE	See Part 3 Chapter 1 definition.
ADAPTABLE	See Part 3 Chapter 1 definition.
AMBULANT DISABLED	Refers to any person who, with the help of prostheses (artificial limbs)/ orthotic/ crutches/canes/ sticks or any other walking aid, is able to walk on level plain or suitably graded steps with handrails complying the provision of this Code.

CIRCULATION PATH	See ‘accessibility route’
CURB	Refers to a side barrier between a trafficable surface and adjacent area through level change.
CURB RAMP	Refers to a short ramp cutting through a curb or built on it to negotiate accessibility between levels, which may have a different gradient as per provisions of this Code than a conventional accessible ramp.
GRAB BAR	Refers to a bar of certain specification and height as per provision of this Code which is used for assisting to stabilize a person with disability for performing a particular function.
HANDRAIL AND GUARDS	See Part 4 Chapter 3.
OPERABLE PART	Refers to part or component of any equipment, appliance or fixture which is necessary to operate that equipment, appliance or fixture (for example, handle, lever, push-button etc.).
PERSONS WITH DISABILITIES	Refer to persons whose mobility and capacity to use a building or part there of or a facility are affected due to one or more physical and/or sensory disabilities or impairments. For the purpose of this Code, they will be categorized as following: (a) Wheelchair-bound (b) Ambulant disabled (c) Hearing impaired and (d) Visually impaired.
SYMBOL	Refers to the international symbol of access for persons with disabilities also known as International wheel-chair symbol.
WHEELCHAIR USER	Refers to a person with disability who is depended on a wheelchair for mobility.

D.3 Provisions For Accessibility

D.3.1 Barrier-Free Accessibility

The following building occupancies shall require barrier free accessibility for persons with disability in the areas or facilities as specified in Table 3.D.1.

Table 3.D.1: Requirements of Accessibility for Different Occupancies

Occupancy Type		Accessible Areas
Residential buildings	A3*, A5, MIXED	From public footpath and parking areas to the lift lobby, lift, from lift lobby to all housing units, at least one toilet per housing unit and all communal facilities
Hostels and dormitories	A4, MIXED	All public areas intended for access by staff, students or visitors and at least one room per every hundred rooms or portion thereof including access to public footpath and parking.
Schools, colleges, universities or other educational buildings	B1-B3, MIXED	All areas intended for access by staff, students or public use including access to public footpath and parking.
Hospitals, clinics, homes for the aged and Institutions for the physically challenged	D1, D2, C3	All areas intended for access by staff, patients, inmates or public use.
Office buildings	E1, E3, MIXED	All areas intended for access by employees or public including parking and at least one accessible toilet facility in each floor
Small shops and markets, Kitchen markets	F1, F3, MIXED	From parking and/or public footpath to sales counter service
Large shops and markets	F2, MIXED	All areas intended for access by employees or public including access to public footpath and parking.
Factories, workshops, industrial buildings and administration buildings in depots	G1, G2	All areas intended for access by employees or public use.

Occupancy Type		Accessible Areas
Religious buildings, crematoria	I1-I4, MIXED	All areas intended for access by worshippers or public including access to public footpath and parking.
Restaurants, food-courts, fast food outlets and other public eating outlets	I1-I4, MIXED	All areas intended for access by employees or public including access to public footpath and parking.
Cinemas, theatres, stadia or other places of assembly with permanent seating	I1-15, MIXED	All areas intended for access by performers and areas prescribed by this Code for spectators or public use (Sections D.3.2.3 and D.3.2.5) including access to public footpath and parking.
Sports complexes, public gymnasiums and public swimming pools	I5	All areas intended for public access with at least one accessible shower compartment and one water closet compartment.
Stations, airports, river-ports, bus terminals, interchanges and other passenger transport terminal	MIXED	All areas intended for access by employees or public use including areas prescribed in Sec D.3.2.7.
Parking garage, private garage, repair garage and showrooms	K1-K3	Prescribed areas in accordance to Sections D.24 and D.25.

Note:*Excluding apartments without lift

D.3.2 Minimum Accessible Provisions

D.3.2.1 Mess, dormitories and hostels

In residential occupancies such as mess, dormitories and hostels, at least one room in every 100 rooms or part thereof shall be accessible.

D.3.2.2 Hotels and lodging houses

In room based residential occupancies e.g. Hotels, lodging houses etc. at least one in every 200 guest-rooms or a portion thereof shall be made accessible.

D.3.2.3 Banks, ticketing offices and counter services

Public facilities with counter services, such as, banks, information desk, ticketing offices etc. at least one service counter shall be designated in accordance to the requirements of Sec D.13.

D.3.2.4 Large shops and markets, foyers and public concourses

In large shops and markets such as large departmental stores, shopping mall etc. seats shall be provided for persons with disabilities who are unable to stand or walk for long periods. There shall be at least one accessible toilet per floor connected to the accessible route within such facilities. Seats and free spaces for wheelchair users shall also be required in foyers and concourses of public buildings.

D.3.2.5 Movie theatres, theatres, stadiums or other places of fixed-seat assemblies

Assemblies with permanent fixed seats such as movie-theatres, theatres, stadium, indoor stadium etc. shall have one wheelchair space per every 150 seats or a portion thereof. Such spaces should be located at a level that is easy to access for the wheelchair users. Seat arrangements shall facilitate wheelchair users to sit with their able bodied companion together. All such assemblies shall facilitate accessible counter facilities and toilets for persons with disability within the accessible route.

D.3.2.6 Eating outlets

Eating outlets with fixed seats such as fast food shops, food courts etc. shall have one accessible wheelchair seating per 10 seats or portion thereof. Seat arrangements shall facilitate wheelchair users to sit with their able bodied companion together.

D.3.2.7 Public transport terminals, bus stops, railway stations

All public transport terminals including bus stops and railway stations shall be accessible to persons with disabilities. The waiting areas of all such facilities shall be provided with seats for such persons who are unable to stand for long periods. Aisles for movement in such spaces shall be not less than 1200 mm.

Necessary signs and symbols shall denote the accessible routes and facilities within or outside all such buildings or facilities where 'Tactile indicators' shall guide the passengers form public footpath and accessible parking areas to specific ticket counter, waiting areas, toilets and other service facilities, arrival and departure platforms and to exits.

Doors of all public transports shall be accessible universally. Minimum clearance of all such doors shall be 900 mm. All such public transport should have at least two designated seats per coach near the door reserved for people with disability. A stall for a wheel chair per coach near the door should also be designated with provision for wheel stop blocks, safety bar with safety straps and adequate signage.

In all public railway carriages with toilet or dining facilities, the aisle width within the carriage should be at least 1200 mm.

It is preferable that there should be no level change between the platform level and the deck of the transport in use. However, where such level differences occur, ramp or lift facilities should be available to ensure universal accessibility.

D.4 Minimum Space Allowances

D.4.1 Minimum Space Requirement for Wheel Chair Users

Depending upon the nearest obstruction and the direction of movement a wheelchair user shall be considered for two approaches and shall require a minimum of 900 mm x 1200 mm unobstructed floor area as shown in Figure 3.D.1.

To facilitate both parallel and forward approaches for wheelchair users, a minimum clear floor space of 1200 mm x 1200 mm, as shown in Figure 3.D.2.

The minimum clear turning space for a manually operated wheel chair shall be 1500 mm x 1500 mm. For a powered wheel chair the requirement of turning space shall be 2250 mm x 2250 mm.

Where two wheelchairs are required to cross side by side, a minimum accessible clear width of 1800 mm shall be provided. The minimum width of an accessible route shall be 1200 mm, as shown in Figure 3.D.3.

D.4.2 Projection, Protrusion and Obstacles in an Accessible Route

All along the pedestrian areas accessible to persons with disabilities (e.g. walkways, halls, corridors, passageways etc.) any kind of obstacle, projection or protrusion shall be avoided. For all such areas an obstacle or projection or protrusion of 100 mm or less from side walls within the circulation space may be exempted. When such protrusion is more than 100 mm, the bottom edge of the protruding object shall not be more than 580 mm above the floor level, as illustrated in Figure 3.D.4. Such projections or protrusions shall not reduce the clear width required for an accessible route or maneuvering space; to provide such protruding objects, space shall be provided to accommodate those objects in addition to the required clear width.

The minimum clearance for headroom in all accessible areas such as walkways, halls, corridors, passageways or aisles shall be 2000 mm. Any free standing post or object on or beside an accessible route shall follow the guidelines of Figure 3.D.5.

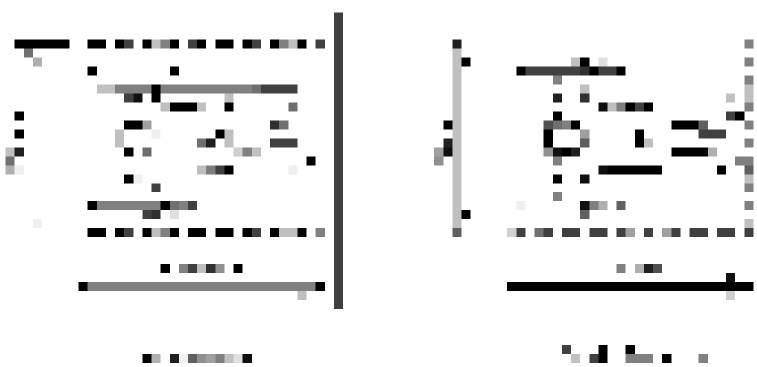


Figure 3.D.1 Minimum clear floor space

Figure 3.D.2 Minimum clear floor space (both frontal parallel approach)

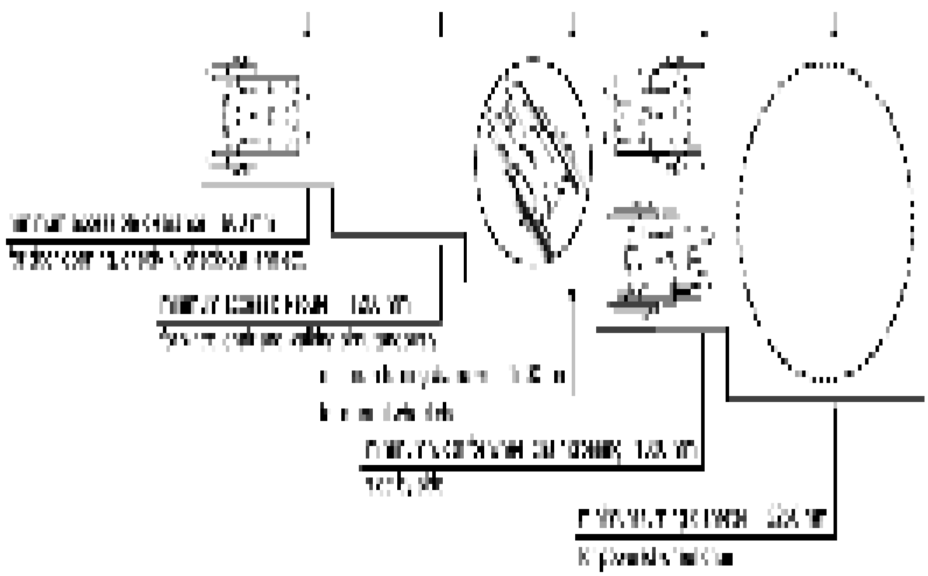


Figure 3.D.3 Minimum width of accessible routes

D.5 Surface Quality of Floor Space

D.5.1 General

All pavement or floor surfaces required to be accessible shall be firm, even, slip-resistant and stable. Any change of level of such surfaces shall be negotiated in compliance with Sec D.5.2 or through accessible lifts as per provision of this Code. To assist persons with visual impairment, such floors or their skirting shall have finishes of contrasting color with adjacent walls.

D.5.2 Change in Level

Any change of level in an accessible route shall generally have gradient of at least 1 vertical to 12 horizontal towards the direction of travel. All such slopes shall have special markings with contrasting colors at the top and the bottom of the ramp or on the ramp slope as shown in Figure 3.D.6.

However, for change of vertical level up to 150 mm within any accessible route a steeper slope may be allowed in accordance to Table 3.D.2.

Since for some ambulant disabled persons, steps are convenient and safer to use than ramps, accessibility provision by both ramps and steps should be given.

Figure 3.D.4 Limit of protruding objects

D.5.3 Gratings

For safety of people with disabilities, the elevation of gratings located on an accessible route shall be at the same level and aligned perpendicular to the direction of travel. The gap of such gratings shall not be more than 12 mm at any direction.

D.5.4 Surface Texture

In an accessible route, apart from the general requirement of Sec D.5.1, floor surfaces with tactile indicators shall be required. In such cases dot type surface texture on floor shall indicate a warning, while line type surface texture on floor shall indicate the intended path of travel.

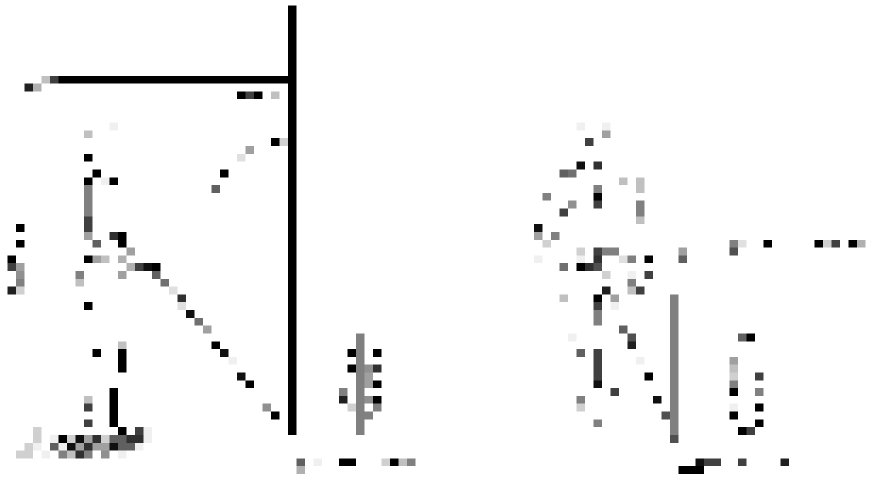


Figure 3.D.5 Limit for free-standing objects mounted on post

Figure 3.D.6 Markings on an internal ramp

Table 3.D.2: Gradient for Changes in Levels

Maximum Vertical Change of Level (mm)	Maximum Allowable Length (mm)	Maximum Slope Ratio
0 to 75	600	1 vertical to 8 horizontal
more than 75 to 150	1500	1 vertical to 10 horizontal
More than 150	9000	1 vertical to 12 horizontal

D.6 Approaches

D.6.1 Public Access Ways

The minimum unobstructed width of an accessible public access way such as footpath, corridor, foot over bridge, under pass etc. shall be 1200 mm. All such ways shall have a 1500 mm x 1500 mm space per every 30 m of length to facilitate crossing or turning of users. However for pathways with width of 1500 mm or more no additional width shall be required. The minimum access width shall not be encroached by obstruction or protrusion of any kind and shall comply with provisions of Sec D.4.2.

D.6.2 Vehicular Approach to Building

To facilitate persons with disability approaching by vehicles the driveway, walkway and accessible parking surfaces within a site shall either be merged to a common level or be connected by ramp (Sec D.5.2 and Table 3.D.2).

For occupancies mentioned in Sec D.3.2, at least one accessible route leading to an accessible entrance of the building shall be provided from the descending and boarding point of vehicle parking lots for persons with disabilities.

D.6.3 Access to Building

All accessible buildings or facilities as specified in Sec D.3 shall have at least one accessible entrance door, located preferably with the main entrance and connected to an accessible route which shall be minimum 1200 mm wide. All accessible entrance doors shall comply with the provisions of Sec D.8.

D.6.3.1 Directional signs

To direct persons with disabilities to the accessible entrance/directional signs bearing the symbol shall be displayed at all other non-accessible entrances and accessible parking areas.

D.7 Accessible Routes, Corridors or Paths

D.7.1 Length Width and Height

Any accessible route should not be more than 30 m of length at a stretch. Where such routes exceed this limit, provisions of seating preferably with shading shall be required to reduce strain of persons with disability. The minimum width of all accessible routes shall comply with the provisions of Sections D.4 and D.6.1. Where one way accessible check-in or check-out lanes are provided, the minimum width shall be 900 mm. The minimum height or headroom clearance for any accessible route shall be 2000 mm all along its path of travel.

D.7.2 Surface Finishes

All surfaces, edges, ends and corners of surrounding building and finish materials along an accessible route shall be free from sharp edges and shall comply with provisions of Sec D.5.

D.7.3 Obstruction or Protrusion on Accessible Route

An accessible route shall be free from any kind of obstruction or protrusion. The minimum circulation space required for persons with disability in such route shall not be impeded or obstructed by projection or protrusion from side walls, overhead planes or from floor below.

If incase vertical obstacles such as posts, bollards etc. are inevitable on or beside an accessible route there shall be at least 900 mm clearance between them to allow through circulation. Overhead obstacles such as drop beam, signboards, canopies etc. shall have a minimum clearance of 2000 mm from the floor level of the accessible route. All possible obstacles shall have color contrast with their background to ensure clear visibility.

Protrusion from side walls on or beside an accessible route shall follow the guidelines of Sections D.4.1 and D.4.2. Projections or protrusions shall not reduce the clear width requirement for an accessible route; when such protruding objects shall be there, space shall be provided to accommodate those objects in addition to the required clear width.

D.7.4 Warning for Overhead Hazard

The minimum clear headroom in all accessible areas shall comply with Sec D.4. Whenever the headroom of an area adjoining an accessible route is less than 2000 mm, a detectable guardrail having its detectable edge at or below 580 mm from the floor level shall be provided as shown in Figure 3.D.7 to warn persons with visual impairment.

D.7.5 Physical Cue and Tactile Guidance

All accessible routes shall have provisions for physical cues and tactile guidance for persons with disability as per provisions of Sections D.5.4 and D.29.

D.8 Accessible Doors

D.8.1 General

An accessible doorway shall ensure the access of all people with specific provisions for unassisted wheelchair users safely and without inconvenience. For occupancies mentioned in Sec D.3.1, if revolving doors or turnstiles are required an ancillary swing door with a clear opening of minimum 900 mm shall be required to ensure accessibility.

The door threshold should preferably be at the same level with the floor. However if absolutely necessary, the allowed level change shall be maximum 20 mm from the floor level and shall be sloped to allow wheelchair access. All accessible bathroom and toilet doors should swing outwards to facilitate external emergency assistance. Accessible door shall have color contrast with its adjacent walls.

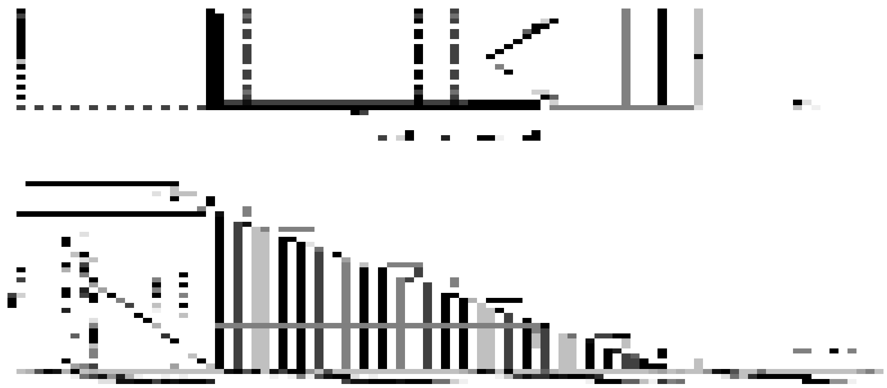


Figure 3.D.7 Protection from overhead hazard

D.8.2 Width of Accessible Door

A single leaf of any accessible doorway shall be 900 mm minimum measured between the face of the door leaf open at 90° and the face of the opposite jamb as illustrated in Figure 3.D.8. Where doorways have double-leaf at least one operable leaf shall allow 900 mm clearance to ensure accessibility.

D.8.3 Unobstructed Spaces for Operating Doors

All accessible swing doors shall have unobstructed spaces for wheelchair users on both side of the door leaf. In such cases the side, in which the door leaf swings open, shall be known as pull side while the opposite as push side. The requirement of unobstructed spaces in both the sides shall be in compliance with Figure 3.D.9.

Where two-way swing doors are used in an accessible route, both side shall be considered as pull sides and a vision panel complying with provisions of Sec D.8.5 shall be provided.

D.8.4 Door Operating Hardware

If not automatic, all accessories required for operating an accessible door such as door handles, fasteners, locks etc. shall be manually operable by one hand with ease. The height of all such accessories shall be within the range of 900 mm to 1100 mm from the floor level. Door handles are recommended over door knobs as knobs may be harder to operate for persons with grip difficulties, Figure 3.D.10.

D.8.5 Vision Panels

For the safety of ambulant disables or wheelchair users, all two-way swing doors across any accessible route shall have transparent vision panels as shown in Figure 3.D.11; where the bottom edge of such panels shall not be higher than 800 mm while the top edge of the panel shall not be less than 1500 mm, both measured from the floor level. The width of the viewing panel shall be not less than 150 mm. Such panels shall always be located at the opposite end of the hinged end on a door leaf.

D.8.6 Turnstiles

Whenever a turnstile is placed on an accessible route, an accessible gate with a clear width of at least 900 mm should be provided beside a turnstile, Figure 3.D.12.

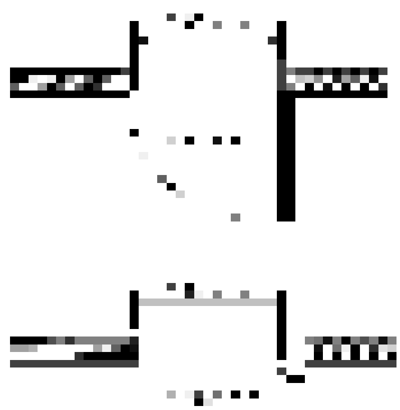


Figure 3.D.8 Clear width of accessible door

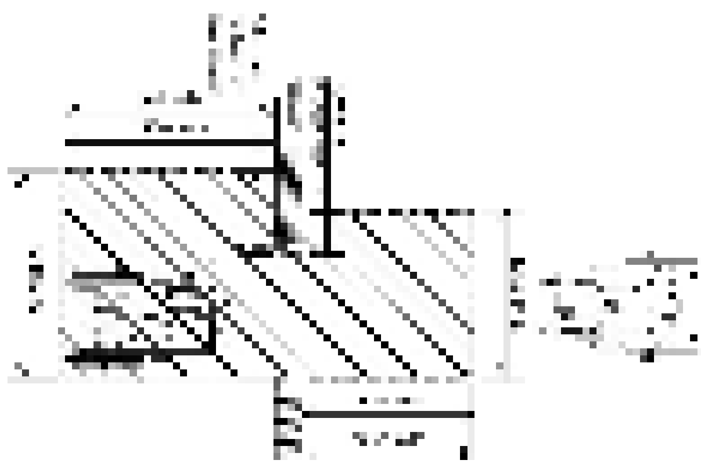


Figure 3.D.9 Minimum unobstructed space at doorway

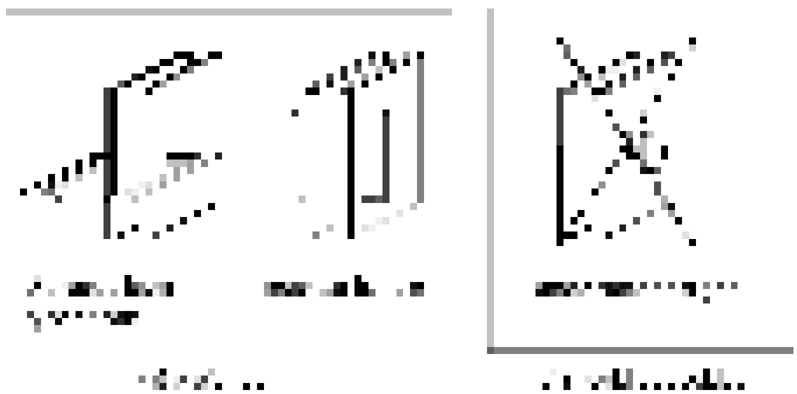


Figure 3.D.10 Preference of door handles

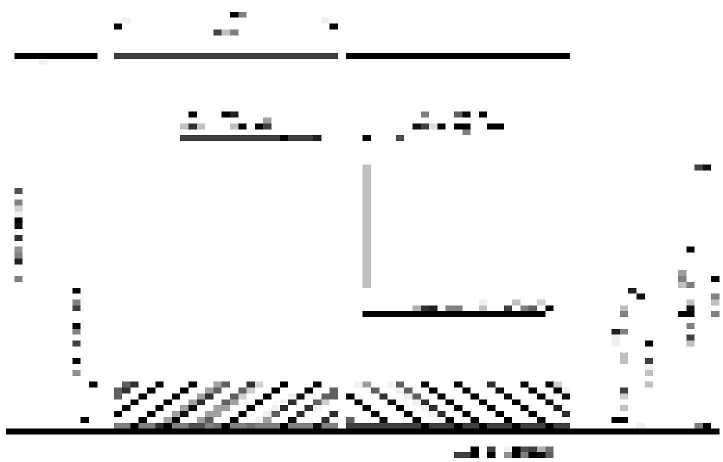


Figure 3.D.11 Dimensions and location of vision panel

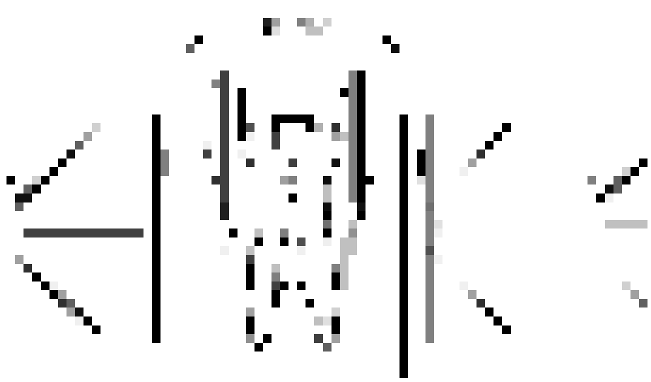


Figure 3.D.12 Access provision for turnstiles

D.9 Handrails and Grab Bars

D.9.1 General

Handrails and grab bars are very important safety features for any accessible facility. Therefore all such rails and bars shall be of accurate size and shape, slip-resistant, free of sharp or abrasive finishes and shall firmly hold with the supporting walls or floors or other form of supports. All such handrails shall have continuous gripping surfaces at a constant height throughout their length so that persons with disability do not lose balance due to loss of grip. There shall not be any sharp edges or corners in a handrail and a grab bar that may pose risk of injury. Handrail and grab bars should have color contrast with the background. Such handrails and grab bars shall not encroach on the minimum clear space for circulation.

D.9.2 Specific Requirements for Handrails and Grab Bars

All handrails in any accessible facility shall have a circular section of 35 mm to 50 mm external diameter or an equivalent gripping surface of any other section. The clearance between such hand rails and its adjacent wall shall be between 40 mm to 60 mm as shown in Figure 3.D.13.

Any recess containing a handrail shall have at least 450 mm clearance above the top of the rail as shown in Figure 3.D.13. Height of such handrails shall be within a range of 850 mm to 950 mm measured from the floor or in case of a stair from the nosing.

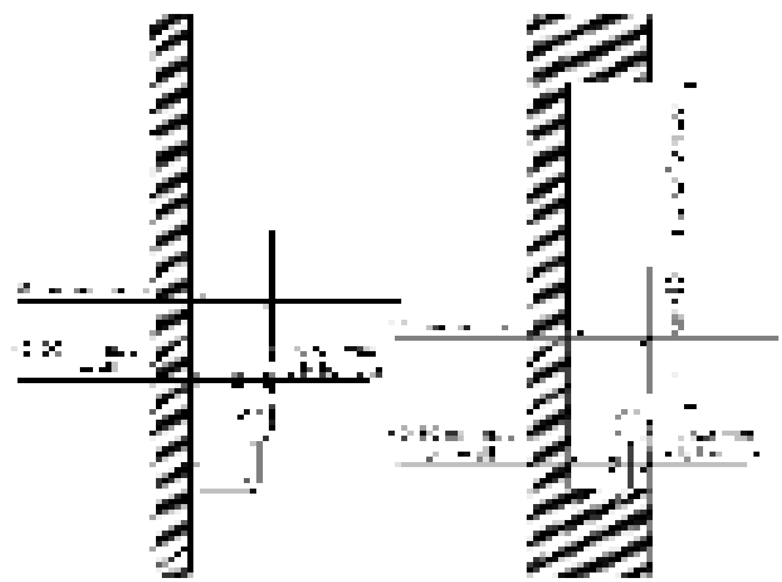


Figure 3.D.13 Handrails

D.9.3 Structural Strength

All hand rails and grab bars in an accessible facility shall be designed and built to resist a force of at least 1.3 kN applied vertically or horizontally.

D.10 Curb Ramps

D.10.1 General

Curb ramps in an accessible route should be kept within the pedestrian part of the circulation route and should not protrude within the vehicular area. If such protrusion is unavoidable, the curb ramps should be constructed with flared sides with gradient specified in Sec D.10.2. Such ramps do not require handrails as long as the level change is not greater than 150 mm.

D.10.2 Gradient, Width and Surface of Curb ramp

The gradient of a curb ramp shall follow the provisions of Table 3.D.2. The width of a curb ramp shall not be less than 900 mm. Where the vertical change of level is greater than 150 mm or the horizontal run is more than 1500 mm, it shall constitute an accessible ramp and shall conform to the requirements of Sec D.11. All surfaces of curb ramps shall be slip-resistant and shall have a detectable warning surface of contrasting color and texture complying with provisions of Sec D.29 for visually impaired persons. Curb ramps with flared sides shall not be steeper than 1:10 and shall follow the specifications shown in Figure 3.D.14.

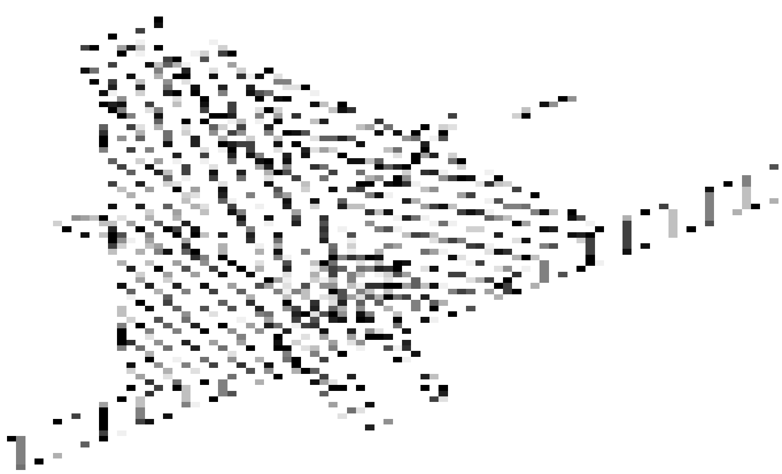


Figure 3.D.14 Curb ramp with flared sides

D.10.3 Location of Curb Ramps

All curb ramps should be built within the pedestrian zone and should not protrude to parking or any other vehicular area. However, in locations such as street crossing, road islands, road dividers and so on, curb ramps shall strictly be located within the pedestrian areas and shall follow the guidelines of Figure 3.D.15 and 3.D.16.

D.11 Accessible Ramps

Accessible ramps shall be used to provide connectivity between levels having height difference of more than 150 mm within a facility which are not served by accessible lift facilities. All such ramps shall comply with the provisions of Sections D.4 and D.5.

D.11.1 Gradient, Width and Surface of Accessible Ramp

The gradient of an accessible ramp shall follow the provisions of Table 3.D.2. The width of an accessible ramp shall not be less than 1200 mm. All surfaces of curb ramps shall be slip-resistant and shall have a detectable warning surface of contrasting color and texture complying with provisions of Sec D.29 for visually impaired persons. Where the horizontal run of an accessible ramp exceeds 9.0 m in length, there shall be a landing of at least 1500 mm length with tactile warning surface as shown in Figures 3.D.17 and 3.D.18. All such ramps shall have hand rails on both sides complying with provisions of Sec D.9.

D.12 Accessible Stairs

D.12.1 General

Stairs cannot provide accessibility for all persons with disability. Therefore stairs can only be an optional requirement for the ambulant disabled along with lifts or ramps. Any such stair or staircase should comply with the requirements of Sections D.12.2 to D.12.4. All handrail of accessible ramp shall have extensions either to floor or to wall as shown in Figure 3.D.19. For safety reason stairs with open risers or risers with projecting nosing as shown in Figure 3.D.20 shall not be considered as accessible stair for ambulant disabled.

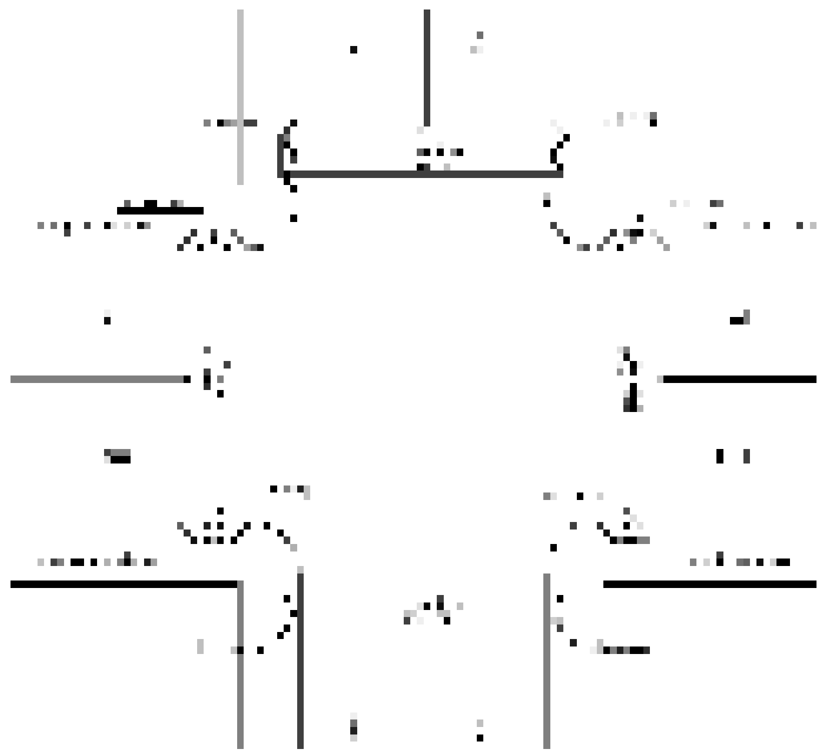


Figure 3.D.15 Location of curb ramp at street crossing

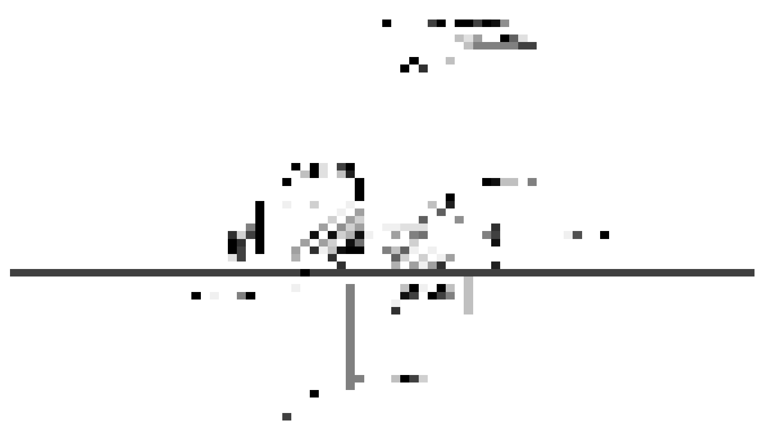


Figure 3.D.16 Location at road dividers

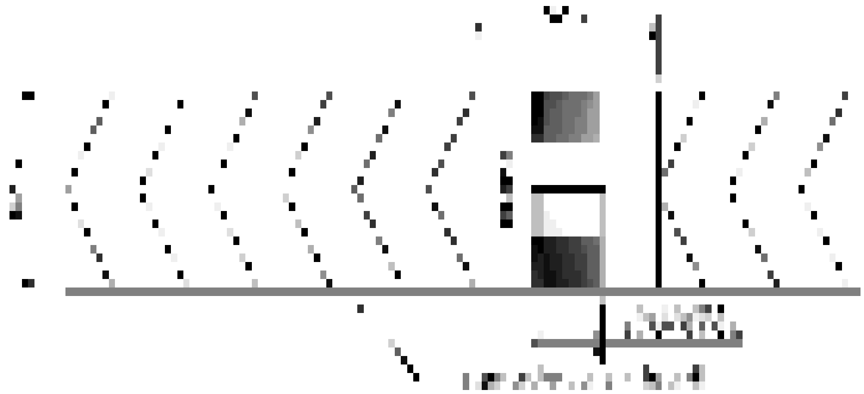


Figure 3.D.17 Plan of straight ramp and landing

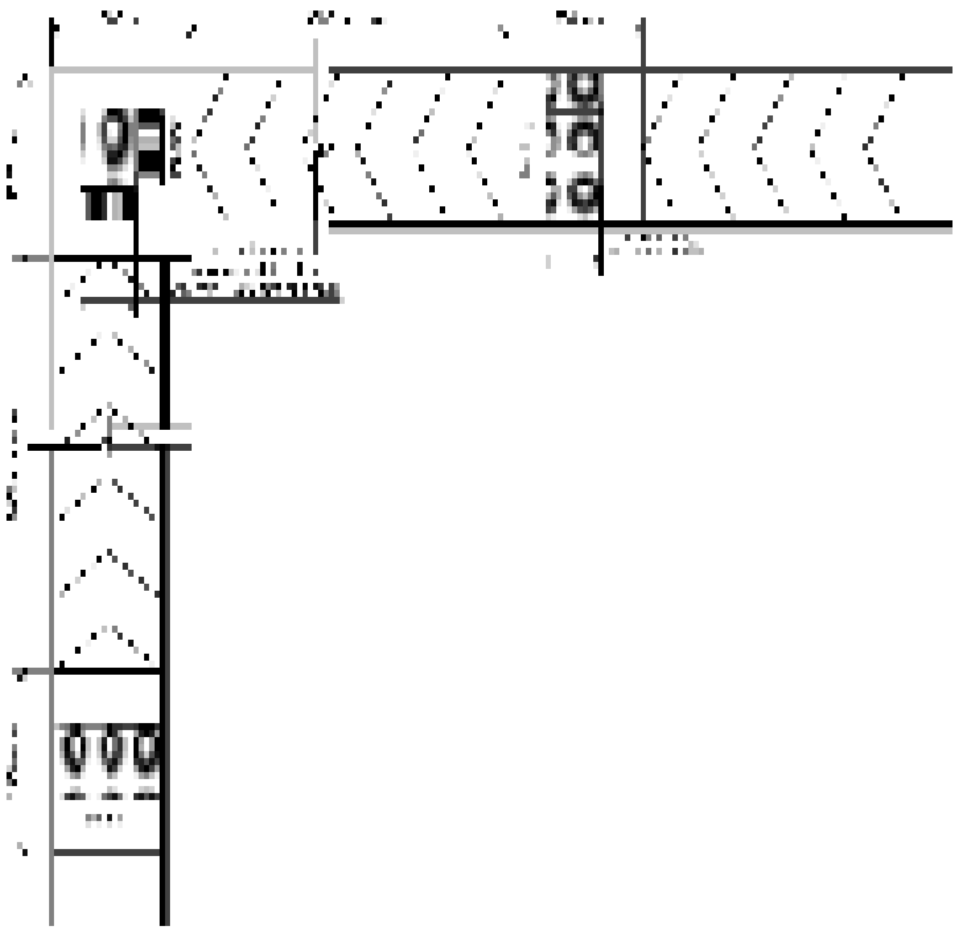


Figure 3.D.18 Plan of right-angled ramp and landing

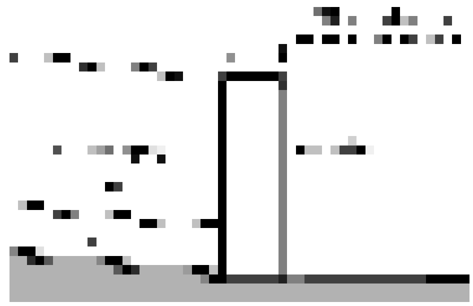


Figure 3.D.19 Hand rail extension (to floor or wall)

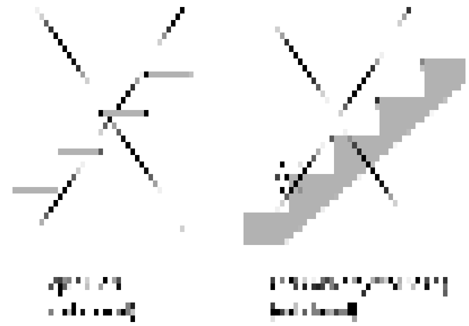


Figure 3.D.20 Stair detail

D.12.2 Tread, Riser and Nosing

All continuous flights of steps shall have uniform riser height of maximum 150 mm and tread width of minimum 300 mm. The risers shall be either vertical or receded back as per guidelines of Figure 3.D.21.

All steps should be fitted with contrasting visually detectable non-slip nosing as shown in Figure 3.D.21.

D.12.3 Warning Indicators

Stairs like any other level changes poses risks of accidents to persons with visual impairment. So all stairs in an accessible facility shall have detectable tactile warning strips provided at the top, bottom and intermediate landings in compliance to provisions of Sec D.29, Figures 3.D.21 and 3.D.22.

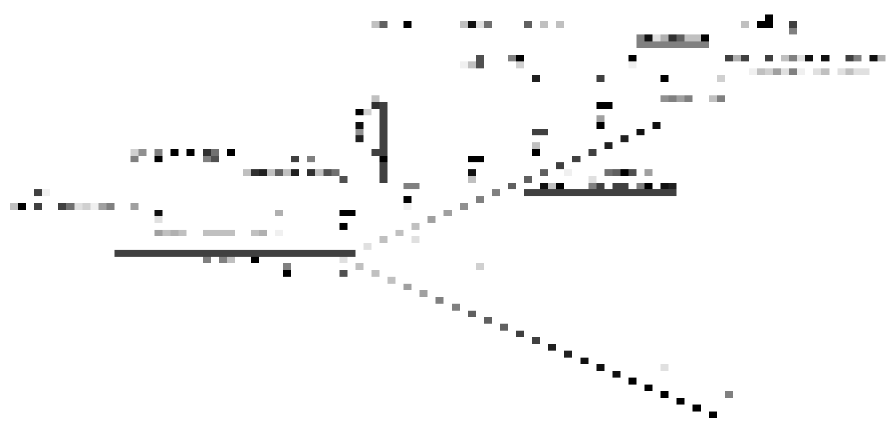


Figure 3.D.21 Tactile warning at beginning and ending of stairs and detectable edges

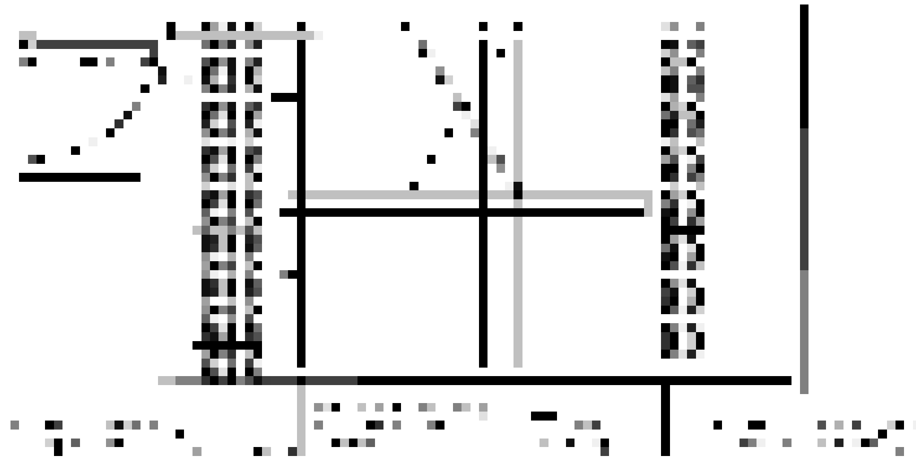


Figure 3.D.22 Tactile warning in staircase

D.12.4 Stair Handrails

Stair handrails shall comply with provisions of Sec D.9.2. Such handrails shall be installed on both sides of a stair as shown in Figure 3.D.23 and shall be installed between 800 mm and 900 mm height measured vertically from the pitch line of the steps to the top of the handrails. Stair handrails shall be continuous throughout the entire length of the stair and extend at least 300 mm beyond the top and bottom step as shown in Figure 3.D.23.

D.13 Accessible Seating Space and Counter Services

Any accessible seating space for wheelchair users such as work stations, tables, service counters in any building occupancy shall have a clear floor space not less than 900 mm x 1200 mm. Where a forward approach is required, the clear knee space shall be at least 900 mm wide, 480 mm deep and 700 mm high as shown in Figure 3.D.24. Writing surfaces or service counters shall not be more than 800 mm from the floor as shown in Figure 3.D.24.

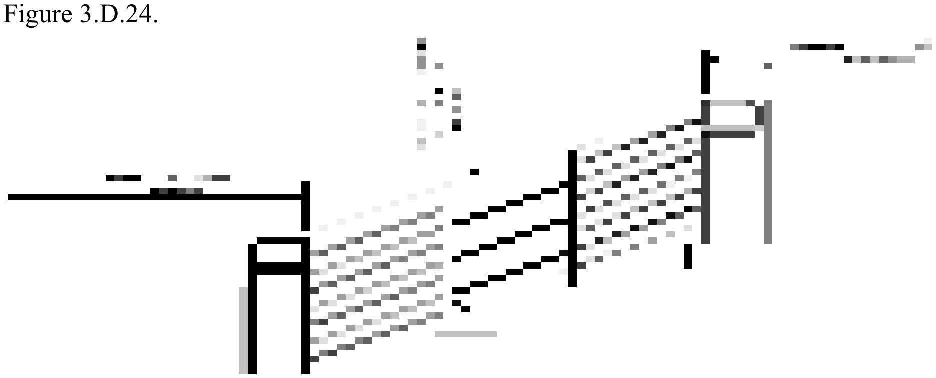


Figure 3.D.23 Handrail in stairway

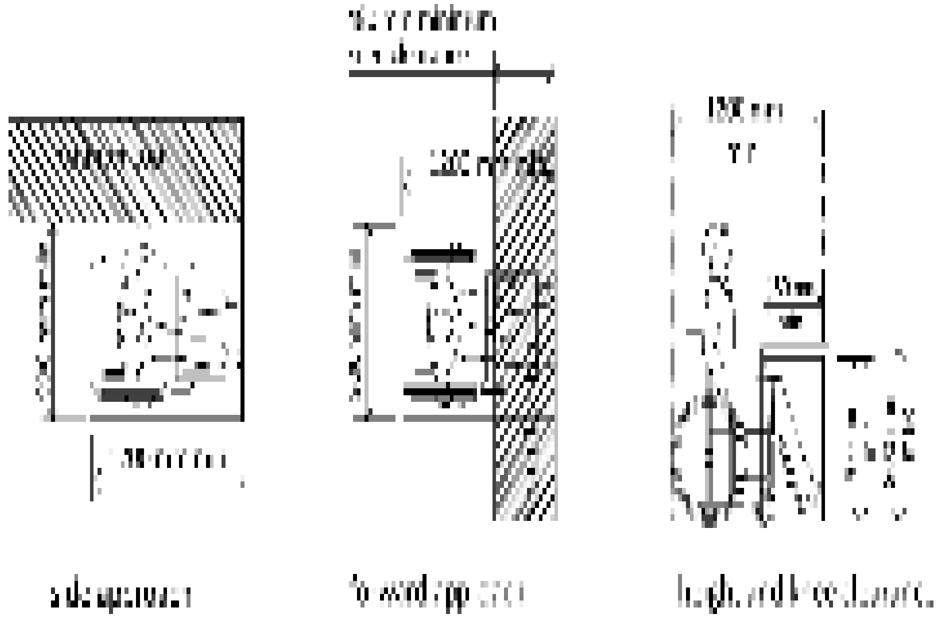


Figure 3.D.24 Forward or side approach to table or counter

D.14 Sanitary Provisions

D.14.1 General

In building occupancies described in Sec D.3, at least one toilet in each floor of a building or 5 percent of total toilets of the building, whichever is large, must be accessible. Among all accessible toilets preferably all or at least one shall be unisex in design provisions. The minimum dimension of an accessible WC compartment shall be 1500 mm x 1750 mm. All such toilets shall preferably have access directly from the accessible route. When they are part of a group of toilets, a clear approach path up to the accessible compartment with minimum of 1200 mm width shall be ensured. All accessible toilets shall have an emergency call button complying with the provisions of Sec D.17.

D.14.2 Fixtures and Accessories of Accessible Toilets

All fixtures in an accessible toilet shall abide by the provisions of this Section along with Sections D.16 and D.20 for minimum dimension, clearance from wall and other accessible clearances and limits along with minimum clear space for wheel chair maneuvering.

D.14.2.1 Accessible wash basin

All accessible basins shall comply with the provisions of Figure 3.D.25.

The faucets and other controls of such basins shall not involve powerful grasping or twisting of wrist and shall preferably be automatic or lever operated. If hot water provisions are there, proper insulation must be made to ensure safety of user.

D.14.2.2 Accessible water closet

The center line of a water closet in an accessible toilet shall maintain a distance of 460 mm to 480 mm from the adjacent sidewall. The front edge of such water closet shall be at least 750 mm away from the rear wall to allow side transfer for wheel chair users. The seating top shall have a height between 450 mm to 480 mm from the floor level. All such water closet shall have a back support to lean against it in the form of a seat lid or a flush tank or an added support. The flushing control if not automatic shall be located on the transfer side of the water closet. Figure 3.D.26 shows the basic requirements for such water closets.

D.14.2.3 Accessible urinals

Where urinals are provided, at least one shall be of wall hung type with a clear floor area of 750 mm x 1200 mm with level floor plane. The rim height of such urinals shall not be more than 400 mm measured from the floor. Any privacy shield on side shall have at least 120 mm clearance from the grab bars as shown in Figure 3.D.27. All such grab bars shall be installed as per provisions of Figure 3.D.27.

D.14.2.4 Washroom accessories

All washroom accessories such as towel rail, soap dispenser, waste bin, hand dryer, mirror, emergency call bell etc. shall be located within close proximity and shall comply with the provisions of Figure 3.D.28 and Sec D.17.

D.14.2.5 Signs at washroom entrances

All accessible toilets shall have clearly visible signs at washroom entrances complying with the provisions of Sections D.26.2 and D.27.

D.15 Doors of Accessible Washroom and Water Closet Compartment

Any door of an accessible washroom and water closet compartment in fully open position shall have an unobstructed opening of at least 900 mm. For such doors, pull and push bars, sufficient clearance at both pull and push side of the door for wheel chair maneuvering shall be provided in compliance with guidelines of Sec D.8, Figure 3.D.29 to 3.D.31. All doors for accessible washroom and water closet compartments should preferably swing outward.

D.16 Grab Bars

Any accessible toilet and bathing facility shall be mounted with at least two grab bars for each toilet fixtures except wash basin. The grab bars shall have a cross-sectional area complying with the provisions of Sec D.9. The length of such grab bars shall not be less than 600 mm. When both horizontal and vertical grab bars are required it is preferable that they should be continuous. All such grab bars shall follow the guidelines of Figure 3.D.28.

A horizontal grab bar mounted to the closest side wall of the water closet shall have a length starting from the rear wall and extending at least 450 mm beyond the front edge of the water closet and the same wall shall have a vertical grab bar as illustrated in Figures 3.D.31 to 3.D.33.

A foldable grab bar shall be mounted on the wider transfer side of the compartment as illustrated in Figures 3.D.26 and 3.D.29. Keeping a clearance of 380 mm to 400 mm from the center line of the water closet and same height with other grab bars. Foldable grab bars shall not extend more than 100 mm from the front edge of a water closet.

D.17 Emergency Call Bell

All accessible toilets, water closet compartments and wash rooms shall have a water proof emergency call bell in each compartment. Such emergency call bells shall be either push-button type or pull-chord type located for convenience of use at a height between 600 mm to 650 mm above the floor level. The buzzer of such call bells shall be so located that immediate attendance shall be available quickly.

D.18 Individual Water Closet Compartment

Any accessible water closet compartment for wheel chair users as required in Sec D.14.1 shall have a minimum internal dimension of 1500 mm x 1750 mm. All such water closet compartments shall comply with the provisions of Sections D.14.2.2, D.15, D.16, D.17, Figures 3.D.26 and 3.D.29.

D.19 Water Closet Compartment In Public Toilet

Any accessible water closet compartment for wheelchair users in a public toilet facility shall have a clear internal dimension of not less than 1500 mm x 1750 mm. All such water closet compartments shall comply with the provisions of Sections D.8, D.14.2.2, D.15, D.16, D.17 and Figure 3.D.30.

D.20 Bath Facilities

D.20.1 General

All residential occupancies, where accessible toilets are required by the provisions of Sec D.3, shall be provided with accessible bathing facilities either by providing bathtub complying with Sec D.20.2 or by providing shower stall complying with Sec D.20.3. Sports facilities and public swimming pools that need accessible provisions according to Sec D.3.1 shall also be provided with shower compartments in both male and female areas complying with Sec D.20.3.

Figure 3.D.25 Details of accessible wash basin

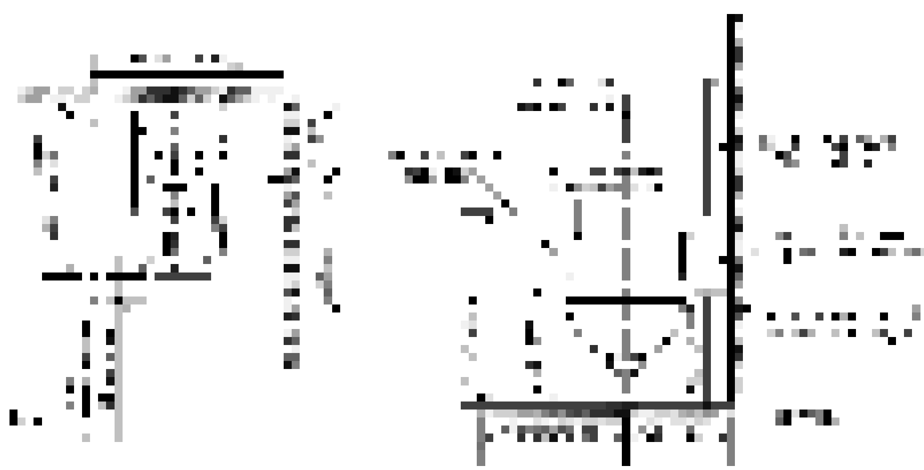


Figure 3.D.26 Accessible water closet for wheel chair users

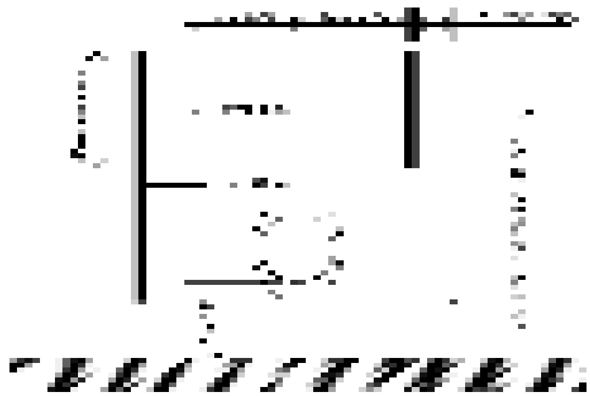


Figure 3.D.27 Basic dimensions for accessible urinals

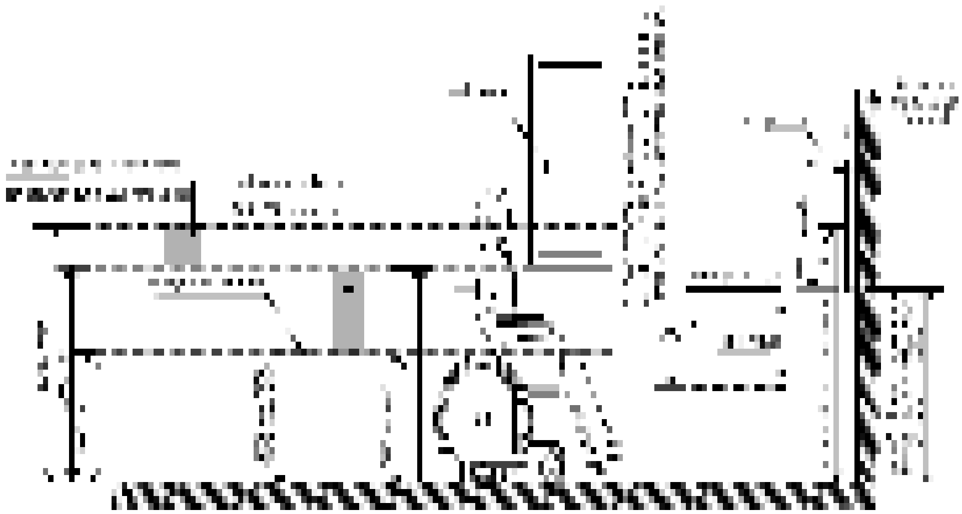


Figure 3.D.28 Standard dimensions for wash-room accessories and grab bars

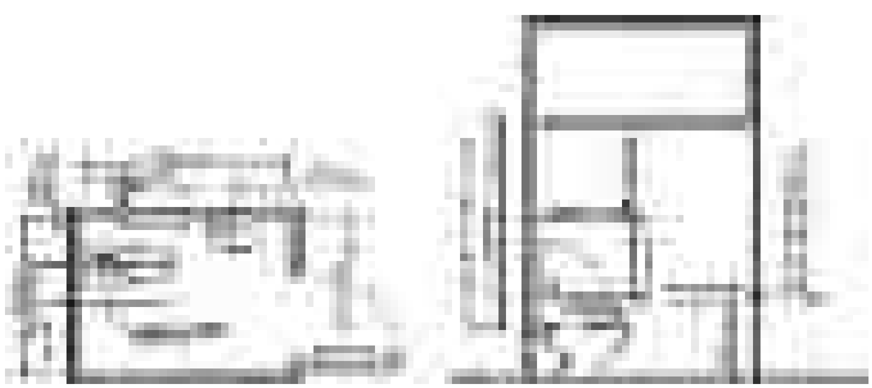


Figure 3.D.29 Water closet compartment for wheel chair users

Figure 3.D.30 Detail of water closet compartment in group toilet

D.20.2 Accessible Bathtub

Any accessible bathtub shall have a clear floor space of at least 750 mm x 1200 mm along its length as shown in Figure 3.D.31. A seat of at least 250 mm width along the entire length or width of such bathtub as shown in Figure 3.D.31 and shall be required. The floor of accessible bathtubs shall be slip resistant. The base of such bathtubs shall be slip-resistant. All accessible bathing facility shall have grab bars complying with Sec D.9 and with the provisions of Figures 3.D.31 and 3.D.32. Shower heads in such facilities shall be hand-held type with flexible cords and shall comply with the provisions of Figures 3.D.31 and 3.D.32. All other accessories of such facilities shall comply with Sec D.14.2.4.

D.20.3 Accessible Shower Stall

An accessible shower stall shall have internal dimensions of at least 1500 mm x 1500 mm and shall comply with the provisions of Figure 3.D.33. The floor and seat of such accessible shower compartment shall be slip-resistant. The shower heads of such showers shall be hand-held type with flexible cord. All faucets and accessories of such shower compartments shall follow the guidelines of Sec D.14.2 and Figure 3.D.33. All such shower compartments shall have grab bars in compliance with Sec D.9, Sec D.16 and Figure 3.D.33. Any level change of such floor shall not be more than 10 mm and shall be negotiated with a slope ratio of one vertical to two horizontal.

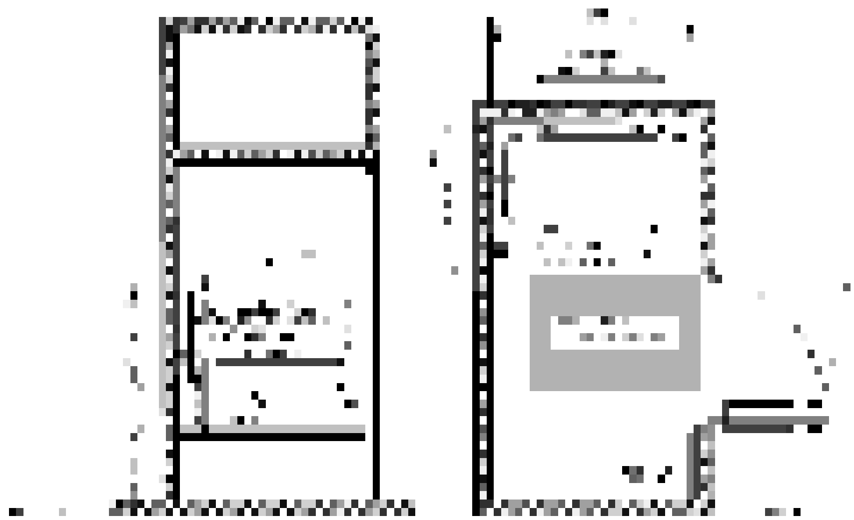
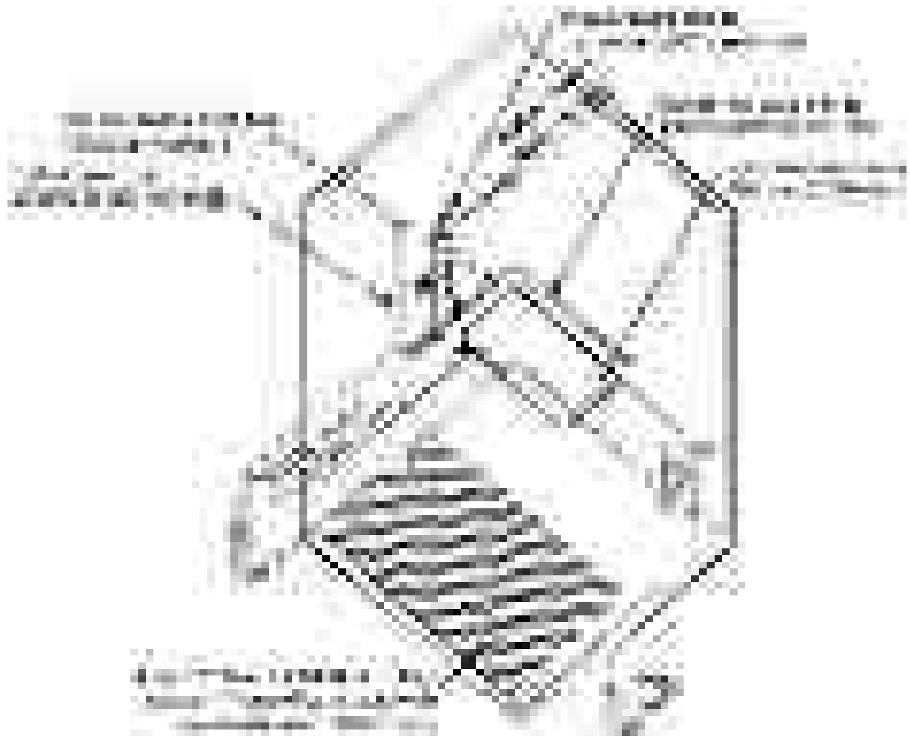


Figure 3.D.31 Bathtub for persons with disabilities

Figure 3.D.32 Layout plan for 3 fixture toilet



D.21 Kitchen Facilities

An accessible kitchen may have an open layout (e.g. pass-through type) or a closed layout (e.g. U-shaped). The open layout consists of a straight pass through aisle, which can be entered from both ends and where working top, appliances and cabinets are on two opposing sides as shown in Figure 3.D.34 (A).The clear width of such aisle shall be not less than 1015 mm.

The closed layout requires turning radius of a wheel-chair within the kitchen area resulting in a layout enclosed on three contiguous sides ensuring a minimum clearance of 1525 mm between all opposing cabinets, working tops, appliances and walls, as shown in Figure 3.D.34 (B).

All appliances shall be clearly approachable either by front approach or by parallel approach. Where a forward approach is provided, the clear floor or ground space shall provide knee and toe clearance as per provision of this Code. Knee and toe space under cooking range shall be insulated to prevent burns or abrasions or electrical shock. At least fifty percent of all cabinets and storage spaces shall be accessible as per provision of this Code. The height of the working top, sink, cooking range and all necessary appliances shall follow the guidelines for accessibility of this Code.

D.22 Lifts

D.22.1 General

Buildings, where lifts are needed as part of requirement by the building authority, should have at least one accessible lift for vertical circulation from the entrance level and serve all levels intended for use by persons with disabilities. Lift lobby for such facilities shall have a minimum dimension of 1500 mm × 1500 mm. The minimum size of an accessible lift car shall be 1500 mm × 1725 mm with a clear door opening of not less than 900 mm. Such accessible lifts shall follow the guidelines of Sec D.5 for floor finish, Sec D.9 for horizontal grab bar on back and side walls and the guidelines of Figure 3.D.35. All accessible lift shall have tactile marking and Braille on all buttons.

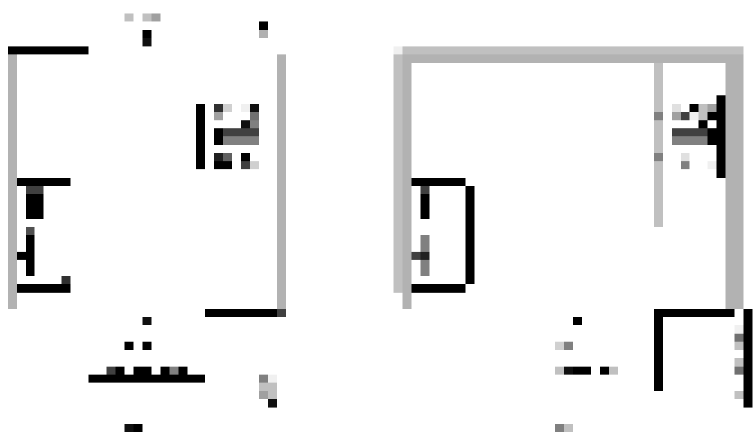


Figure 3.D.34 Accessible kitchen clearance

Figure 3.D.35 Lift for persons with disabilities

D.23 Eating Outlets

D.23.1 General

All eating outlets, with or without fixed seats, as mentioned in Sec D.3 shall have provisions of access for persons with disabilities complying with Sec D.23.2. Any aisle of circulation in such outlets shall be at least 1200 mm wide.

D.23.2 Seating

In an accessible eating outlet, the minimum clear space between seats in the required number of accessible tables shall be 750 mm measured along the edge of the table as shown in Figure 3.D.36. All such tables provided for persons with disabilities shall comply with provisions of Figure 3.D.37. All such tables should be clearly marked with accessibility symbol and shall have directional signage for indicating location.

Figure 3.D.36 Space requirement for accessible seating

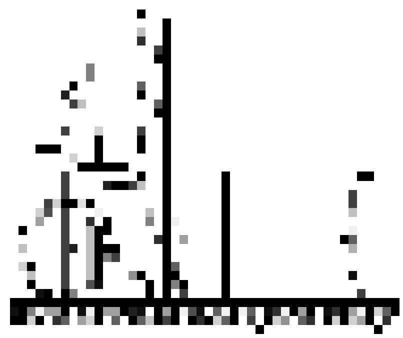


Figure 3.D.37 Clearance for accessible seating

D.24 Accessible Parking Area

D.24.1 Parking Provision

In all occupancies referred in Sec D.3 where vehicle parking is required, the number of accessible parking stalls for vehicles for persons with disabilities shall be in accordance with Table 3.D.3. Such parking lots should be located as nearer as possible to the accessible entrance of the building. Pedestrian accessible routes connecting accessible parking shall be such that it avoids the risk of collision between an ambulatory disabled person and a backing out vehicle in a parking lot. Such parking shall not be occupied by vehicles of persons without disability.

Table 3.D.3: Minimum Number of Accessible Parking Stalls

Number of vehicle park stalls	Number of accessible stalls
For first 50 stalls	1
Next 400 stalls	1 additional stall per 100 parking stalls or portion thereof
Above 450 stalls	6

D.24.2 Symbols and Signage

Each accessible parking stall shall be clearly designated with the symbol of access, in accordance with the requirements of Sec D.26.2.

Such Symbol shall be painted in contrasting color at the center of the accessible parking stall, having a dimension between 1000 mm × 1000 mm to 1500 mm × 1500 mm and complying with provisions of Sec D.26.2.

The symbol of accessible parking shall be displayed at all approaches and entrances of parking lot indicating the location of such parking within the lot. Directional signs shall be displayed at every change of direction to direct persons with disabilities or their vehicle to the point of accessible parking stall.

D.25 Accessible Vehicle Parking Stalls

The minimum dimension of an accessible vehicle parking stall shall be 4800 mm × 3200 mm. All such parking shall be provided on a firm, non-slippery, leveled solid surface and if possible, be covered. Figure 3.D.38 shows the detail of an accessible parking stall.

Figure 3.D.38 Accessible parking stall and approach

D.25.1 Signage

Any accessible parking lot shall be identified by the symbol of access in accordance with Sec D.26.2. The size and location of all signs should be such that they ensure clear visibility all along the accessible route.

D.26 The Symbol of Accessibility

D.26.1 General

The Symbol of Accessibility is an internationally accepted language that shall be permanently and clearly displayed to indicate and/or direct to the location of various accessible facilities in and around a building. All buildings or facilities mentioned in Sec D.3.1 shall display the required symbol of accessibility in compliance with the guidelines of this Code. Any such signs and symbols shall be simple, short and easy to understand. The text and use of pictographs shall be consistent throughout the building and outdoors in any accessible facility.

D.26.2 Symbol of Access

The form of the symbol of access shall consists a symbolized figure on a wheelchair and a contrasting plain square background as shown in Figure 3.D.39 where the symbolized figure shall be white on a blue background and shall always face to the right.

Any building that offers accessible facilities shall clearly display the symbol of access at road front. Inside the premise the symbol shall denote the location of the accessible facilities including accessible parking, accessible routes, entry and other accessible services and facilities for persons with disabilities.

D.26.3 Directional Signs

Whenever changes in direction occurs directional signs incorporating the symbol of access similar to Figure 3.D.40 shall be displayed. This shall include main lobbies, passageways and all points where there is a change of direction to direct persons with disabilities to various accessible functions and facilities such as lifts, entrances, toilets, car parks and the like.

Where the location of the designated facility is not obvious or is distant from the approach viewpoints, directional signs incorporating the symbol of access, as shown in Figure 3.D.41, should be placed along the route leading to the facility.

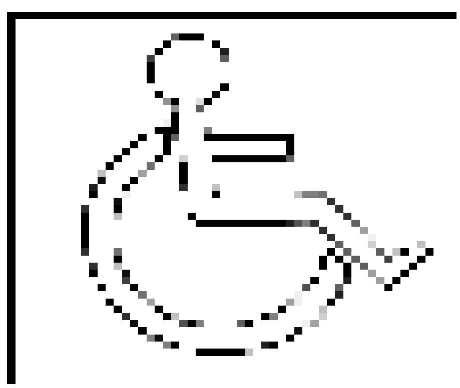


Figure 3.D.39 Symbol of access for persons with disabilities

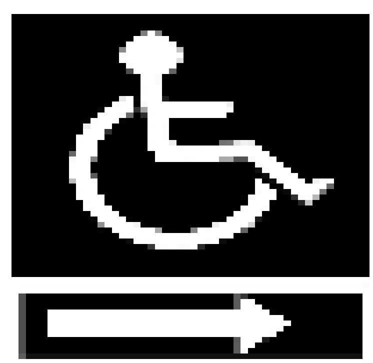


Figure 3.D.40 Accessible directional sign

D.26.4 Service Identification Signs

Every accessible route shall contain service identification signs showing appropriate symbol of accessibility for persons with disabilities, as shown in Figure 3.D.42, to indicate the presence and direction to various service facilities such as entrances, lifts, telephone booths, toilets, vehicle parks, staircases and the like. Tactile pictographic signs shall distinguish between male and female toilets.

Figure 3.D.41 Signs directing to facility

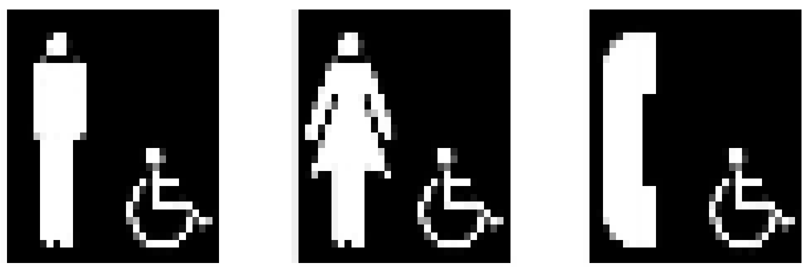


Figure 3.D.42 Service Identification Signs at Destination

D.27 SIGNAGE

D.27.1 Specifications for Characters and Symbols

Letters and numbers, when put on signs shall be legible and shall be consistent in font type all along the accessible facility. Only 'CAPITAL LETTER's shall be used in such signage. Braille, if written, shall be located directly below the text or arrow in a signage.

D.27.2 The Size of Symbols

The size of symbols depending upon the distance, it is intended to be first viewed from shall vary and shall be in accordance with Table 3.D.4.

Table 3.D.4: Size of symbols varying with distance

Viewing distance (m)	Symbol size (mm)
Up to 7.0	60 × 60
7.0 to 18.0	100 × 100
Above 18.0	200 × 200 to 450 × 450

D.27.3 The Height of Letters

The height of letters in signs depending upon the distance, it is intended to be viewed from shall vary and shall be determined in accordance with Table 3.D.5.

Table 3.D.5: Height of letters varying with distance

Required viewing distance (m)	Minimum height of letters (mm)
1.5	50
2.0	60
2.5	100
3.0	120
4.5	150
6.0	200
8.0	250

D.27.4 Location of Signs

All signs shall be located such that they are clearly and legibly identifiable form an accessible route. Any change of direction in an accessible route shall always contain necessary directional signs for users. In case of internal signs the center line of the sign shall be at a height within the field of vision and preferably at 1500 mm above the floor level.

D.27.5 Tactile Characters or Symbols

Tactile characters or symbols when used on a sign shall have a size between 16 mm to 50 mm and shall be raised at least 0.8 mm above the background surface. All such signs shall be mounted at a height complying with Sec D.27.4.

D.27.6 Braille and Pictographs

When Braille, the tactile language, is used the Braille dot shall be raised in dome shape from the base and the sign shall be easy to touch and read. The height of all such signs shall comply with Sec D.27.4.

Pictographs, when used shall be supported by equivalent textual description placed directly below it.

D.28 Grading of Slip Resistance

For the purpose of accessibility, surface materials to be used as floor finishes should be graded for slip resistance in both dry and wet conditions. Table 3.D.6 indicates the slip resistance of some commonly used finish materials.

Table 3.D.6: Slip resistance grading

Grading	Co-efficient of surface friction	Example
Very good	More than 0.75	Clay tiles, carpets, dry rubber
Good	Between 0.75 to more than 0.4	Concrete pavers, dry terrazzo tiles, dry marble and granite
Poor to Fair	Between 0.4 to 0.2	Wet and polished terrazzo tiles, marble and granite
Very poor	Less than 0.2	Wet rubber

D.29 Tactile Ground Surface Indicators

D.29.1 Path of Travel and Mobility

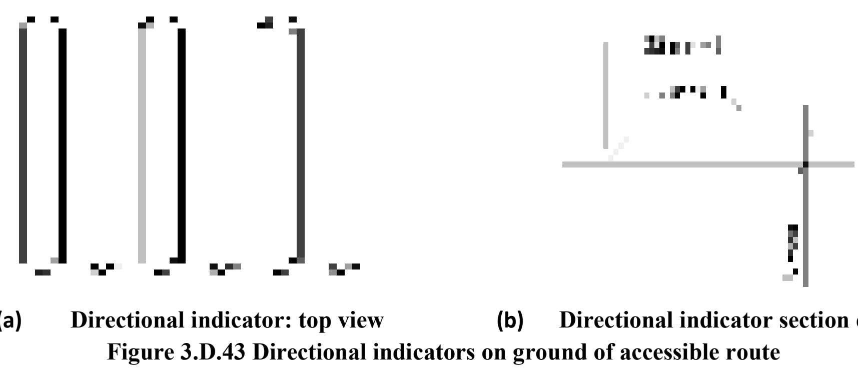
People with different forms of visual impairments can be assisted to find their way independently with the help of some physical or sensory cues e.g. landmarks and mind maps. For such users a predictable, logical and barrier free access route is required. Therefore all such path of travel dedicated to universal accessibility should be designed as free from barriers, hazards or obstructions along with physical and sensory cues for such users.

D.29.2 Physical Cues

Physical cues are designed elements including buildings, walls, ground surfaces, railings, fences and curbs that can act as cues or clues to assist a visually impaired person. Such persons can identify physical cues either by use of a white cane, under foot, or by echo-location. All public buildings referred in Sec D.3 shall have physical cue both inside and outside the building to assist visually impaired persons.

Tactile ground indicators are designed physical cue to convey two important indications to visually impaired persons- a. directional indications and b. caution or warning indications.

Directional indicators, Figure 3.D.43, act as physical cues to guide persons with visual impairment to travel through an accessible route free from obstructions from beginning to end.



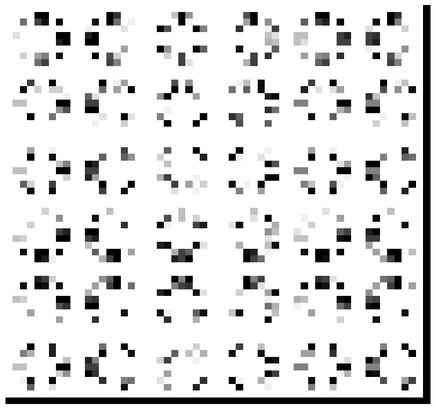
(a) Directional indicator: top view **(b) Directional indicator section detail**

Figure 3.D.43 Directional indicators on ground of accessible route

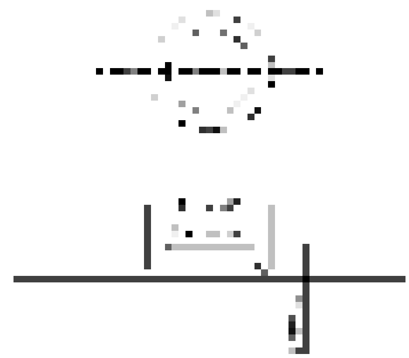
Warning indicators are physical cues for warning users of an adjacent hazard or a destination. Such hazards include but not limit to level changes, change of direction, approaching vehicular roads, obstructions etc.

The pattern and dimensions of warning indicator are shown in Figure 3.D.44 (a) and (b).

Figure 3.D.45 shows the combined use of both directional and warning tactile indicators in an accessible route.



(a) Warning indicator: top view



(b) Stud detail

Figure 3.D.44 Warning Indicators on ground of accessible route

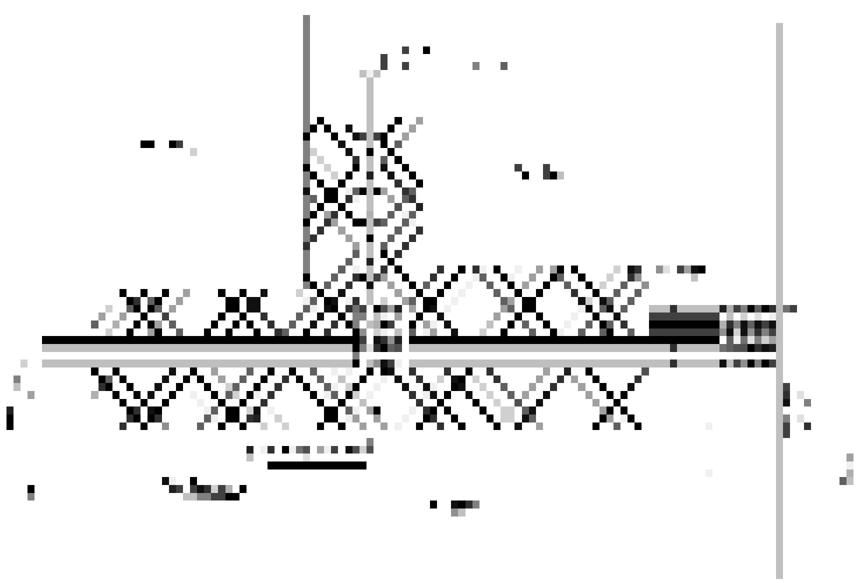


Figure 3.D.45 Use of tactile indicators in accessible route

PART III

Appendix E

Building Types, Development Rights and Buildings Abutting Property Lines

E.1 General

These guidelines cover the planning and general building requirements of different building types such as attached type, detached type, semi-detached type etc. regarding their development rights and rights regarding building on abutting property lines. These requirements are applicable to all occupancy types taken up by public, private or co-operative agencies.

E.2 Definition and Typology

The following are some definitions and diagrams to explain different typologies and terminologies relevant to this Code:

E.2.1 Row Type Building

A row type building abuts two side plot party-lines and is one of a row of buildings on adjoining zoning lots. The end buildings of a row of attached buildings are considered semi-detached buildings if they each have minimum side setback. Here the rest two (non-abutting) sides of the building are surrounded by yards or open areas within the plot confirming at least to minimum setback requirements (Figure 3.E.1).

E.2.2 Semi-detached Building

A semi-detached building is a building that abuts one side on the party-line of a plot and does not abut any other building on any other side of any adjoining plot/s; here the rest three (non-abutting) sides of the building are surrounded by yards or open areas within the plot confirming at least to minimum setback requirements (Figures 3.E.2 and 3.E.3).

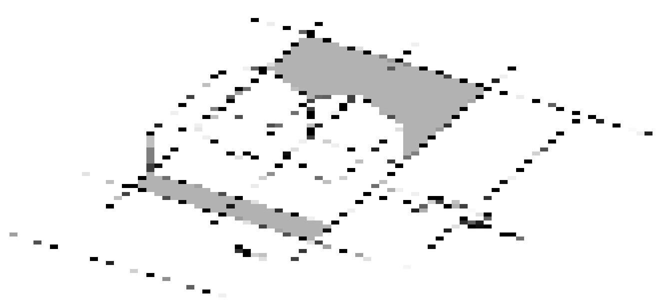


Figure 3.E.1 Row type buildings

Figure 3.E.2 Semi-detached building (not abutting neighboring building)

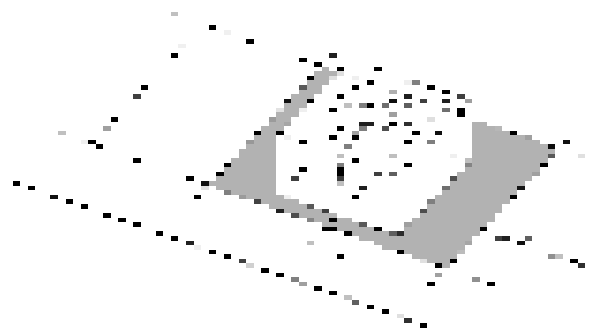


Figure 3.E.3 Semi-detached building (abutting neighboring building)

E.2.3 Detached Building

A detached building is a freestanding building that does not abut any other building on an adjoining plot and where all sides of the building are surrounded by yards or open areas within the plot confirming at least to minimum setback requirements.

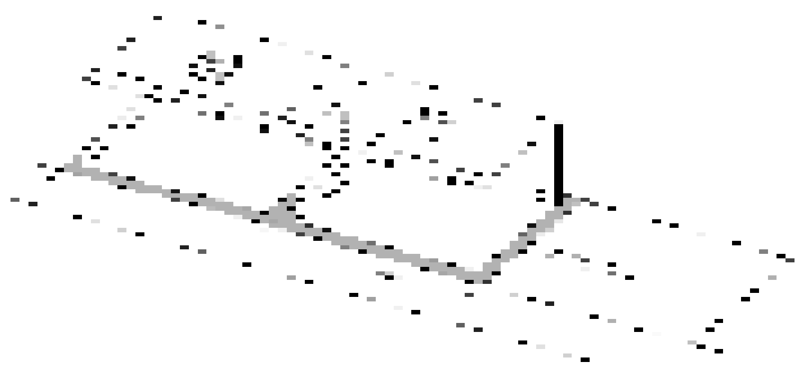


Figure 3.E.4 Detached building

E.3 Special Provisions For Construction of the Wall Abutting Parti-Line/ Site Boundary Line

E.3.1 General Requirements

Walls abutting any parti-line shall have no opening.

A parti-wall shall not contain any concealed lines (water, gas, sanitary, electricity etc.).

A parti-wall shall be so constructed that it remains moisture free, leak proof and confirms to the fire safety requirements as referred in Part 4.

A parti-wall shall confirm with structural guidelines for earthquakes and pounding gaps given in Chapter 1, Part 6 of this Code. When the adjacent buildings are designed separately, pounding gaps are mandatory. When adjacent buildings are designed and built in an integrated way through plot consolidation, no such gap is required.

E.3.2 Shared Parti-Wall

A parti-wall, if formally (in written form) agreed upon between two adjoining owners, may become a shared wall between the two properties built half on the land of each of the two owners or in such other position as may be agreed between the two owners, with both having equal rights on the use of that wall. In such cases the structural guidelines of this Code shall be abided by both parties to allow future modifications on either side of the party-wall.

E.3.3 Independent Parti-Wall

For all other cases, the owner who intends to build a parti-wall, shall make it wholly on his own land abutting the parti-line or plot boundary line and shall reserve individual right/s (not shared) of use of that wall. Parti walls, in such cases, are one or two walls abutting parti-line or having pounding gap as per structural requirement of Part 6 built on same or different times.

E.3.4 Non-Crossing of Parti/Property Line

Under all circumstance, above ground or underground, the foundation/ footing of any wall/ column or any other part of building or services, shall not cross the parti-line or plot demarcation line.

E.3.5 Foundation of Parti-Wall

For all foundation work the safety of the buildings on adjoining land or plot must be ensured. The building owner who intends to erect wall abutting the side parti-line, if necessary shall, at his own expense underpin or otherwise strengthen or safeguard the foundations of the building or structure of the adjoining property.

E.3.6 Structural Independence

Buildings with parti-wall on adjacent plots must be structurally independent. Depending upon the plot frontage and the choice of structural system a parti wall may also be non-structural or non-load bearing wall. Where parti-wall or any part of it is structural, the footing/foundation provisions must comply with Part 6 of this code.

E.3.7 Utility Lines and Drainages

E.3.7.1 Underground or surface lines and drainages

For all underground/concealed utility pipe lines, drains, ducts, etc., the outer edge of such utility pipe lines/ drains/ ducts shall be at least 900 mm inside from the parti-line within the plot to ensure maintenance accessibility without hampering parti-wall or the adjacent neighbor. For all such lines provisions for easy maintenance must be kept beforehand.

Any surface drain/ inspection pit (on finished ground level) shall be at least 250 mm inside from the parti-line within the plot.

E.3.7.2 Vertical utility lines or drainages

Under all circumstances, vertical utility lines/ducts shall not cross the parti line or plot demarcation line. No pipes, gutters, spouts, surface holes or any other type of drainage outlet to the adjoining properties can be given; special measures shall be taken to contain all drainages within the site.

E.3.7.3 Prevention of leakage

Any leakage or infiltration from these lines to neighbouring property must be prevented during and after construction. For such leakages the landowner who owns such utility lines shall be responsible to repair and compensate for any damage thereby.

For two adjacent walls abutting the same property line, the gap in between must be properly sealed.

E.4 Other Requirements

With the exception of clauses mentioned above, requirements of building will be governed by the provision of this Code and good practice.

Measures must be taken for prevention of infiltration of rain, dust and moistures through joineries of parti-walls with pounding gaps between two adjoined properties. Proper treatment for damp proofing and termite proofing in compliance with Part 6 shall be ensured. Chemical treatment to prevent long term growth of fungi and other microbial forms are recommended on such walls.

A parti-wall shall be a barrier wall in compliance with the guidelines of Part 4 to ensure fire safety of the adjoining property. Other requirements of fire safety, structural design, building services and plumbing services shall be as specified in this Code.

PART III

Appendix F

Road Hierarchy, On-Street and Off-Street Parking

F.1 Introduction

Road is an integral part of a settlement and its land-use planning. For public safety in any new development the hierarchy of road network with measures for gradual traffic calming and adequate safe parking, both on-street and off-street, are of vital importance. Road width and road components, junctions, features of controlling vehicles' speed and turning, forward visibility and visibility splay at junctions are important tools of traffic and speed control.

With increased density and parking demand, on-street parking shall be an important tool to increase overall parking capacity as well as accommodation for service vehicles (e.g. garbage collection vehicle, maintenance vehicles etc.). This measure is also expected to keep the pedestrian walkways free from unauthorized vehicular parking.

F.2 DEFINITIONS

CARRIAGEWAY	Refers to driveway that provides access to the parking place. They do not have parking stalls adjacent to them.
CARRIAGEWAY RAMPS	Refers to inclined floors that provide access between two levels.
INSIDE LANE	Refers to the innermost lane of a curve ramp, which is nearest to the center point of curve.
INSIDE RADIUS OF LANE	Refers to curved carriageway and driveway is the distance measured from the inside curve edge to the center point of the curve.
MAXIMUM GRADIENT	Refers to the steepest gradient of ramp measured along the center line of the ramp. Gradient refers to the ratio of the inclination of the ramp (height length).
OUTSIDE LANE OF CURVED CARRIAGEWAY	Refers to any lane positioned after the innermost lane.
PARKING AISLE	Refers to an access lane or driveway with adjacent parking

	stalls.
PARKING ANGLE	Refers to the angle measured between the longer side of the parking stall and the line of traffic flow of the aisle.
PARKING STALL	Refers to the space required for parking of one vehicle. The space of the stall shall be rectangular. The area of each stall shall be flat and free from curbs and other obstructions.
SINGLE-LANE	Refers to a lane where only one vehicle can pass through at any given time.
TRAFFIC FLOW	Refers to the direction of vehicle movement.

F.3 Road Hierarchy Guidelines For New Development

For any new development, at least three hierarchically interlinked road patterns should be followed (Figure 3.F.1): a. primary road, b. secondary road and c. internal/ access/ residential road. Though it is not mandatory to provide wider than regulation pedestrian walkways (with provisions for street furniture), bicycle lane and plantation zones parallel to walkways, it is strongly recommended that such provisions should be made, especially at primary and secondary road level, as much as possible.

Primary Road: This refers to a Public way or portion thereof, on which vehicular traffic is given preferential right-of-way, and at the entrance to which from intersecting public ways is required to be in obedience to a traffic signal, stop sign, or yield sign as per traffic code. Primary roads connect settlements/ zones/ sectors with rest of the city, which are capable of and usually are serviced by public transport facility (e.g. Bus service, Tram service etc.). These roads define the edge of the settlement and shall be capable of hosting traffic interchange (e.g. changing from one mode of transport to another). No individual plot should be accessed directly from a primary road (Figure 3.F.2). The right of way of a primary road shall not be less than 18 meter (Table 3.F.1 and Figure 3.F.3).

Figure 3.F.1 Road hierarchy

Secondary Road: A secondary Road is a collector road or distributor road of low to moderate-capacity, which serve to move traffic from internal/local streets to primary/arterial road. The flow of a collector road usually consists of a mixture of signaled intersections or traffic circles with primary arterial roads or other collector roads and un-signaled intersection with local/internal/residential roads.

A secondary road shall not be less than 13.5 m (Table 3.F.1 and Figure 3.F.4).

Internal/access/residential road: At the bottom of the hierarchy are local/Internal/ access streets and roads. These roads have the lowest speed limit, carry low volumes of traffic and often have pedestrian priority. The minimum width of such road will depend on the density of the adjacent plots (Table 3.F.1, Figure 3.F.5).

Figure 3.F.2 Parking beside primary/arterial road

Figure 3.F.3 Minimum dimensions for primary road

Figure 3.F.4 Minimum dimensions for secondary road

Figure 3.F.5 Minimum dimensions for Internal/access/local/residential road

F.4 Guidelines For Pedestrian Walkway

F.4.1 Any pedestrian path should be part of a pedestrian network connecting building users to different facilities and part of a city or a settlement and should enhance pedestrian friendly environment.

F.4.2 Pedestrian path or walkways should be separated and protected from vehicular driveways and any conflict between vehicular and pedestrian crossing shall be designed to ensure pedestrian safety.

Table 3.F.1: Minimum Widths of Public Means of Access to Residential Plots of new development

	2-way vehicular road width	On-street Parking/ Emergency Vehicle	Pedestrian walkway	Bus Lane / Bus Stop	Minimum right of way/ ROW Option-A*	Minimum right of way/ ROW Option-B**	Minimum right of way/ ROW Option-C***	Minimum right of way/ ROW Option D****
	V (m)	P (m)	W (m)	B (m)	V+W (m)	V+2W (m)	V+P+2W (m)	
Main/ primary road (V+2W X 1.25+2B)	6	Nil	2	3.5	Nil	Nil	Nil	18.00
Secondary road (V+2P+2W)	5.5	2	2	Nil	Nil	Nil	Nil	13.5
Internal/ access road	4.8	2	2	Nil	6.8	8.8	10.8	Nil

Note:

* For serving residential occupancy A1 and A2, the minimum right of way shall be option A

** For serving residential occupancy of A3 within walkup range the minimum right of way shall be option B

*** For serving residential occupancy of A3 above walkup range , the minimum right of way shall be option C

**** For serving residential occupancy of A3 in mixed use , the minimum right of way shall be option D

F.4.3 The minimum width of a pedestrian walkway which is not enclosed by adjacent walls on both sides shall be 1 m; otherwise the minimum width shall be 1.25 m. However, depending upon the frequency of pedestrian users the recommended minimum width for footpath or walkway shown in Table 3.F.2 may be followed.

F.4.4 All public transport terminal and stoppages shall have dedicated planning for pedestrian users to and from the facilities showing connection to the public pedestrian and vehicular network adjacent to the site. Pedestrian walkways or footpaths in all such facilities shall be of sufficient width to cater the pedestrian need of the facility.

Table 3.F.2: Recommended minimum width of pedestrian walkway based on frequency of use

Peak pedestrian frequency	Width of walkway	Width for street furniture and plantation	Total recommended width
(pedestrian user per minute)	(m)	(m)	(m)
Up to 60	2.5	1.5	4
Above 60- 80	3.25	1.5	4.75
Above 80- 100	4.0	1.5	5.5
Above 100	5.0	1.5	6.5

F.5 Guidelines For On-Street Parking

For on-street car parking in any new settlement, the guidelines of Table 3.F.1 shall be the minimum requirement depending upon the type of road and expected traffic density.

F.5.1 Parking along Primary Road

For primary roads, fast moving uninterrupted traffic flow needs to be ensured. Therefore such roads shall not serve for on-street parking. If, parking besides such primary roads becomes necessary, then an additional carriageway with on-street parking and an additional pedestrian walkway as shown in Figure 3.F.6 may be planned, keeping the main traffic flow uninterrupted.

F.5.2 Parking along Non-Primary Roads

On-street parking should normally only be considered on local distributors and roads lower in the hierarchy. On such roads, on-street parking spaces may be provided where off-street facilities are inadequate to meet demand and where provision would not adversely affect the flow of traffic. On-street spaces should generally cater for short term parking needs and parking meters may be installed to encourage such usage.

F.5.3 Parking for Service Vehicles

In most situations, it will not be necessary to provide parking spaces specifically for service vehicles, such as delivery vans, which are normally stationary for a relatively short time. If such parking bays are considered necessary, other vehicles may need to be prevented from using the spaces by regulation and enforcement.

F.5.4 Omission and Conversion of Existing Parking

Omission and conversion of existing parking spaces shall not be permitted if it results in parking deficiency for the occupancy type. That is, after omission and conversion, the remaining number of parking spaces must be sufficient to meet the minimum requirement of the existing, proposed and approved development.

F.5.5 Parking Design Considerations

Any parking space design will consider the following two factors :

(a) Minimum parking requirement ascertained for each type of occupancy

(b) Parking layout for the required number of parking (Figures 3.5.7 to 3.F.13)

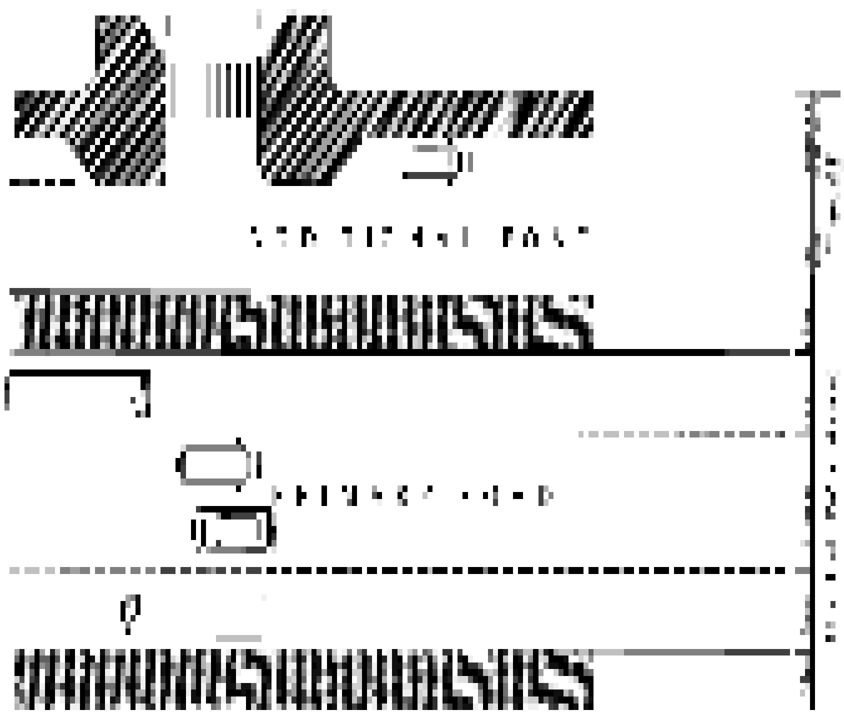


Figure 3.F.6 Provision for on-street parking beside primary road

F.6 Minimum Requirement For off-Street Parking

The number of minimum parking spaces required shall be based on the total floor area of the building and shall depend on its occupancy type and number of users. The following tables (Table 3.F.3) shall form the basis for computation of minimum parking requirement:

Table 3.F.3: Off-street parking requirements for different occupancies

Occupancy type/use		Minimum off-street parking requirements
Residential (occupancy type-A)	Residential (occupancy type-A) Small private dwellings/ row house with plot size not more than 134 m²	1 car parking
	Small private dwellings/ row house with plot size 134-268 m²	2 car parking
	Multi-family housing with flats/ apartments with gross area more than 200 m²	1 car parking/unit+5% guest parking
	Flats/ apartments with gross area more than 140 m² to 200 m²	2 car parking per 3 units
	Flats/ apartments with gross area more than 90 m² to 140 m²	1 car parking per 2 units
	Flats/ apartments with gross area more than 60 m² to 90 m²	1 car parking per 4 units
	Flats/ apartments with gross area up to 60 m²	1 car parking per 8 units
	Flats/ apartments with gross area up to 90 m² (in addition to required car parking)	1 motorcycle parking per 5 units
	Hotels (star category)	1 car parking per 5 guest rooms
	Hotels (other category)	1 car parking per 200 m² gross area
	Others	1 car parking per 300 m² gross area
	The minimum parking requirements for all residential occupancies within 500 m radius of MRT or BRT stations shall be 25 percent of the calculated requirement from above.	
(Oc cup anc	Educational (Occupancy type-B)	1 car parking per 200 m² gross area

Occupancy type/use		Minimum off-street parking requirements
	Kindergarten, primary schools, high schools, colleges, tertiary educational institution, training centers, universities and other educational institutions.	For plots with 25 m or more frontages, an uninterrupted dropping bay of at least 25 m length and 4.25 m width shall be given at ground level within the school premises. For plots with less than 25 m frontage, an uninterrupted dropping bay with length equal to total frontage of the plot and 4.25 m width shall be given at ground level within the school premises. At primary and secondary schools there should be a minimum of 3 lay-bys for school buses within the school boundary.
Institutional and Health care Type (Occupancy type-C and D)	Institutional Type (Occupancy type-C)	1 car parking per 200 m² gross area
	Hospitals, clinics (Occupancy type-D)	1 car parking per 5 beds
	Medical diagnostic centers	1 car parking per 100 m² gross area
	Others (outdoor treatment facilities, collective practice of physicians etc.)	1 car parking per 200 m² gross area
Business and Mercantile (Occupancy type-E and F)	Mercantile (Occupancy type-F) Shops, department store	1 car parking per 200 m² gross area 1 loading/ unloading bay of heavy goods vehicle per 2000 m² gross area or portion thereof
	Restaurants	1 car parking per 100 m² gross area
	Business (Occupancy type E) and Offices	1 car parking per 200 m² gross area
	Others	1 car parking per 200 m² gross area
Industrial (Occupancy type-G) and Storage (Occupancy type-H)	Industries (Occupancy type-G) Storage (Occupancy type-H)	For all such installations, at least 1 truck parking along with loading unloading bay and at least 1 car parking. For administrative or sales centers within these installations, 1 car parking per 200 m² for such parts only are required.

Occupancy type/use			Minimum off-street parking requirements
Assembly (Occupancy type- I)	Assembly (Occupancy type- I) Cinema		1 car parking per 40 seats
	Theatre, auditorium		1 car parking per 20 seats
	Sports facilities		1 car parking per 200 seats
	Transportation terminals, airports, railway stations, etc. (Occupancy I or MIXED)		1 car parking per 50 m² gross area
	Wedding/ party center (Occupancy I or MIXED)		1 car parking per 25 m² gross area. For plots with 25 m or more frontage, an uninterrupted dropping bay of at least 25 m length and 4.25 m width shall be given at ground level within the school premises.
	Religious structure	Up to 300 m²	At least 1 car parking
		More than 300 m²	1 car parking per 50 m² gross area
	Others		1 car parking per 200 m² gross area
Hazardous (Occupancy type-J)	Hazardous (J1 and J2)		For all such installations, at least 1 truck parking along with loading unloading bay and at least 1 car parking. For administrative purpose within these installations, 1 car parking per 200 m² for such parts only are required.

Note:

For mixed-use situation, parking requirement shall be calculated by adding up the individual parking requirements of each types based on their use area per floor and respective parking ratio for each type.

For different types of flats/apartments within the same complex, parking requirements shall be determined by determining requirement for each type separately and then adding them together.

Fractional results in parking calculation shall be considered as 1 (one) full parking space.

With recommendation from the permitting authority parking requirement for low income residential areas may be reduced.

For flats with area less than 90 m², parking requirement of 1/3rd requirement of cars can be calculated by combining cars and motorcycles in the ratio of 1 car to 2 motorcycles.

For any building type, at least 1 (one) car parking shall be required.

F.7 Car Parking Layout Guidelines

F.7.1 Parking Stalls

Parking stall is a rectangular space with defined length and width, where the length is subject to variation depending on its relationship with the aisle, Figure 3.F.7.

For perpendicular or angular parking, the minimum dimensions required of a car parking stall shall be:

Stall width: 2400 mm

Stall length: 4800 mm

For parallel parking minimum dimensions required of a car parking stall shall be:

Stall width: 2000 mm

Stall length: 6000 mm

For parallel parking, where cars cannot be parked by reversing, minimum stall length shall be 7200 mm; the floor of each stall shall be flat and free from curbs and other interferences.

Where parallel parking stalls have frontal obstruction or perpendicular parking stalls have side obstruction, the stall sizes will vary in accordance with guidelines of Figure 3.F.8.

F.7.2 Minimum Width of Driveway

The minimum width of parking aisle or driveway shall follow the requirements of Table 3.F.4.

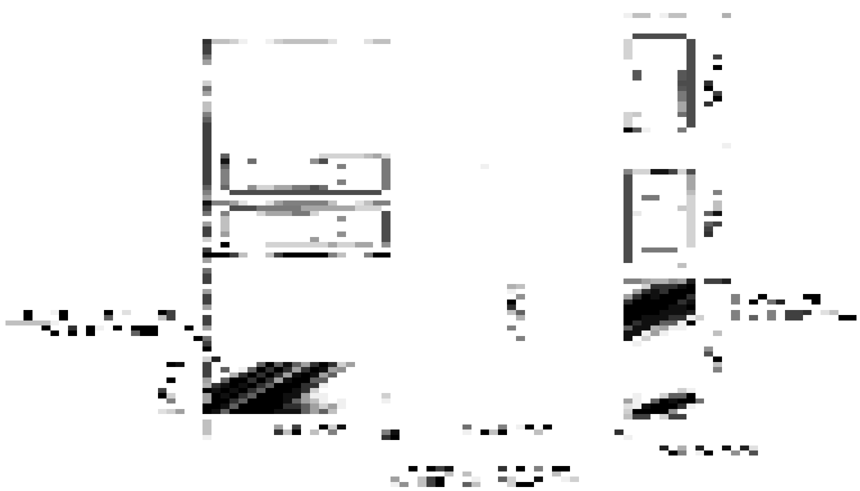


Figure 3.F.5 Parking stall requirements

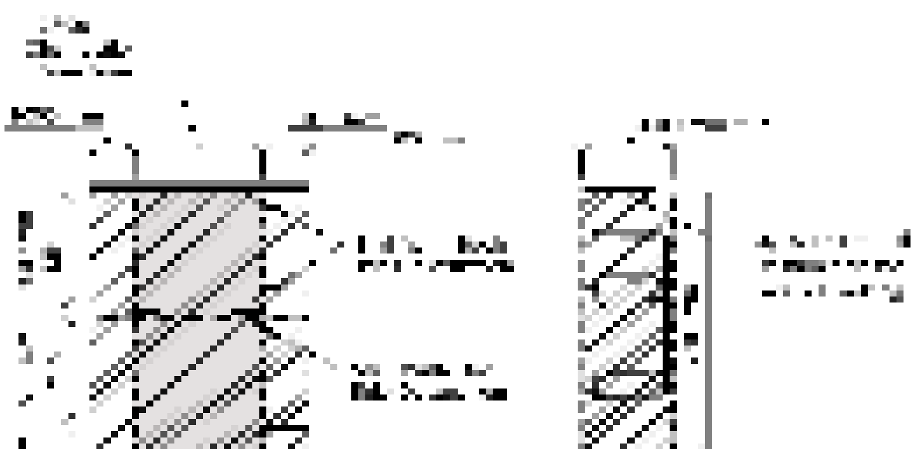


Figure 3.F.8 Parking stall size variation due to obstructions

Table 3.F.4: Minimum width of Parking Aisle/Driveway

Parking Angle to Aisle	One Way Traffic		Two Way Traffic
	Bay on One Side	Bays on Both Side	Bays on One or Both Side
0° (parallel)	3600	3600	6000
30°	3600	4200	
45°	4200	4800	
60°	4800	4800	
90° (perpendicular)	6000	6000	

F.7.3 Minimum Dimension for Carriageway Ramps

Carriageway ramps are sloped driveway connecting and providing access between two levels for vehicles. For safe maneuvering of vehicle on carriageway ramp guidelines of the subsections as under shall be followed.

F.7.3.1 Width of carriageway ramp

The width of a carriageway ramp shall comply with the guidelines of Table 3.F.5 and Figure 3.F.9.

Table 3.F.5: Minimum width of Carriageway Ramps

Type of Carriageway	Single	Dual Lane

Ramp	Lane		
(a) Width of straight carriageway ramp	3600 mm	6000 mm (3000 mm/lane)	
(b) Width of lane of curved carriageway ramp	4200 mm	3600 mm for inside lane	3300 mm for outside lane
(c) Inside radius of curved carriageway ramp	4500 mm		

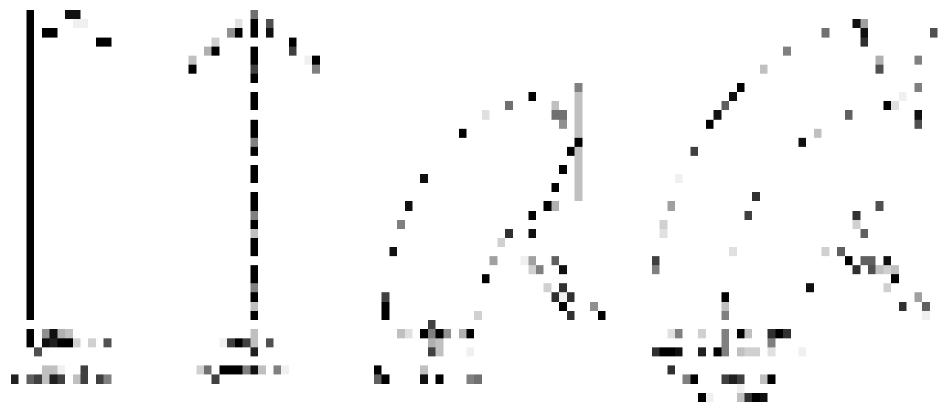


Figure 3.F.9 Carriageway ramp width

F.7.3.2 Carriageway ramp gradient

Ramp gradient specify the slope of a ramp expressed either in percentage or in ratio and calculated as follows:

$$\text{Ramp gradient (slope)\%} = \frac{\text{Floor-to-floor height} \times 100}{\text{Ramp length along the horizontal plane}}$$

For same gradient on ramps, as shown in Figure 3.F.10 (a), the maximum slope shall be 12.5 percent (or 1 : 8). Ramp gradient shall be measured along the center line of the ramp.

For change of gradient on ramp, as shown in Figure 3.F.10 (b), this slope may be increased up to maximum 20 percent, with transition slopes on both end that are sloped at half of the slope of the main ramp.

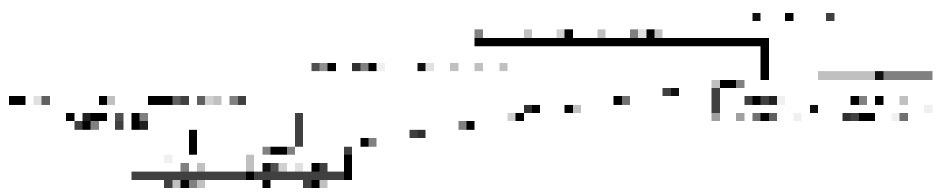
F.7.3.3 Parking ramp

When sloped parking stalls are directly approached from a same sloped ramp it is known as parking ramp and the maximum gradient of such ramps shall be 5 percent (or 1 : 20).

F.7.4 Minimum Headroom

The height clearance from parking level floor to the bottom of the ceiling above shall be 2400 mm minimum. However, for downward projection from overhead ceiling (e.g. beams, direction signs, sprinkler heads, electrical fittings etc.) the clearance shall be minimum 2200 mm. Figure 3.F.11 shows the variable gradient of ramps used for calculating changing gradients.

(a)



(b)

Figure 3.F.10 Ramps with (a) Same gradient; (b) Change in gradient

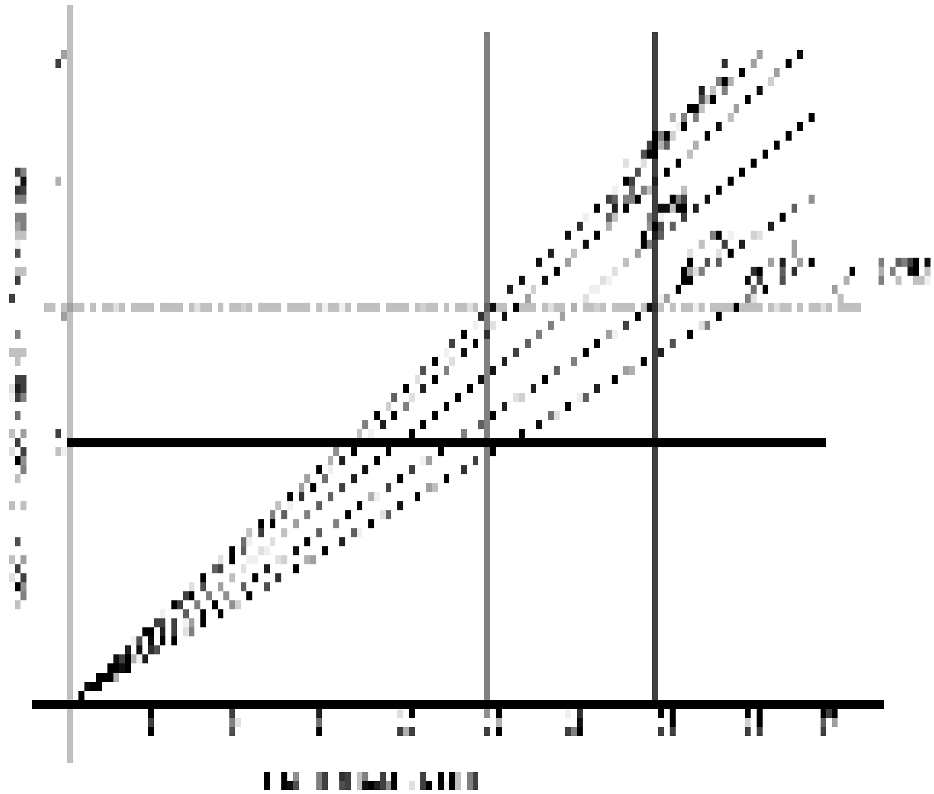


Figure 3.F.11 Relationship between floor-to-floor height, ramp gradient and ramp length

F.8 Motorcycle Parking Provisions

F.8.1 Motorcycle Parking Stall Dimensions

The minimum stall length and stall width of motorcycle parking shall be 2400 mm and 1000 mm respectively.

F.8.2 Stall Location

Motorcycle parking stalls can be provided at corners or any available space within the parking area provided that they do not obstruct movement of other vehicles and pedestrians.

F.9 Provisions For Large Vehicles

For other vehicles the minimum dimension of parking stall, headroom clearance, carriageway width and turning radius shall be in compliance with Table 3.F.6.

Table 3.F.6: Minimum Requirements for Large Vehicle Parking and Maneuvering

<table>
<tr><th>Type of Vehicle</th><th>Stall Length</th><th>Stall Width</th><th>Headroom clearance</th><th>Minimum carriageway width</th><th>Inside turning radius</th><th>Maximum ramp gradient ratio</th></tr>
<tr><td>Light goods vehicles e.g. pickup, vans etc.</td><td>7 m</td><td>3.5 m</td><td>3.6 m</td><td>4.5 m (single straight lane)
5.5 m (single curve lane)
7.4 m (dual straight lane)</td><td rowspan="5">6.0 m</td><td rowspan="4">1:12</td></tr>
<tr><td>Mini buses</td><td>8 m</td><td>3.0 m</td><td>3.3 m</td><td rowspan="3">4.5 m (single straight lane)
7.5 m (single curve lane)
7.4 m (dual straight lane)</td></tr>
<tr><td>Buses</td><td>12 m</td><td>3.5 m</td><td>3.8 m</td></tr>
<tr><td>Heavy goods vehicle e.g. trucks</td><td>11 m</td><td>3.5 m</td><td>4.7 m</td></tr>
<tr><td>Articulated vehicles e.g. container carriers, trailers etc.</td><td>16 m</td><td>3.5 m</td><td>4.7 m</td><td>4.5 m (single straight lane)
9.0 m (single curve lane)
7.4 m (dual straight lane)</td><td>1:15</td></tr>
</table>

F.10 Provisions For Car Lift and Mechanized Parking

F.10.1 General

To connect between different levels with vehicular access, car lifts can be used instead of a carriageway ramps. However, where a site is so constrained that it is not technically feasible to place a conventional ramp to connect between levels, mechanized parking may be installed. All such parking shall require queuing space as per provisions of this Code.

F.10.2 Guidelines for car lifts in a parking

A car lift shall have two openings, allowing entry of a car from one direction and exit of the car to the opposite direction. After entry to a facility or a building having car lifts, a minimum queuing space for at least 15 percent of the total parking shall be provided within the site. On departure from site, at least one holding bay having equal space of a parking stall shall be provided within the site (Figures 3.F.12 and 3.F.13).

The internal dimension of all such lifts shall not be less than 2600 mm x 6200 mm with a minimum discharge capacity of 30 vehicles per hour. For every 50 vehicles 1 car lift shall be installed. To reduce queue at least two lifts shall be installed in a facility or a building. Maximum number of parking using car lifts shall not exceed 200.

F.10.3 Guidelines for Mechanized Parking

A mechanized parking may involve stacking system or lateral displacement system or a combination of both. The approach driveway width for a mechanized parking shall be at least 3.6 meter for one-way traffic and 6.0 meter for two-way traffic. After entry to a facility or a building having mechanized parking, a minimum queuing space for at least 5 percent of the total parking shall be provided within the site.

Mechanized parking varies widely in type and specification; and shall be installed according to its manufacturer's specifications. In doing so and during its operation it shall not compromise the safety of the building or the users in any way.

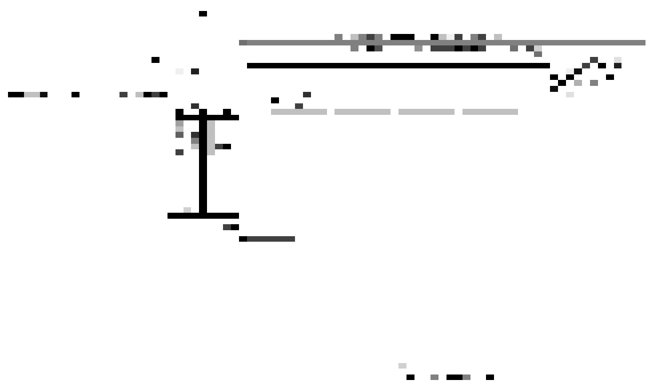


Figure 3.F.12 Queuing space and loading from and unloading to same road

Figure 3.F.13 Queuing space and loading from and unloading to different roads

PART IV
Chapter 1
General Provisions

1.1 Scope

This Part of the Code prescribes regulations for safeguarding life and property in the use or occupancy of buildings or premises from the hazards of smoke and fire, and explosions. The provisions of this Part include general requirements of fire protection, precautionary requirements, means of egress, equipment and in-built facilities standard installations required for firefighting, and firefighting arrangements required for all occupancy groups.

1.2 Terminology

This Section provides an alphabetical list of the terms used in and applicable to this Part of the Code. In case of any conflict or contradiction between a definition given in this Section and that in Part 1, the meaning specified in this Part shall govern for interpretation of the provisions of this Part.

ALARM CONTROL UNIT	It consists of a circuit, controls, relays, switches and associated system which receive signals from alarm initiating devices and transmit to alarm signaling devices.
ALARM INITIATING DEVICE	An equipment operated manually or automatically which, when activated, initiates an alarm through an alarm signaling device.
ALARM SIGNAL	Signals of audible or visual in nature, indicating the existence of a fire and/or smoke condition. Audible devices may be bells, horns, chimes, speakers or similar devices. Visual Alarms is a strobe light emitting bright white light with approved insanity.
ALARM SIGNAL DEVICE	The equipment that produces the alarm signal.
ALARM SYSTEM	It is a combination of compatible devices, which when activated with necessary electrical energy can produce an alarm in the event of fire.
ALARM ZONE	It describes a defined area of the building or buildings for alarm initiating locations.

ANNUNCIATOR	Equipment capable of indicating the zone or area of a building from which an alarm has been initiated or the location of such devices and the operational condition of alarm circuit of the system.
AUTOMATIC FIRE DETECTING AND ALARM SYSTEM	These include all types of fire detecting and alarm signaling devices which activate themselves during a fire without manual intervention. The equipment/devices include temperature sensitive fuses, thermostat, fluid filled tubes and electronic devices which can detect a fire and transmit automatic alarm signals.
AUTOMATIC SPRINKLER SYSTEM	The system consists of an array of pipe-works fitted with fusible solder or glass bulb. This system shall activate at a predetermined temperature and the required water shall be fed to the system from any source. In the event of fire or smoke the system shall activate automatically by sensing the temperature of fire and discharge water to extinguish. These devices also actuate an audible alarm automatically.
AUTOMATIC HIGH VELOCITY WATER SPRAY SYSTEM	This system applies water in the form of a conical spray consisting of droplets of water discharged at high velocity through specially designed projectors to extinguish fire by emulsification, cooling and smothering.
BUILDING	Any structure used or intended for supporting or sheltering any use or occupancy.
BUILDING, EXISTING	A building erected or officially authorized prior to the effective date of the adoption of this edition of the Code by the agency of jurisdiction.
CARBON DIOXIDE EXTINGUISHING SYSTEM	This installation consists of a group of one or more cylinders of carbon dioxide, interconnected by a manifold and feeding into a system of high pressure distribution pipe work fitted with special discharge nozzles.
COMBUSTIBLE MATERIAL	Any material which burns and enhances the magnitude of fire.
DRY-CHEMICAL EXTINGUISHING SYSTEM	This system consists of specially designed pipe works and discharge nozzles linked to the dry powder containers and gaseous cylinders which are automatically/manually operated in case of fire.

DRY RISER	A riser or standpipe system is normally kept empty of water, but is capable to discharge water within 45 seconds and its installation is equivalent to wet-riser system.
ELEVATOR EVACUATION SYSTEM	A system, including a vertical series of elevator lobbies and associated elevator lobby doors, an elevator shaft(s), and a machine room(s), that provides protection from fire effects for elevator passengers, people waiting to use elevators, and elevator equipment so that elevators can be used safely for egress.
ELEVATOR LOBBY	A space from which people directly enter an elevator car(s) and to which people directly leave an elevator car(s).
EXTERIOR STAIRWAY	A stairway in which at least one side have openings more than 50% in an Exterior wall in such a way that there shall be no accumulation of smoke during fire.
FIRE BARRIER	A fire-resistance-rated wall inside a building, designed to restrict the spread of smoke and fire. Opening in that wall, shall be protected by fire protected doors or windows.
FIRE COMPARTMENT	A space within a building that is enclosed by fire barriers on all sides, including the top and the bottom to limit the transfer of fire.
FIRE DAMPER	A device installed in air ducts or air transfer openings or any openings designed to close automatically upon detection of fire or smoke.
FIRE DOOR	See Fire door assembly.
FIRE DOOR ASSEMBLY	Any combination of door leaf, frame, hardware and all other accessories that together provide a specific degree of fire and smoke protection to the opening where it is placed.
FIRE RESISTANCE RATING	It expressed as a period of time and denotes the property of a building construction material or elements and/or construction as a whole during which the materials or elements or constructions are (a) resistant to collapse due to fire, (b) resistant to flame penetration and (c) resistant to excessive temperature rise to the unexposed surface.
FIRE SEPARATION	Refers to a fire-resistance-wall or slab between two buildings or two spaces to protect spread of smokes or fire vertically and horizontally.

FIRE TOWER	Refers to a stairway open or enclosed, detached and isolated from any building by a distance and can be approached from various floors of a building or buildings by connecting passage only.
FLOOR AREA, GROSS	The floor area within the inside perimeter of the outside or exterior walls of the building under consideration with no deduction for hallways, stairs, closets, thickness of interior walls, columns, or other features. Gross floor area of a building means summation of gross floor areas of all the floors of a building.
FLOOR AREA, NET	The floor area within the inside perimeter of the outside or exterior walls of the building under consideration with deduction for hallways, stairs, closets, thickness of interior walls, columns, or other features or spaces not used for human occupancy.
FOAM EXTINGUISHING SYSTEM	This system discharge foam to extinguish special fires.
HORIZONTAL EXIT	Crossing a fire barrier of a building or connecting building in the same level shall be treated as horizontal exit.
INTERIOR STAIRWAY	A designated area on ground or on water or on a portion of a building for helicopter landing or takeoff without servicing, repairing and refueling facilities.
INTERIOR STAIRWAY	A stairway within a building envelope.
PARTY WALL	A fire resistance rated wall where openings are protected, which is constructed from the ground level and continued up to at least 1m above the roof of a building to restrict the spread of a fire.
PUBLIC WAY	A Street, alley, or other similar parcel of land essentially open to the outside air deeded, dedicated, or otherwise permanently appropriated for building users or for public use or a single loaded corridor that is one lateral side opened to outer air, designed in such a way that there shall be no accumulation of smoke in case of fire. This corridor may be placed at any level of a building having a clear width and height of not less than 3 meter having guards and connected to the exit termination or refuge areas by exterior or enclosed stairs shall be treated as public way.

ROOF REFUGE AREA	When occupants are relocated at the flat roof of a building which are not connected with any means of exit shall be treated as isolated refuge area and must have provisions for placing of leaders of fire department excess vehicles.
SMOKE DETECTOR	A devise capable of sensing visible or invisible particles produced during combustion.
TRAVEL DISTANCE	Straight line distance between the remotest point of a space of a floor and the exit access door placed thereof.
TRAVEL PATH	Length of a passage from the remotest point of a space up to the exit access door placed thereof.
VENTILATION	Natural or mechanical intake of fresh air from outside and removal of inside air of an enclosed space.
VESTIBULE	A compartment provided with two or more doors with smoke lock system where the intended purpose is to prevent continuous and unobstructed passage by allowing the release of only one door at a time.
VENT, FIRE	A system which activates itself automatically or manually during a fire or can be activated manually to release the heat and smoke generated by the fire and smoke.
RAMP	A walking surface that has a slope steeper than 1 in 20 and accessible ramps are not steeper than 1 in 12.
WET-CHEMICAL EXTINGUISHING SYSTEM	A system where a solution of water and potassium carbonate and/or potassium acetate based chemical forms the extinguishing agent.
WET RISER STAND PIPE SYSTEM	A vertical pipe or consists of an array of pipes installed vertically in a building having landing valves with appropriate outlets at various levels of a building containing charged water at a specified pressure for fire extinguishing purposes.

1.3 General Requirements

The provisions of this Section shall specify the general requirements in respect of height and area limitations, open space requirements and access facilities for the fire service, which are to be provided for a building to protect it from potential fire hazards.

1.3.1 Height and Area Limitations

The height and area limitations of all buildings and structures shall be governed by the occupancy group classification, floor area ratio and type of construction, which are specified in Part 3 of this Code.

1.3.2 Open Space or Fire Separation Requirement

For the purpose of applying the provisions of open space or fire separation requirements of a building at its side, rear and frontages in Part 3 of this Code shall be followed.

1.3.3 Access Facilities for Fire Service

The access facilities for fire service vehicles and engines shall meet provisions provided in Part 3 of this Code.

1.4 Fire Drill

Fire drills based on fire order shall be arranged to train the occupants of a building in first-aid firefighting, relocation and orderly evacuation. The occupants shall be made thoroughly conversant with fire order, firefighting, and relocation and evacuation procedures in the event of an emergency. The guidelines of fire drill, relocation and evacuation procedure are given in Appendix A.

1.5 Fire Tests and Fire Resistance Rating

The fire resistance rating of individual building construction components shall be determined by standard materials testing procedure as detailed below.

(a) The fire resistance ratings of building assemblies and structural elements shall be determined in accordance with ASCE 29 or ASTM E 119.

(b) The construction materials which are intended to be classified as non-combustible shall be tested in accordance with ASTM E 136.

(c) Flame resistance rating of all materials used for interior finish and trim shall be tested in accordance with ASTM E 84.

(d) The fire door assemblies shall conform to the test requirements of ASTM E 152.

(e) The fire windows and fire shutters shall meet the test requirements of ASTM E 163.

(f) The fire resistances rating of structural elements are provided in Table 4.1.1. For details refer to ASCE 29.

Table 4.1.1: Fire Resistance of Structures

Concrete Aggregate Type	Minimum Equivalent Thickness of Concrete Walls, Floors, and Roofs for Fire Resistance Rating									
	1 hr		1.5 hr		2 hr		3 hr		4 hr	
	in	mm	in	mm	in	mm	in	mm	In	mm
Siliceous	3.5	89	4.3	109	5.0	127	6.2	157	7.0	178
Carbonate	3.2	81	4.0	102	4.6	117	5.7	145	6.6	168
Sand-light weight	2.7	69	3.3	84	3.8	97	4.6	117	5.4	137
Lightweight	2.5	64	3.1	79	3.6	91	4.4	112	5.1	130

Concrete Aggregate Type	Minimum Column Dimension for Fire Resistance Rating									
	1 hr		1.5 hr		2 hr		3 hr		4 hr	
	In	mm	in	mm	in	mm	in	mm	in	mm
Siliceous	8	203	9	229	10	254	12	305	14	356
Carbonate	8	203	9	229	10	254	11	279	12	305
Sand-light weight	8	203	8.5	216	9	229	10.5	267	12	305

Clay Masonry Unit	Minimum Required Equivalent Thickness of Masonry for Fire Resistance Rating													
	0.5 hr		0.75 hr		1 hr		1.5 hr		2 hr		3 hr		4 hr	
	in	mm	in	mm	in	mm	in	mm	in	mm	in	mm	in	mm
Brick of clay or shale, unfilled	1.7	43	2.0	51	2.3	58	2.85	72	3.4	86	4.3	109	5.0	127
Brick of clay or shale, grouted or filled with perlite, vermiculite, or expanded shale aggregate	2.3	58	2.65	67	3.0	76	3.7	94	4.4	112	5.5	140	6.6	168

1.6 Related Appendix

Appendix A Fire Drill and Evacuation Procedure

PART IV
Chapter 2
Precautionary Requirements

2.1 Occupancy Classification

All buildings shall be classified according to their use or by considering the character of their occupancy. For the purpose of this Code, the occupancy classification groups shall be as follows:

Occupancy A:	Residential
Occupancy B:	Educational
Occupancy C:	Institution for care
Occupancy D:	Health Care
Occupancy E:	Business
Occupancy F:	Mercantile
Occupancy G:	Industrial
Occupancy H:	Storage
Occupancy I:	Assembly
Occupancy J:	Hazardous
Occupancy K:	Garages
Occupancy L:	Utilities
Occupancy M:	Miscellaneous

The details of occupancy classification of buildings are provided in Part 3 of this Code.

2.2 Classification of Construction Types

For the purpose of this Code, every room or control area or space of a building or a building itself hereafter altered or erected shall be classified in one specific type of construction as grouped as follows:

GROUP I- Non-combustible, subdivided as follows:

Type- I A	4 hour protected
Type- I B	3 hour protected
Type- I C	2 hour protected
Type- I D	1 hour protected
Type- I E	Unprotected

GROUP II- Combustible, subdivided as follows:

Type- II A	Heavy timber
Type- II B	Protected wood joist
Type- II C	Unprotected wood joist
Type- II D	Protected wood frame
Type- II E	Unprotected wood frame

The types of construction are based on fire resistance of construction elements, which are detailed in Part 3 of this Code.

2.3 Fire Zones

The development areas of a city, township or municipality or union shall be divided into Fire zones as distinct areas based on the inherent fire hazards of the buildings to be constructed and the degree of safety desired for the occupancy group accommodated therein.

2.4 Mixed or Separated or Detached Occupancy

Where two or more occupancy types are amalgamated in a floor or in a building shall be designated as mixed occupancy shall be allowed as per provisions of A-Z list of Part 3 and this Code.

Where two or more occupancy types are in groups in a floor or in a building and separated as specified in the Table 3.2.1 of Part 3 and as per provisions of this Code shall be designated as separated Occupancy.

Hazardous occupancy J shall not be allowed as mixed or separated occupancy with any other occupancy classification as per provisions of this Code.

Building structures are isolated by fire separation distances as per provision of this Code shall be designated as detached occupancies.

2.5 Openings In Separation Wall

Opening means a hole or an aperture in the building envelope or in any wall within the building through which air can pass. Protective type opening means a hole or an aperture shall have open able closures with fire resistive assemblies to restrict air movement.

Separation wall not constructed monolithically or homogeneously and having joints shall be complied with requirements of smoke lock and fire resistance rating as per provisions of this Code.

Vertical solid elements which create a barrier within a space or create a building envelope shall be designated as wall or partitions as per provisions of this Code.

(a) The openings in occupancy separation wall shall conform to the provisions set forth in the Part 3 of this Code.

(b) Openings in fire separating walls and floors shall not exceed the approved limit and the opening shall be of protective type and conform to the approved provisions of this Code.

(c) Fire separation walls shall not have opening exceeding 11.2 m^2 in area and the aggregate width of all openings at any floor level shall not exceed 25 percent of the length of the wall. When an entire storey floor area has fire separation walls on two opposite sides have openings shall be covered by automatic fire suppression system, the maximum allowable opening may be doubled with a minimum distance of 0.9 m between adjacent openings.

(d) Each protected openings in a fire separation wall shall be limited to 5.6 m^2 in area with a maximum height of 2.75 m and width of 2.20 m. Wall or floor openings shall be protected with approved fire resisting means conforming to approve standards as per provision of this Code. When openings in floors have protected enclosures or have enclosure walls which form a shaft and have openings on enclosure wall shall be protected by fire assemblies.

(e) Openings of service lines like cables, electrical wirings, telephone cables, plumbing fixture etc. shall be protected by enclosures having an approved fire resistance rating. Medium or low voltage electrical wire running through shaft or ducts shall be either armoured or cased within metal conduits as per provisions of Part 8 of this Code.

(f) All openings in the fire separation walls shall be protected with fire resistance assemblies or automatic fire suppression system as per provisions of this Code.

2.6 Smoke and Heat Vents

Interior or indoor air qualities are maintained as good as natural outdoor air qualities as per provisions of this Code through openings in the building envelope shall be designated as Natural Ventilation.

Interior or indoor air qualities are maintained by the means of mechanical devices shall be designated as Mechanical Ventilation. Restricted ventilation means excessive smoke accumulation within a building during fire.

(a) Smoke and heat vents shall be installed in areas of restricted ventilation such as windowless buildings, underground structures, and factories floor spaces of restricted ventilation.

(b) Where exit access travel distance is more than 23 m, smoke and heat vents shall be constructed in accordance with the provisions of this Code.

(c) The vent area and spacing of the vents shall comply with Table 4.2.1.

(d) Closures of natural draft, smoke and heat vents shall be installed in such a way that fire service personnel can open it easily during a fire.

(e) Smoke and heat vents on roof or ceiling or wall shall normally be kept open. In case of closed vents, automatic activation of the openings by heat responsive device rated at 38^o C to 104^o C above ambient shall be a requirement. The releasing mechanism shall be capable of opening the vent fully when the vent is exposed to a time-temperature gradient that reaches an air temperature of 260^o C within 5 minutes. The vents shall also be capable of being opened by manual operation.

(f) Fire Vents requirements for Industrial and Storage Buildings are given in Appendix B of Part 4.

Table 4.2.1: Smoke and Heat Vent Size and Spacing

Use group	Hazard Condition	Vent Area to Floor Area Ratio	Max Spacing of Vent Centres
Occupancy H1	Low Hazard	1:150	45 m
Occupancy H2	Moderate Hazard	1:100	36 m
Occupancy J1	High Hazard	1:30 to 1:50	22.5 m to 30 m
Occupancy J2, J3, J4	High Hazard	1:30 to 1:50	22.5 m to 30 m
Occupancy K1, K3	Low Hazard	1:150	45 m

2.7 Electrical, Gas and Hvac Services

The requirements of the electrical, HVAC and gas services shall meet the provisions of Part 8 of this Code.

(a) Air-conditioning and ventilation systems shall be installed and maintained as per provisions of this Code so that the fire, fumes or smoke do not spread from one area of fire to other area of a building through the ducts or vents.

(b) Properly designed fire dampers shall be installed within the air-conditioning and ventilation ducts, which shall automatically close the flow of air in case of fire.

(c) For large assembly areas, department stores and hotels with more than 100 rooms in a single block, effective means for preventing circulation of smoke through the air-conditioning ducts shall be installed. Such means shall consist of approved smoke sensing control devices, where fuses of dampers may not function during early state of a fire due to insufficient heat as per provisions of this Code.

2.8 Surface Finishes

Materials used to trim or cover the interior and the exterior surfaces of a building have the potential of generating smoke and toxic fumes during a fire and have the potentiality of changing the nature of fire due to its ignitability as fuel. Use of such finish materials shall be classified as per provisions of this Code.

(a) The fire susceptibility of various types of surface finishes shall be determined in terms of the rate of spread of fire (ASTM E 84). Based on the rate of spread of fire, the surface finish materials shall be classified into three (3) classes:

Class I Surfaces of low flame spread: Flame does not effectively spread more than 300 mm in the first 1.5 minutes with an ultimate value of 600 mm.

Class II Surfaces of medium flame spread: Flame does not spread effectively more than 300 mm and 850 mm in the first 1.5 minutes and 10 minutes respectively.

Class III Surfaces of rapid flame spread: Flame spreads effectively more than 300 mm and 850 mm in the first 1.5 minutes and 10 minutes respectively.

(b) Interior finish of walls and ceilings shall have a flame spread rating not greater than those in Table 4.2.2 for various occupancy classes.

Table 4.2.2: Acceptable Flame Spread Rating Classes of Interior Finish

Occupancy Class/Use Group		Vertical Exits and Passage Ways	Corridors Providing Exit Access	Rooms or Enclosed Areas
A1	Detached single family dwelling	III	III	III
A2	Two family dwelling	I	I	II
A3	Flats or Apartment	I	I	I
A5	Hotels and Lodging Housing	I	I	I
B	Educational	I	I	I
C1, C2	Institutional, Residential & custodial	I	I	III
C3	Institutional-Incapacitated	I	I	I
C4	Institutional- Restrained	I	I	I
D	Health Care	I	I	I
E	Business	I	II	II
F	Mercantile	I	II	II
G	Industrial	I	II	II
H	Storage	III	II	III
I1	Large assembly with fixed seats	I	I	I
I2	Small assembly with fixed seats	I	I	I
I3	Large assembly without fixed seats	I	I	I
I4	Small assembly without fixed seats	I	I	I
J	Hazardous	I	II	III

Note: Class III may be adopted Instead of Class II where the area is covered by automatic fire suppression system.

2.9 Glazing Assemblies

(a) Buildings of construction shall use any one of the following types of glazing using wire glass by electro-copper or equivalent. Building of construction types as designated as unprotected or combustible may use hardwood sashes or frames or both.

(b) Glazing system used partially or as a whole to fulfill fire separations or fire barriers requirements as per provisions of this Code shall be the equivalent of required fire resistance rating. Glazed doors, windows or partitions or wall with appropriate smoke lock along with other safety due to fragility, translucency or transparency shall be correctly installed. Such fire-resistant glazing assembly must function as an integral system together with the frame, beads, bead fixings, glazing materials and frame fixings all working together with compatibilities with the standards installation as per provisions of this Code.

(c) Wired glass panels shall comply with the following requirements:

(i) Thickness of the glass shall not be less than 6 mm.

(ii) Embedded wire netting mesh in the glass shall not be more than 25 mm mesh.

(iii) The sashes or frames or both shall be entirely made up of iron or any other approved metal. The frame shall be securely fixed into the wall (except panels of internal doors).

(iv) Setting of the panels of glass shall be achieved by rebates or grooves of not less than 6 mm diameter/width or depth keeping due allowance for expansion. The glass shall be secured to the frame by hard metal fastenings. Approved sealants may be used for weather proofing.

(v) Where wired glass panels are labelled as protective openings, they shall conform to the size limitations shown in the Table 4.2.3.

Table 4.2.3: Limitations of Wired Glass Panel sizes in Protective openings

Required Fire Resistance Rating	Opening Size		
	Max Height (m)	**Max Width (m)**	**Max Area (m^2)**
3 hours	NP	NP	NP
$1\frac{1}{2}$ hour door in exterior walls	NP	NP	NP
$1\frac{1}{2}$ hour fire rating	0.85	0.25	0.065
$\frac{3}{4}$ hour fire rating	1.4	1.4	0.85
Fire windows	1.4	1.4	0.85

Note: Size limitations are not applicable for Fire Rated Glazing Assemblies.
NP = Not Permitted.

(d) Electro-copper glazing shall comply with the following requirements:

(i) Thickness of the glass shall not be less than 6 mm.

(ii) Not more than 0.4 m^2 of square glass shall be formed by electro-copper glazing in sectional lights.

(iii) The sashes or frames or both shall be entirely made up of iron or any other approved metal. The frame shall be securely bolted into the wall (except panels and internal doors).

(iv) Setting of the panels of glass shall be achieved by rebates or grooves of not less than 6 mm width or depth keeping due allowance for expansion. The glass shall be secured to the frame by hard metal fastenings. Approved sealants may be used for weather proofing.

(e) Wall opening more than 5 m^2 shall not be deemed to be effectively protected by wired glass or electro-copper glazing.

(f) Wired glass or electro-copper glazing not exceeding 0.85 m^2 in area shall be allowed provided it is cased in hard metal and secured to the frames by hard metal hinges not exceeding 60 mm apart and by fastening at top, centre and bottom.

2.10 Skylights

(a) Wired glasses used in skylights shall comply with the following requirements:

(i) Thickness of the glass shall not be less than 6 mm;

(ii) Wire netting mesh embedded in the glass shall not be more than 25 mm square;

(iii) The glazing shall be caged in frame of continuous metal divided by bars 750 mm apart centre to centre. The frame and bars shall be iron or other approved metal (or of hard wood covered with sheet metal). The glass shall be secured to the frame by hard metal fastenings. Approved sealants may be used for weather proofing.

(b) Single opening for Skylight more than 5 m^2 shall not be deemed to be effectively protected by wired glass.

2.11 Fire Lifts

(a) Fire lifts shall be installed as per provisions of this Code. Fire lifts, where installed shall be fully automated from the ground level with all though fire rated and protected wiring and switches and shall have a minimum capacity of 8 persons.

(b) Fire lifts shall be operated and maintained by the inmates of building except during fire. During fire, Firemen shall takeover to operate such lifts.

(c) Fire lifts shall be equipped with approved two way voice communication with the fire command station or control room or security room on the exit termination level of a building.

(d) Number and location of fire lifts in a building shall be decided on the basis of total occupant load, floor area and compartment.

(e) A Lift shaft or bank shall be dedicated to Fire lift.

(f) The speed of the lift shall be such that it can reach the top floor from ground level (non-stop) within 1 minute.

(g) Smoke detectors shall be installed at a distance of 3m from every entry doors of Fire Lifts and links with corresponding lift control panel to prevent lift doors to open in case of fire at any level.

(h) All lifts in tall structure shall be operable during fire. There shall be provisions for firemen to take over the control of lift operation as per provision of this Code.

(i) All stretcher and hospital lifts shall be operable during fire. There shall be provisions for firemen to take over the control of lift operation as per provision of this Code.

(j) Lifts installed for accessibility shall be operable during fire. There shall be provisions for firemen to take over the control of lift operation as per provision of this Code.

(k) Lift lobby shall be connected with at least one fire stair by a means of exit component.

2.12 Utilities (Occupancy L) and Exempted Quantities of Hazardous Material

Occupancy type L is a separated occupancy from the main occupancy classifications to provide ancillary electro-mechanical service facilities require a special attention which shall be taken as per provision of this Code. Utilities (Occupancy L) and exempted quantities of hazardous materials for different occupancies are given below:

2.12.1 Occupancy A: Residential

(i) Flammable liquids used for domestic purposes shall be kept adequately sealed in approved containers within the limit of exempted quantity at all times.

(ii) Stoves and heaters using open flame shall be so located at defined space with proper precaution.

(iii) Exhaust fans used in kitchens shall be placed on a peripheral wall of the building or to a duct connected directly to outside and shall be made of non-combustible material. The duct shall not pass through combustible materials.

(iv) Doors leading into a room containing flammable liquids shall be provided with self-closing devices. Appropriate signs identifying the storage materials and requesting the users to keep the door closed shall be marked on both sides of the door.

(v) All outdoor roof top antennas shall be protected by proper lightning arrester.

(vi) Rooms containing boiler shall be separated from the main building by appropriate separation wall with all its openings protected as per provisions detailed in Sec 2.3 of Part 3 and Sec 2.5 of this Chapter.

(vii) Areas or rooms within the building identified as Control Area shall be protected or segregated by appropriate separation wall or by other approved means as per the provisions of this Code.

2.12.2 Occupancy B: Educational

(i) Control areas containing volatile flammable liquids shall be separated from the adjoining areas in as per provisions of this Code.

(ii) Gas pipeline entering any building shall be equipped with shutoff valves outside the building with conspicuous marking clearly delineating the location as per provisions of Part 8 of this Code.

(iii) The openings of boiler rooms shall be adequately protected by fixed, automatic or self-closing fire assemblies.

2.12.3 Occupancy C: Institutional

Permit shall not be granted for storage or handling of any hazardous material even in control areas, except for normal use in amounts not exceeding the exempted amounts specified in Chapter 2 of Part 3, in a building or part thereof classified as Occupancy C.

2.12.4 Occupancy D: Health Care

Storage of volatile flammable liquids such as chloroform, ethyl alcohol, mentholated spirit etc. shall be stored in Control Areas and no unauthorized person shall be allowed to handle such liquids.

2.12.5 Occupancy E: Business

(i) Exit aisles or approaches in self-service in a space shall not be obstructed by placing checkout stand with associated railings or barriers on its passage.

(ii) All operations in open air markets, refuelling stations, road side stands for sale of farm products etc. shall be so conducted that unobstructed access to exits are always maintained.

2.12.6 Occupancy F: Mercantile

Provisions are same as those of Sec 2.12.5 (Occupancy E).

2.12.7 Occupancy G: Industrial

(i) Apparatus are not capable to igniting flammable vapour shall be permitted within a control area of a building using or processing or storing volatile flammable liquid. Control Areas of a building using or processing or storing such flammable liquid shall be covered by exhaust ventilation system.

(ii) Boiler rooms and areas containing heating plants shall be separated from the rest of the occupancy as per provisions of this Code.

(iii) Adequate protective measures shall be taken against hazards associated with distribution and use of electricity and gas in accordance with the provisions of Chapters 2 and 8 of Part 8.

(iv) The machine layout shall be congenial to safe fire practice.

2.12.8 Occupancy H: Storage

(i) Apparatus are not capable to igniting flammable vapour shall be permitted within a Control area or part of a building using or storing volatile flammable liquid. Control Areas of a building using or storing such flammable liquid shall be covered by exhaust ventilation system.

(ii) Boiler rooms and areas containing heating plants shall be effectively segregated from the main occupancy.

(iii) Adequate protection shall be taken against hazards associated with distribution and use of electricity and gas in accordance with the provisions of Chapters 2 and 8 of Part 8.

2.12.9 Occupancy I: Assembly

(i) All materials used for decorative purposes in buildings of Occupancy I shall be non-combustible. If fabrics and papers are used for decorative purposes, shall be treated with flame resistant chemicals/materials.

(ii) Rooms and parts of a building containing high pressure boilers, refrigerating machinery, large transformer or other service equipment having explosion potential shall not be located on or adjacent to the defined exit route. Such rooms shall be effectively cut off from the rest of the building and connected to open air through approved ducts or openings.

(iii) Rooms or parts of a building used for storage of combustible materials such as paints or other items shall be effectively cut off from main assembly building or protected by approved automatic sprinkler system. Such areas shall be away from staircases.

(iv) Legitimate stages having such facilities as fly galleries, gridirons and rigging shall be covered by an automatic sprinkler system above and below such stage areas or spaces. Auxiliary spaces such as dressing rooms, store rooms, and workshops and the proscenium opening shall be effectively covered by fire resistant curtains capable of withstanding a lateral pressure of 4 kN/m^2. The curtain shall be equipped with self-closing emergency device and when closed shall be tight enough to prevent spread of smoke.

(v) Legitimate stage roof above every theatre using movable scenery or motion picture screen constructed of highly combustible materials shall be fitted with ventilators in or above it. The ventilators shall be operable from the stage floor manually or by fusible links or some approved automatic heat actuated device to give an opening to sky with an area of one-eighth the area of the stage.

(vi) In theatres not protected by automatic fire sprinklers, the proscenium wall using movable scenery of decorations shall be provided with maximum of two openings to enter the stage and each opening shall not be of more than 2 m^2.

(vii) Film projection apparatus shall be enclosed within fire resistant enclosures.

(viii) Auditoriums of theatres and cinemas shall be installed with vents on roof having vent area equal to the floor area including balconies and galleries, boxes and tiers. Larger numbers of smaller vents shall be preferable over smaller number of larger vents.

2.12.10 Occupancy J: Hazardous

(i) Equipment and machinery in operations, igniting and/or emitting combustible volatile substances shall be installed in a standard environment as recommended in NFPA or equivalent standards.

(ii) Rooms containing boiler or heating plant shall be effectively separated from the main occupancy.

2.12.11 Occupancy K: Garage

As per safety requirement of NFPA or equivalent standard.

2.12.12 Occupancy L: Utility

As per safety requirement of NFPA or equivalent standard.

2.12.13 Occupancy M: Miscellaneous Buildings

As per safety requirement of NFPA or equivalent standard.

2.13 RELATED APPENDIX

Appendix B Fire Protection Considerations for Venting in Industrial and Storage Building.

PART IV
Chapter 3
Means of Egress

3.1 scope

The provisions of this Section shall control the design, construction and arrangement of building components to provide a reasonably safe means of egress. Any repair or alteration works within a building shall be prohibited unless the existing means of egress and fire protection system are continuously maintained or a continuous alternative exits and protection measures are taken to provide an equivalent degree of safety for the occupant and the workers for the total duration of such project.

3.2 Components of Means of Egress

3.2.1 A means of egress is an evacuation system with the provisions of reentry for rescuers and fire fighters where a continuous and unobstructed way of exit travel shall be provided from any point within a building to a designated area of refuge for allowable delayed evacuation and ended up with the exit termination by reaching a street abutting building or plot or an safe area which is open to air and designated assemblies for evacuees.

The way of exit travel within a building form any point thereof along a means of egress shall consist of three parts: (1) the exit access, (2) the exit, and (3) the exit discharge

(a) A way or path of evacuation from any point of an area affected due to fire incident leads to a protected entry to another separated area of a building shall be termed as exit access. Straight line distance between the remotest point of an area of incident and the entrance point of a separated area shall be measured and termed as a travel distance.

(b) The exit is a component or a group of components start with a protected opening to evacuate an area of fire incidence and provides a safe entry to a separated area which is component of means of egress and subsequently leads to the exit discharge.

(c) The outer edges or peripheral points of a building from where occupants shall evacuate the building envelope termed as Exit discharges which shall lead evacuees to the terminal points at a safe distance from thereof.

An area or any plot abutting street which is open to air and designated for systematic assemblies of evacuees to complete the process of egress system shall be termed as exit termination.

3.2.2 The parts of the means of egress consist of any of the following exit components:

(a) A doorway, separated or refuge area like smoke and fire proof enclosure, compartment, corridor, passage, ramp, balcony, an exterior or open or interior fire stair, or any combination of these, leads orderly to the exit discharge which offer safety from fire or smoke from the area of incidence.

(b) Horizontal exit shall provide a delayed egress by relocating the occupants from their initial location due to a fire incident to a separated area at same level of a same building or at the same level of adjoining or detached buildings connected through a fire door or a vestibule or a passage or corridors for relocation of evacuees. Receiving areas are capable to accommodate expected evacuees for certain time period, free from heat, smoke and aggressive fire, from the area of incidence and shall lead to exit discharges without returning the evacuees to their initial locations.

3.2.3 Generally lifts, escalators and moving walks shall not be regarded as components of means of egress. When they are designed and installed for safe operation during fire shall be included as components of means of egress.

3.2.4 Means of Escape: A way out of a building or structure that does not conform to the formation of means of egress but does provide a safe way out.

3.3 General Requirements

3.3.1 Design considerations or assumptions:

(a) Fire initiated from only one source in single space shall aggravate within a building or adjacent structures over a time period.

(b) More than one space or source of fire at the same time shall not be considered.

(c) All Construction Materials by qualities and quantities including surface finish, utilities, fabrications of movables and immovable, stored materials shall be approved types as per provisions of this Code.

(d) Stability of structural elements or building itself shall be as per provisions of this Code.

(e) Occupants, Rescuers and fire fighters life safety shall be the prime consideration thus egress system including relocation and fight in place or evacuation and reentry provisions shall be as per provisions of this Code.

(f) Fire suppression and extinguishment arrangement for life safety and minimize property damages shall be performance based as per provisions of this Code.

(g) Provisions of this Part shall be the minimum standard, in excess of these provisions shall not be prevented to design a egress system or to install advance and higher standard of detection and extinguishment equipment or both which shall be approved by the authorities having jurisdiction.

3.3.2 All buildings constructed for human occupancy or control areas or storages shall be provided with adequate exit facilities to permit safe and quick unaided egress of the occupants in the event of fire or other emergency.

3.3.3 Exits shall not be used for any other purpose at any time that would obstruct the intended use of those components during emergency.

3.3.4 Where corridors or passages are components of exits shall not be designed or used as components to supply or return air.

3.3.5 Preferences of levels of walking surfaces in the means of egress shall be more than 1 in 20. Ramps or stairway shall be used in case of changes in elevations of walking surfaces.

(a) Abrupt changes not exceeding 130 mm but exceeding 60 mm shall be beveled 1 in 2.

(b) Changes in elevation exceeding 130 mm shall be considered as a change in level.

(c) A stairway in walking surface of the means of egress shall consist of minimum two steps and all of them shall be identical and shall have tread depth not less than 330 mm and height of risers shall not be exceeded more than 230 mm but shall comply tread and riser combination as per provision of this Code.

(d) Changes in levels 530 mm or more in walking surfaces of the means of egress shall be achieved either by a ramp or by a stairway.

(e) Presence and location of such steps or ramps in the walkways shall be readily apparent.

(f) Other than ramp, a slope of walking surfaces along the direction of travel shall not be steeper than 1 in 20 and slope perpendicular to the travel direction shall not be exceeded 1 in 48.

(g) Slope of ramps shall be complied with the accessibility where required as per provisions of this Code.

3.3.6 From the exit access all exits shall be clearly visible. Corridors and passages leading to the exit discharge shall be marked and signposted to guide the evacuees as per provisions of this Code. A space used in darkness having more than one exits shall be illuminated exits sign as per provision of this Code.

3.3.7 The owner or lessee of all new and existing buildings shall be responsible to provide the safety provisions for all occupants and rescuers and firefighters. If in any existing building, the exit facilities are deemed inadequate in view of the requirements of this Code, the authority having jurisdiction may order to comply with the provisions of this Code.

3.4 General Provisions of Exits

3.4.1 All exits shall be easily discernible and accessible from the areas served by them.

3.4.2 Exit from any room or space shall not open into an adjoining or intervening room or area except where such adjoining room or area is an accessory to the area served, is not a hazardous occupancy. If hazardous or a control area, provide a direct exit to the outside of a building envelope or directly connect with the components of egress system.

3.4.3 No portion of Exits shall pass through a room that may be subject to lock with detachable key or be intervened by a door that may have detachable key operated lock and the door is locked when the building is occupied.

3.4.4 All entry points to the assembly occupancy shall serve as Exits and shall have the total capacity for at least one-half of the total occupant load. Provisions of exits other than entries shall have capacity to evacuate at least two-thirds of occupant from each level of assembly occupancy.

3.4.5 All exits shall be so located and arranged that they shall provide continuous and unobstructed means of egress up to the exit discharge.

3.5 Occupant Load

Total occupant load means summation of all occupants of only one level at the pick hour occupancy where maximum occupants are present.

Occupant load shall be considered as per provisions of this Code to design each and every component of means of egress system shall be termed as design occupant load.

3.5.1 Design Occupant Load

The design occupant load for which the component of means of egress is to be provided shall be the highest number computed as per the provisions of (a), (b) and (c) as stated below:

(a) The actual number of occupants for whom the area served by the exits is designed;

(b) Number of occupants shall be computed as prescribed in Table 4.3.1.

(c) The number of occupants in any area shall be computed as per provisions of (a) or (b) as stated above and in all cases the higher value shall govern the design.

(d) The computation of design occupant load shall be the summation of occupants of a space and the evacuees of other spaces whose are using the said space as for waiting or passing through in case of emergency to gain an access to a component of means of egress.

Table 4.3.1: Occupant Load Factor

	Occupancy	Unit of Floor Area in m^2 per Occupant*
A	Residential	18 gross
B	Educational:	
	Class room	2 net
	Preschool	3.5 net
C	Institutional	12 gross
D	Health Care:	
	In patient areas	15 gross
	Out-patient areas	10 gross
I	Assembly:	
	with fixed seats	Number of seats designed.
	with movable seats	0.93 net
	standing space only	0.37 net
	with table and chairs	1.5 net
	Passengers that can be unloaded simultaneously to a terminal or a platform	0.15 net
E	Business: Office Space	3 gross

	Occupancy	Unit of Floor Area in m^2 per Occupant*
F	Mercantile:	
	Retail sales Area, Ground floor or Basement	2.3 net
	All other floor	4.6 net
G	Industrial	10 gross
H	Storage	20 net
K	Garages and open parking structures	23 net
L	Utility	Actual occupant load
M	Miscellaneous Building	Actual occupant load

* As per Sec 3.5.1(b) of this Chapter, design occupant load shall be calculated and any fraction shall be rounded to next higher integer value. Width of all components of egress system shall satisfy requirements of specified in the Table 4.3.2

3.5.2 Fixed Seats

The occupant load for an assembly or educational area having fixed seats shall be determined by the seating capacity of the area. For fixed seats without dividing arms, the capacity shall be taken as one person for every 500 mm of seat.

3.5.3 Maximum Occupant Load

The design occupant load, need not to be calculated more than one person per 0.3 m^2 of usable floor space.

3.5.4 Mezzanine Floors

The occupants of a mezzanine floor evacuating through other floors the occupant load shall be added to the receiving floors.

3.5.5 Roofs

A Roof, an open air space used as assembly or refuge area, educational or other types of human occupancy shall be provided with exit facilities as per provisions of this Code.

3.6 Capacity of Exit Components

3.6.1 The capacity of egress components shall be complied with the occupant load of the area served. The required width of each component shall be computed on the basis of the allotted width per occupant prescribed in Table 4.3.2, subject to the minimum widths of such components specified in Sections 3.7 to 3.12 and the travel distances of such components as per provision of this Code.

Table 4.3.2: Required Width per Occupant

Occupancy		Buildings without Sprinkler System (mm per person)			Buildings thoroughly Sprinkled (mm per person)		
		Stairways	Ramps & Corridors	Doors	Stairways	Ramps & Corridors	Doors
A B E F1, F2 G H	Residential Educational Business Mercantile Industrial Storage	8	5	4	5	4	4
C1, C2	Institutional	8	5	4	5	5	4
C3, C4, C5	Institutional	10	5	4	8	5	4
D	Health Care	25	18	10	15	12	10
I F3	Assembly Mercantile	10	7	5	7	5	5
J	Hazardous	8	5	4	8	5	**4**
K, L, M		8	5	4	5	4	4

Note: width of the components of egress shall be divided by value specified in this table to determine the maximum allowable occupant load served by them.

3.7 Corridors and Passageways

3.7.1 Occupants commencing exit travel along a corridor or a passageway shall be lead to an exit discharge. Length of dead end corridors and passageways and branches thereof shall not be exceeded as per Sec 3.15.4 of this Chapter.

3.7.2 The required width of corridors and passageways shall be calculated on the basis of the occupant load in accordance with the provisions of this Code and shall not be less than as per Sec 3.15.4 of this Chapter.

3.7.3 The minimum ceiling height of the corridors and passageways used as a means of egress shall not be less than 2.4 m.

3.7.4 All exit corridors or passages shall have a fire resistance rating of 1 hour or more as per provisions of this Code.

3.7.5 Protective opening leads to an exit shall be fire doors or fire windows or a fire assembly having a fire resistance rating of at least 20 minutes or more as per provisions of this Code.

(a) Certified Fire resistance rating of Doors shall be in accordance with ASTM E152 without the hose stream test.

(b) Fire resistance rating of the fire door assembly has to perform as required 20, 30, 60, 90, 180 minutes or more shall be leveled A, B, C, D, E and F respectively.

(c) Fire door assembly of any approved materials shall qualify through ASTM E152 without the hose stream test.

3.8 Assembly Seating and Waiting

(a) Assembly buildings primarily meant for theatrical, operatic performances or cinematic projection shall have the seats securely fastened to the floor with exceptions as permitted in this Code. All seats in balconies and galleries shall be securely fastened to the floor except boxes with level floor and less than 14 seats.

(b) Seats not fixed to the floor shall be permitted in restaurants and such other places provided that 1.25 m^2 of floor space is allotted for every seat excluding dancing floor and stage. Adequate aisles shall be maintained at all times to reach exits without obstruction when such occupancies are in use.

3.8.1 Assembly buildings which contain seats, tables, equipment or exhibitions or displays shall be provided with aisles, free of obstructions, leading to the exit.

3.8.2 Minimum clear widths of steeped aisles and other means of egress serving assembly seating shall be calculated on the basis of number of seats and in accordance with Table 4.3.3. Interpolation shall be permitted between the specific values shown thereof. The minimum clear width of steeped aisles as found by above calculation shall be modified in accordance with the conditions stated below:

(i) If risers exceed 178 mm in height for steeped aisles the width of the steeped aisles as shown in the table shall be multiplied by factor *a*,

Where, $$a = 1 + \frac{Riser\ Height - 178}{125} \qquad (4.3.1)$$

(ii) In the Table 4.3.3 values of steeped aisles not having a handrail within a 760 mm horizontal distance shall be 25 percent wider.

(iii) In Table 4.3.3 values of width of ramps used for ascending and steeper than 1 in 10 slope shall be increased by 10 percent.

Table 4.3.3: Capacity Factors for Assembly Seating

Number of seats within a single assembly space.	Clear Width per Seat Served	
	Steeped aisles (mm)	Passageways, Ramps and Doorways(mm)
≤ 2,000	7.6 *a*	5.6
5,000	5.1 *a*	3.8
10,000	3.3 *a*	2.5
15,000	2.4 *a*	1.8
20,000	1.9 *a*	1.4
≥25,000	1.5 *a*	1.1

3.8.3 The minimum width of level or ramped aisles shall be as specified below:

Seats on both sides of the aisle 1.0 m

Seats on one side of the aisle 0.9 m

3.8.4 The minimum width of stepped aisles shall be as specified below:

Seats on both sides of the aisle 1.2 m

Seats on one side of the aisle 1.0 m

3.8.5 The minimum clear gap between rows, measured as the clear horizontal distance between the back of the row ahead and the nearest projection of the row behind shall be 300 mm. For chairs having automatic or self-rising seats, the measurement shall be made with the seats in the raised position, for non-automatic seats the measurement shall be taken with the seats in the down position.

3.8.6 For rows of seating served by an aisle or doorway at only one end of the row, the path of travel shall not exceed 10 m from any seat to the aisle or doorway. The minimum clear gap between rows shall be increased beyond 300 mm specified in Sec 3.8.6 by 15 mm for each seat in excess of 7, but the clear gap need not exceed 550 mm.

3.8.7 In any assembly occupancy spectators are allowed to wait in the lobby or similar space within the building until seats are available. Exits shall be provided for the waiting spaces on the basis of 0.28 m^2 areas per person waiting space and one wheel chair space for every 100 occupant. Such waiting occupant load shall be added with main assembly load for calculating exit size for the assembly as per provisions of this Code.

3.9 Doorways

One surface of a door leaf which is exposed to a fire incident is the terminal point of exit access and other surface of that said door which is unexposed to that fire incident is the starting point of an exit. A door or an opening protective assembly is an obstruction for occupants to pass through from exit accesses to exits until and unless it is installed as per provisions of this Code.

3.9.1 Each occupant of a room or space shall have access to at least one exit door or exit access assembly. The occupant load per exit door and the travel distance up to that door shall not exceed the values specified in Table 4.3.4.

3.9.2 Where either the occupant load or the travel distance exceeds the values specified in Table 4.3.4 shall have multiple exit doors to comply the both.

3.9.3 The width of a door shall not be less than 1 m and the height shall be not less than 2 m. Exit doors shall be side swing or pivoted of side hinge type.

3.9.4 No sliding or hanging door shall be used as a means of exit. In pressurized areas and when occupant load is less than 10, restriction of Sec 3.9.3 may be exempted.

3.9.5 All exit access doors shall be of a side-swinging type. When the occupant load exceeds 50 or in a hazardous occupancy, the doors shall swing outward from the room or towards the direction of travel. Swinging of the door shall not constrict the width of the corridor narrower than 0.9 m measured at the most critical position.

3.9.6 Exit doorways shall not open directly on a flight of stairway. A clear area which more than the width of the door leaf as specified in the above Sec 3.9.5 shall be maintained immediately outside the doorway. The floor levels shall be same in the direction of travel as per provisions of this Code.

Table 4.3.4: Maximum Occupant Load and Travel Distance for Spaces with One Exit Door

Occupancy		Maximum Design Occupant Load	Maximum Travel Distance (m)
A C D	Residential Institutional Health Care	12	23
B I E F G	Educational Assembly Business Mercantile Industrial	50	23

H	Storage	30	30
J	Hazardous	5	8

3.9.7 Revolving doors shall not be used as a means of exit in assembly, educational or institutional buildings or in spaces with an occupant load of 200 or more. In all other cases revolving doors shall not constitute more than half of the total required exit door width and each revolving door with least diameter of 2.7 m shall be credited not more than 50 persons. Exit doors shall be installed in the same wall within proximity of 3m of Revolving doors and shall comply with the following:

3.9.7.1 Revolving doors shall be positioned with a dispersal area at a distance of 3m or more from the foot or top of stairway or escalators or moving walks or lift lobbies.

3.9.7.2 Revolving doors shall stop rotating and stand still in a book-fold position at a force not more than 800 N or when a force is applied not more than 578 N to a wing within 760 mm of outer edge or due to sudden power failure catch automatically released and ready to manual revaluation and that provide a path which shall have aggregate width minimum 910 mm.

3.9.7.3 A manual control switch shall be installed in an approved location.

3.9.7.4 Speed of revolving door shall not exceed the revolution per minute shown below:

Speed of Revolving Door		
Inside Diameter (m)	**Manual-mode Speed limit (rpm)**	**Power-mode Speed limit (rpm)**
2	12	11
2.1	11	10
2.3	11	9
2.4	10	9
2.6	9	8
2.7	9	8
2.9	8	7
3	8	7

3.9.7.5 All exit doors shall be operable without the using a detachable key from the side they serve to evacuate.

3.10 Stairways

Change in level in elevations achieved by steps combination of identical risers and treads as per provisions of this Code shall be termed as Stairway irrespective of their locations. Stairways within an envelope shall be termed as Staircase. Exception: stepped aisles with in an assembly.

Width of Stairways shall be a length perpendicular to the direction of travel, a clear distance measured between inner edges of handrails or a clear distance between inner edges of a handrail of exposed side to its opposite and parallel surface measured at a height of inner edge of cross section of that handrail. In case of variation in width measurement the smallest value shall represent the width of a stairway. Required combination of dimensions for risers and treads given in Table 4.3.5.

Required guards and handrails shall continue for the full length of each flight of stairways. Inner turns of handrail of flights shall be at the landings and grasp ability of handrails shall be smooth and continuous, Handrail Brackets or balusters attached to the bottom surface of handrail shall not be considered to be obstructions to grasp ability. Gap between any surface and handrail shall be not less than 63.5 mm.

Stairways serving more than three storey building having capacity more than 10 occupants shall have visual enclosures to avoid any impediments to stair use by persons having fear of height, any arrangement intended to meet this requirement shall be at least 1070 mm in height.

3.10.1 The required width of exit stairways shall be computed in accordance with the provisions of Sec 3.6, but it shall not be less than the minimum widths specified in Tables 4.3.6 and 4.3.7

3.10.2 The least dimension of landings or platforms in exit stairways shall not be less than the required width of stairway and shall be leveled, except that the landing between two stair flights in a straight run shall not be required to be wider than 1.2 m in the direction of travel.

When two stair flights are not straight or nonparallel to each other, a turning in the path of travel direction occurred which is other than U turn. Landing width shall be the required width of stairway and length of the common landing between such flights shall be one tread depth more lengthen when measured from both edges of stairway from both the flights.

Table 4.3.5: Combination of Risers and Treads

<table>
<tr><th colspan="3">Gradients</th><th colspan="2">Step Dimensions</th><th rowspan="2">Available Headroom Clearance of Flight</th><th rowspan="2">Handrail or Guard Height</th><th rowspan="2">Maximum Number of Flights</th></tr>
<tr><th>Grade</th><th colspan="2">Angle of Flight</th><th>Tread Depths</th><th>Risers</th></tr>
<tr><th>(%)</th><th>Deg</th><th>(mins)</th><th>(mm)</th><th>(mm)</th><th>(mm)</th><th>(mm)</th><th></th></tr>
<tr><td>31.25</td><td>17</td><td>21</td><td>406</td><td>127</td><td>2159</td><td rowspan="4">851</td><td rowspan="4">6</td></tr>
<tr><td>33.87</td><td>18</td><td>43</td><td>394</td><td>133</td><td rowspan="3">2184</td></tr>
<tr><td>37.28</td><td>20</td><td>27</td><td>375</td><td>140</td></tr>
<tr><td>41.07</td><td>22</td><td>20</td><td>356</td><td>146</td></tr>
<tr><td>44.44</td><td>23</td><td>58</td><td>343</td><td>152</td><td rowspan="2">2210</td><td rowspan="7">838</td><td rowspan="11">Unlimited</td></tr>
<tr><td>48.07</td><td>25</td><td>40</td><td>330</td><td>159</td></tr>
<tr><td>53.06</td><td>27</td><td>57</td><td>311</td><td>165</td><td rowspan="2">2235</td></tr>
<tr><td>57.44</td><td>29</td><td>52</td><td>324</td><td>171</td></tr>
<tr><td>63.63</td><td>32</td><td>28</td><td>279</td><td>178</td><td>2261</td></tr>
<tr><td>69.04</td><td>34</td><td>37</td><td>267</td><td>184</td><td>2286</td></tr>
<tr><td>75</td><td>36</td><td>52</td><td>254</td><td>190</td><td>2311</td></tr>
<tr><td>81.57</td><td>39</td><td>12</td><td>241</td><td>197</td><td>2362</td><td rowspan="4">851</td></tr>
<tr><td>88.88</td><td>41</td><td>38</td><td>229</td><td>203</td><td>2388</td></tr>
<tr><td>97.05</td><td>44</td><td>9</td><td>216</td><td>210</td><td>2438</td></tr>
<tr><td>103.02</td><td>45</td><td>51</td><td>210</td><td>216</td><td>2464</td></tr>
<tr><td>107.07</td><td>46</td><td>57</td><td>206</td><td>222</td><td>2489</td><td rowspan="2">864</td><td rowspan="2">10</td></tr>
<tr><td>112.5</td><td>48</td><td>22</td><td>203</td><td>229</td><td>2515</td></tr>
</table>

Note: Allowable length of nosing at the outer edge of tread shall not be included in the tread depth measurement. The maximum rise of a single flight between landings shall not be exceeded 3658 mm and in case of large assembly maximum rise of a single flight between landings shall not be exceeded 2438 mm.

Table 4.3.6: Minimum Width of Stairways in Egress System

Occupancy		Minimum Width of Each Stairway (mm)
A	Residential: A1, A2	As per Table 4.3.6
	A3, A4, A5	1120
B	Educational	
	Occupant load up to 130	1120
	Occupant load more than 130 but not more 250	2235
D	Hospital	
	Patient area	2235
	Staff area	1120
I	Assembly: I1, I2, I3, I4, I5	As per provisions of this Code.
All others		As per provisions of this Code.

Note: The required number of stairways shall be determined by dividing the calculated total widths of stairways as per sections 3.5, 3.6 and Table 4.3.2 of this Chapter by applicable minimum stair width as specified in this table and any fractions thereof shall be rounded up with the next higher integer. Unit width of stair and multiple even numbers shall be maintained as per provisions this Code.

Table 4.3.7: Fire Escape Stairs

Element	Serving More than 10 Occupants	Serving 10 or Fewer Occupants
Clear widths	560 mm between handrails	
Minimum horizontal dimension of any landing or platform	560 mm	
Maximum riser height	230 mm	

Element	Serving More than 10 Occupants	Serving 10 or Fewer Occupants
Minimum tread, exclusive of nosing	250 mm	
Tread construction	Solid, 13 mm diameter perforation permitted	
Winders	Not permitted	permitted subject to Sec 3.10.7
Spiral	Not permitted	permitted subject to Sec 3.10.7
Maximum height between landings	3.70 m	
Headroom, minimum	2.00 m	
Access to protected openings	Door or casement windows, 600 mm x 2000 mm or double-hung windows 70 mm x 900 mm clear opening	Window providing a clear opening of at least 500 mm in width, 600 mm in height, and 0.53 m^2 in area
Level of access openings	Not over 300 mm above floor; steps if higher	
Discharge to ground	Swinging stair section permitted if approved by authority having jurisdiction	
Capacity, number of person	13 mm per person if access by door; 25 mm per person if access by climbing over window sill	10
Note: The maximum design occupant load for a Fire escape stair shall not be exceeded 50 occupants from any floor level.		

3.10.3 The rise and tread dimensions in a stairway shall be identical and the headroom requirements shall conform to the provisions of this Code.

3.10.4 Handrails height on stair shall be not less than 860 mm and not more than 960 mm above the surface of the tread, measured vertically from the top of the rail to the outer edge of the tread. Peripheral diameter of circular cross section of a handrail shall not be less than 32 mm and not more than 50 mm. Any other shape with perimeter dimension of not less than 100 mm, but not more than 160 mm and with the largest cross-sectional dimension not more than 55 mm shall be permitted provided that all edges are rounded to provide a radius of not less than 3 mm. Handrails shall be graspable along their entire length. Additional handrails that are lower or higher than main shall be permitted.

3.10.5 The height of guards shall not be less than 105 mm measured vertically from the top of the guards from the surface of adjacent area to be served by them. When blasters are used in the guards rail shall be used to create a pattern as such size that a sphere 100 mm in diameter shall not pass through any opening up to a height of 860 mm. Riser, tread and the bottom rail of guards formed a triangular opening shall not be of such size that a sphere 150 mm in diameter shall not pass through.

3.10.6 There shall be no variation in excess of 5 mm in depth of adjacent treads or in the height of adjacent risers, and the tolerance between the largest and smallest tread or between the largest and smallest riser is 10 mm in any flight.

3.10.7 Monumental stairs, Circular stairs, Curved stairs, Spirals and winders, stepped and rung ladders, alternate tread devices shall be permitted as per provisions of this Code.

3.10.7.1 When the width of stairways exceeded 4475 mm termed as Monumental or Grand Stairway shall be permitted as per provisions of this Code.

3.10.7.2 Curved stairs or circular stairs shall be permitted as a component of means of egress as per provisions of this Code provided that the depth of tread is not less than 280 mm at a point 300 mm from the narrower end of the tread and the smallest radius is not less than twice of stair widths and shall comply with the provisions of this Code.

3.10.7.3 Spiral stairways shall be permitted where occupant load shall not more than five. For spiral stairways the following conditions shall be applicable:

(a) The clear width of the stairs shall not be less than 660 mm.

(b) The height of risers shall not exceed 240 mm.

(c) Headroom shall be not less than 1980 mm.

(d) Treads shall have a depth not less than 190 mm at a point 300 mm from the narrower edge.

(e) All treads shall be identical.

3.10.7.4 Winders shall be permitted in stairs where occupant load shall not be more than three.

(a) Winders shall have a tread depth not less than 150 mm and a tread depth not less than 280 mm at a point 300 mm from the narrowest edge.

(b) The clear width of the stairs shall not be less than 660 mm.

3.10.7.5 Stepped ladders and Rung ladders shall be installed with pitch that exceeds 75 degrees as per standards of ANSI A14.3. The lowest rung of any ladder shall not be more than 300 mm above the level of the surface beneath it.

(a) From towers and elevated platforms around machinery or similar spaces subject to occupancy load not to exceed three persons.

(b) Open structure, observation towers or railroad signals that are designed for occupancy not more than three persons.

3.10.7.6 Alternate tread device

The occupant load shall not exceed three and shall comply with the followings:

(a) Handrail shall be provided on both sides of alternate tread device having clear width not less than 430 mm and not more than 610 mm

(c) Headroom shall not less than 2000 mm and angle of the device shall be between 50 degrees and 68 degrees to horizontal.

(c) The initial tread of the device shall begin at the same elevation as the platform, landing, or floor surfaces and the alternating treads shall not be laterally separated by a distance more than 50 mm.

(d) Treads shall have projected depth not less than 150 mm and each tread providing 240 mm of depth, including overlapping of treads.

(e) The height of the risers shall not exceed 240 mm.

3.10.8 Stairways shall have continuous guards on both side along the direction of travel and a continuous handrail shall be provided with inner edge guard. A stair of width more than 1120 mm but not more than 2235 mm shall have guards and handrails on both of the edges. Inner edge handrails shall be continuous and outer edge handrails shall be along the flights extended up to one tread depth on both the landings. A stair the width exceeds 2235 mm; intermediate handrails shall be installed with similar length of outer edge handrail. Single traffic lane shall be calculated 560 mm in the stairway and two traffic lanes shall be 1120 mm. Widths of stairs shall be multiple of two traffic lane other than width specified in the Table 4.3.6.

3.10.9 All exit stairways shall be constructed by materials that conform to the fire resistance requirements of the type of construction of the building, except that solid wooden handrails shall be permitted for all types of construction.

3.10.10 An exit stairway shall not be built around a lift shaft unless both of them are located in a smoke proof enclosure and made of a material with fire resistance rating required for the type of construction of smoke proof enclosure.

3.10.11 Exterior stairways used as fire stair shall not be considered as a component of means of egress, unless they lead directly to the ground or a refuge area, are separated from the building interior by fire resistive assemblies or walls and are constructed by noncombustible materials and free from smoke accumulation.

3.11 Ramps

3.11.1 Ramp is a sloping surface steeper than 1 in 20 but not steeper than 1 in 8 used by walkers only. Slope of ramps to comply with accessibility requirement shall not be steeper than 1 in 12.

3.11.2 The minimum width of exit ramps shall not be less than that width required for corridors or passages.

3.11.3 The slope of an exit ramp shall not exceed 1 in 8, but for slopes steeper than 1 in 10 the ramp shall be surfaced with approved non-slip material or finished such as to effectively prevent slipping.

3.11.4 Guards and handrails shall be provided on both sides of ramps having slope steeper than 1 in 15.

3.11.5 Ramps shall be straight, in case of changes in the travel direction that shall be made at the level platforms or at the landings except that ramps having a slope steeper than 1 in 12 may be curved at any place.

3.11.6 Length of the sloping portion of ramps shall be at least 915 mm but not more than 9150 mm long between level platforms or landings.

3.11.7 Level platforms or landings shall be at least as wide as the ramps and shall be placed at the bottom, at intermediate levels where required, and at the top of all ramps. Level platform shall be provided on each side of openings into or from ramps having minimum length of 915 mm in the direction of travel and when a door swings on the minimum length of platform or landing shall be 1525 mm.

3.11.8 Doors on ramps shall not be opened on sloping surface shall be complied with the requirements of 3.9 of this Chapter.

3.11.9 Sloping or ramp driveway approaching basements or any parking structures shall not be credited as an exit ramp when slope is steeper than 1 in 8 and not complied with Sec 3.11 of this Chapter. Exits requirement of such basements shall be achieved by stairways or fire lifts within smoke proof enclosure approached by a two doors smoke lock vestibule.

3.12 Horizontal Exits

3.12.1 The connection between two separated areas of a building or connection between buildings at same level which the horizontal exit serves shall be provided with at least 2 hour fire resistance rated walls, or by an open air balcony or a bridge having protected openings.

3.12.2 The horizontal exits shall be protected from the area of incidence by self-closing fire door.

3.12.3 The width of a horizontal exit access door shall not be less than 1 m.

3.12.4 Changes in level in the elevation along the direction of the horizontal exit shall not be achieved by single step but by ramps which is not stepper than 1 in 12.

3.12.5 Where the horizontal exit serves for only one side, fire door shall swing in along the direction of travel. When horizontal exit serves both the side of separated area, the doors shall have two leaves and each leave dedicated to satisfy direction of travel from assigned area, or there shall be two independent doors assigned for two areas each of them serves only one area. When the building is occupied the doors installed in horizontal exit shall be operable at all times without the use of a detachable key.

3.12.6 Horizontal exit relocates occupants to an area which is either a public space or a space used by other occupants and shall be termed as a refuge area. The capacity of the refuge area shall be computed on the basis of net floor area excluding stairways, shafts and spaces allotted to occupants of the receiving end. The required capacity of a refuge area shall be 0.28 m^2 per healthy occupant and 0.3 m^2 per wheelchair or 2.8 m^2 per patients retained in bed for delayed egress or an area equivalent to a passage or a corridor having width to comply the capacity of evacuees and connected with the components of exits up to exit discharge.

3.13 Smoke Proof Enclosure

Any compartment or a room or a control area surrounded by barrier walls within a building structure shall be protected from smoke penetration during a fire incident occurred elsewhere in the building shall be termed as smoke proof enclosure.

A stairway with in an envelope shall be termed as Interior stairway or staircase. Any exterior side having opening of 50 percent or more in such a way that there shall be no smoke accumulation shall be termed as open stair.

3.13.1 An interior stairway conforming to Sec 3.10 and having entry from an exterior balcony or through a ventilated vestibule conform a smoke proof enclosure provided no direct opening or any aperture allowed on the walls of the stair from the building side.

3.13.2 All exit stairways mentioned above shall be protected by a smoke proof enclosure when serving occupants are located in a high rise building.

3.13.3 There shall be provision to access enclosed stairways through vestibule or an open balcony. The minimum width of a vestibule shall be equal to width of connected passages or corridors specified in section 3.7 in this Chapter and the minimum length of a vestibule in the direction of travel shall be 1.8 m.

3.13.4 The minimum fire resistance rating of the walls forming a smoke proof enclosure around stairway including the vestibule thereof shall be 4 hours and separated from the area of incidence having no openings other than a fire door for the entry to the vestibule. For fire rating of the door see Chapter 1 Part 3.

3.13.5 All doors in smoke proof enclosure and the vestibule shall be self-closing type or they shall be fitted with automatic closing devices actuated by the fire detection system.

3.13.6 The vestibule shall have adequate natural ventilation. Each vestibule shall have a minimum area of openings of 2 m^2 divided into two in an exterior wall facing a courtyard, street or public way wider than 6 m. The location of one opening measuring 1.5 m^2 shall be as high as possible and another shall be 0.5 m^2 as low as possible.

3.13.7 If the enclosed staircase is windowless, mechanical ventilation shall be installed. If the vestibule is windowless, mechanical ventilation shall also be installed. In addition to ventilation a positive pressure of 50 Pa shall be maintained in the vestibule. This positive pressure must be developed within 30 seconds of the incident of fire. When the staircase and the vestibule are windowless emergency illumination shall be provided.

3.14 Number of Exits

3.14.1 The number of exits shall be determined as per provisions of Sec 3.6, Tables 4.3.1, 4.3.2 and 4.3.8 of this Chapter and complying with maximum dead end passage or corridors and maximum travel distance.

3.14.2 Total required widths of exits shall be calculated as per provisions of the Tables 4.3.2 and 4.3.8 shall be divided and distributed at a distance not less than one-third of diagonal distance of space and the travel distance and the width of each exit shall comply with the provisions of this Code. The required number of exits in a space as specified below:

Occupant load less than 50	Minimum 1 exit
Occupant load 50 to 500	Minimum 2 exits
Occupant load 501 to 1000	Minimum 3 exits
Occupant load more than 1000	Minimum 4 exits

3.14.3 High rise buildings having a floor area larger than 500 m^2 on each floor used as educational, institutional, assembly, industrial, storage or a mixed occupancy involving any of these or hazardous occupancy, shall have a minimum of two staircases. These staircases shall comply with the requirements as specified in Sec 3.13 of this Chapter.

3.14.4 Where two accessible means of egress are required, the exits serving such means of egress shall be located at a distance from one another not less than one-half the length of the maximum overall diagonal dimension of the building or area to be served.

3.15 Travel Path

3.15.1 Travel path shall be measured along the center line of a natural and unobstructed path up to center of an exit access door opening. In case of a stairway exist in the travel path shall be measured along an inclined straight line through the center of outer edge of each tread of a stairway.

3.15.2 Occupant load and components of exits shall be arranged in such a manner that the travel path from any point in the area served shall not be exceeded as listed in the Table 4.3.8.

3.15.3 Unit width shall be 560 mm and fraction of unit width less than 280 mm shall not be credited. Where calculation of total required width give fractional result, next larger integral number of exit units or integral number plus one-half shall be used. Where changes in elevation exist, one-half or less unit of width shall not be permitted.

3.15.4 Capacity of exits shall be measured in unit of width of 560 mm and the number of occupants per unit width shall be determined by the occupancy group and type of exits as listed in Table 4.3.8.

3.15.5 Wherever more than one exit required in a room or in any floor they shall be placed as remote as possible from each other. As far as practicable, exits shall be arranged in such a manner to provide a refuge area or an exit discharge to the occupants irrespective of the direction of travel from any point in an area served.

3.16 Means of Exit Signs and Illumination

3.16.1 All required means of exit or exit access in buildings or areas requiring more than one exit shall be signposted. The signs shall be clearly visible at all times, where necessary supplemented by directional signs. All exit doors shall be clearly marked for easy identification.

Exceptions: Building Occupancy type A.

3.16.1.1 Location: Exit signs shall be installed at stair enclosure doors, horizontal exits and other required exits from the storey. When two or more exits are required from a room or area, exit signs shall be installed to clearly indicate the direction of egress.

Exceptions:

(i) Main exterior exit doors which obviously and clearly are identifiable as exits need not be signed when approved by the Building Official.

(ii) Exit signs are not required for buildings of occupancies A1, A2 and individual units of A3.

(iii) No sign is needed for exits from rooms or areas with an occupant load of less than 50 for Occupancy type C.

Table 4.3.8: Determination of Exit and Access Requirements.

Occupancy Group/ Classification		Maximum Travel Path (meter)		Capacity Number of Occupancy per unit width of the component				Ramp, Passage , Corridors	
				Door openings					
		Unsprinklered	Full fire resistive or sprinklered	To outdoors at Grade	All other Exit and corridor doors	Stairs, Escalators	Ramp, Corridors, Exit passageways, Horizontal exit	Minimum width (mm)	Maximum Dead End (mm)
A1, A2	Residential	N.R.	N.R.	N.R.	N.R.	N.R.	N.R.	N.R.	N.R.
A3,		45	60	50	40	30	50	36	12190
A4,A5		45	60	50	40	30	50	36	12190
B	Educational	45	60	100	80	60	100	915	12190

C1, C2	Institutional	38	53	50	40	30	50	915	12190
C3, C4, C5		38	53	30	30	15	30	2440	9150
D	Health	38	53	30	30	15	30	2440	9150
E	Business	60	90	100	80	60	100	1120	15240
F	Mercantile	45	60	100	80	60	100	915	15240
G1	Industrial	60	120	100	80	60	100	1120	15240
G2		60	120	100	80	60	100	1120	15240
H1	Storage	30	53	75	60	45	75	915	15240
H2		38	45	75	60	45	75	915	15240
I	Assembly	45	60	100	80	60	100	1675	9150
J	High Hazard	15	45	50	40	30	50	915	N.P.

Notes:

1. In Hazardous occupancy (occupancy J) Travel Path should be performance based but shall not exceed 15240 mm.
2. N.P. = Not permitted
3. N.R. = No requirement, (except as provided in Table 4.3.5b)
4. Capacity of ramp shall be reduced by twenty five percent when slope is steeper than 1 in 10.
5. Corridors serving classroom area of an educational building. Other corridors shall have a minimum width of 1120 mm.
6. Applies to corridors to patient area. Staff corridors shall have a minimum width of 1120 mm.

3.16.1.2 Graphics: The color and design of lettering, arrows and other symbols on exit signs shall be in high contrast with their background as per NFPA 170. Words on the signs shall be at least 150 mm high with a stroke of not less than 20 mm. For vernacular alphabet and numeric height shall be at least 150 mm with stroke not less than 20 mm.

3.16.1.3 Illumination: Signs shall be internally or externally illuminated by two electric lamps or shall be of self-luminous type. When the luminance on the face of an exit sign is from an external source, it shall have an intensity of not less than 53.8 lux from either lamp. Internally illuminated signs shall provide equivalent luminance.

3.16.1.4 Source of Power: Supply of power to one of the lamps for exit signs shall be provided by the premises wiring system. Power to the other lamp shall be from an on-site generator set which shall be installed in accordance with the provisions of this Code.

3.16.1.5 Floor-level Exit Signs: For floor-level exit signs additional approved low-level exit signs which are externally or internally illuminated, or self-luminous, shall be provided in all interior exit corridors serving guest rooms of hotels in Occupancy A5. The bottom of the sign shall be 150 mm to 200 mm above the floor level. For exit doors, the sign shall be on the door or adjacent to the door with the closest edge of the sign within 100 mm of the door frame.

3.16.2 Amusement Building Exit Marking: Approved exit direction marking and exit signs shall be provided. Approved low-level exit signs and directional marking shall be located not more than 200 mm above parallel the walking surface and at the exit path.

3.16.3 All exit signs shall be illuminated while in use at night, or during dark periods within the area served, in accordance with the provisions of this Code.

3.16.4 The means of exit and exit access in buildings requiring more than one exit shall be equipped with artificial lighting. The lighting facilities shall satisfy the provisions of this Code.

PART IV
Chapter 4
Equipment and In-Built Facilities Standards

4.1 Scope

The provisions of this chapter shall control standards of the design, installation and maintenance of equipment and in-built fixed, localized, portable facilities required for firefighting within a building and its premises. The regulations of this chapter shall be applicable for all buildings and the provisions stated herein shall not cover the firefighting requirements outside the building premises.

4.1.1 Extinguishing agents can be water, dry sand, ash, inert gas, dry chemical, and wet chemicals or mixed in nature of approved type. Agents will be selected as per the area have to extinguished.

4.1.2 The gaseous system shall be only used where water or foam cannot be used for fire extinguishing because of the special nature of the contents within the building or areas to be protected.

4.1.3 Fixed type fire protection system means there shall be a pipe circuit to cover full or part of a building and extinguishing agents supplied from a point. Localized fixed system means the system will cover a confined space with a self-extinguishing device fitted with a container ready to discharge automaticity. Portable type means the extinguishers can be hand carried in the site of incidents.

4.1.4 Fire Classification

Fire class A: Fire involving common combustibles such as wood, paper, plastics, clothes etc.

Fire class B: Fire involving flammable liquids and gases, such as gasoline, propane, and solvents.

Fire class C: Fire involving live electrical equipment such as computer, fax machine etc.

Fire class D: Fire involving combustible metals such as magnesium, lithium, aluminum etc.

Fire class K: Fire involving cooking media such as cooking oils and fats.

4.2 Fixed Type Fire Hydrant System

General area of application shall be Fire class A. Fixed type fire hydrant system comprises of, stand pipes and hose or reel pipes, sprinklers, drenchers or similar devices in appropriate combinations of these and capable of discharging water in an area which to be extinguish.

4.2.1 Water Quantity for Fire Protection

The required flow rate and duration of water for sprinkler or stand pipe system use within the building according to their occupancy classification shall be in accordance with Table 4.4.1 and size of pipes shall be as per provisions of this Code or on the basis of the hydraulic design of the system to maintain flow rate and duration of water discharge.

4.2.2 Water Sources for Fire Protection

Flow rate and duration of discharging water required for interior fire extinguishment of a building shall be supplied from one or any combination of the following sources.

Table 4.4.1: Fire Protection Flow Requirements*

Building Type**	Sprinkler System (litre/min.)	Standpipe and Hose System (litre/min.)	Duration in Minutes for Building Heights		
			Up to 51 m	51 m to 102 m	Above 102 m
Light hazard- I	1000	1000	30	38	45
Light hazard- II	1900	1900	50	62	75
Ordinary hazard- I	2650	1900	75	95	112
Ordinary hazard-II	3200	1900	75	95	112
Ordinary hazard-III	4800	1900	75	95	112

Notes:

* See also Sec 4.2.2.3.

** Values will be for one riser serving floor area of 1000 m^2.

Light hazard-I	:	Occupancy groups, A1, A2, A3, E1
Light hazard-II	:	Occupancy groups, A4, A5, B, C, D,E2, E3, I2, I4, F1
Ordinary hazard-I	:	Occupancy groups, I1, I3, I5, F2, F3, G1
Ordinary hazard- II	:	Occupancy groups, G2 , H1
Ordinary hazard- III	:	Occupancy groups, H2
Extra hazard	:	Occupancy group J-pressure and flow requirement for this group shall be determined by Fire Department but shall not be less than required value for Ordinary hazard- III

4.2.2.1 Direct connection to water main

For continuous water supply (public water supply system or private system) with sufficient quantity and pressure to feed and discharge firefighting equipment during peak demand period, direct connection of firefighting system to the water main may be adopted, Figure 4.4.1. In this case guidelines specified in NFPA 22 are to be followed.

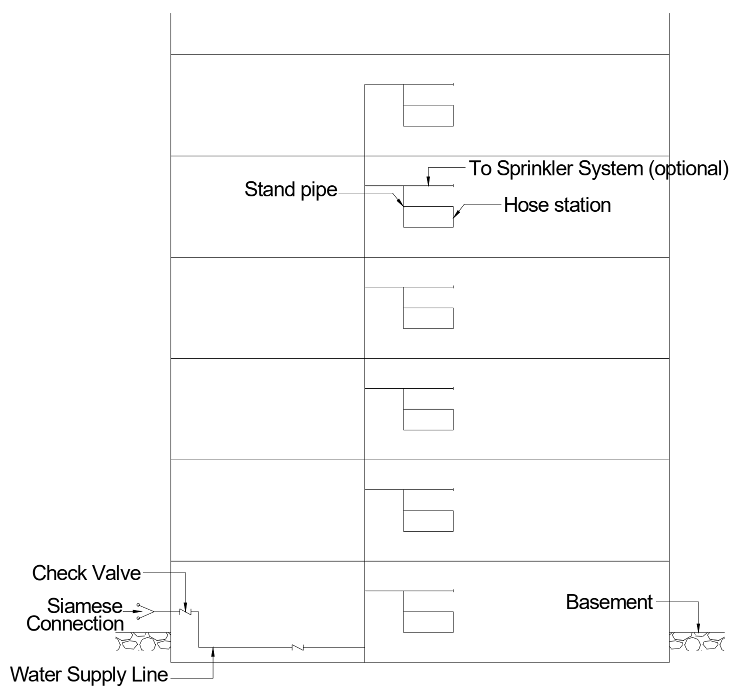


Figure 4.4.1 Typical diagram for standpipe and hose system connected directly to the water main

4.2.2.2 Roof gravity tanks

Any elevated structure holding a water reservoir or water tank or in any level within a building and having downward supply pipelines shall be termed as gravity tank only when a water reservoir located on a roof of a building shall be termed as roof gravity tank.

For water supply system with inadequate quantity or pressure during peak demand period but with sufficient pressure to feed roof tank, a roof gravity tank may be provided. In that case any one of the following steps shall be followed.

(a) If only the static height of the roof gravity tank is used to feed and discharge the firefighting equipment, the height of the roof gravity tank from the top floor must be sufficient to create minimum required pressure at the top floor hydrant point. The minimum pressure at hose outlet for standpipes supplying a 50 mm or larger hose shall be at least 300 kPa. This minimum pressure for standpipe system supplying first aid hose (38 mm nominal) shall be at least 200 kPa. This minimum pressure for combination of sprinkler and hose pipe system shall be 600 kPa. To maintain the above required pressure the vertical distance of the roof gravity tank from the top floor hydrant point shall be at 31 m, 20.5 m and 62 m respectively, Figure 4.4.2.

(b) If the vertical distance between the roof gravity tank and the top floor hydrant point cannot be maintained for gaining required pressure and discharge, fire pump of required size and number shall be installed with standard manufacturer recommended suction and delivery connections, Figures 4.4.3 and 4.4.4.

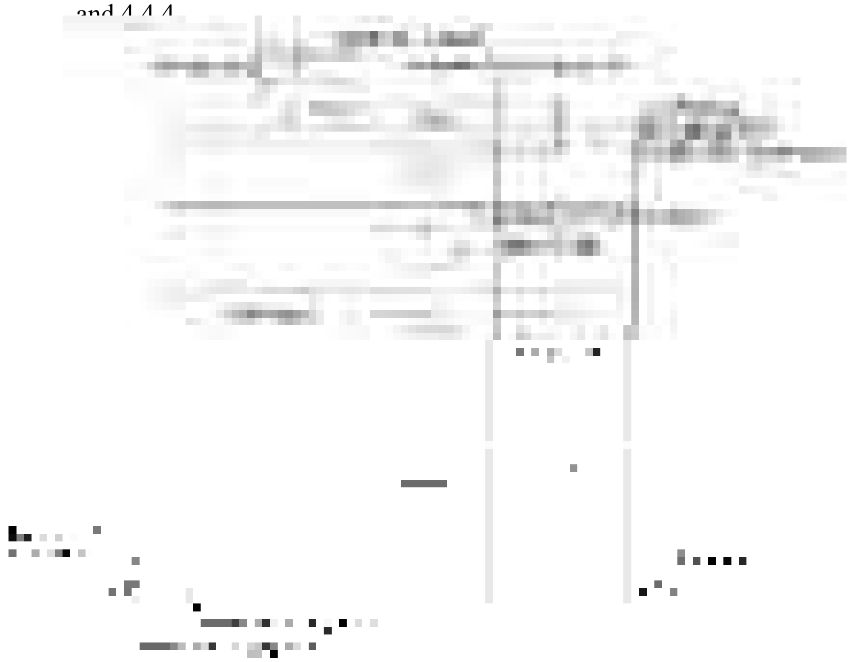


Figure 4.4.2 Typical diagram showing required static head of gravity roof tank with adequate domestic and fire reserve

4.2.2.3 Storage tank

In absence of public water supply system, the building premises shall have individual water sources specified in Part 8. For water supply system, to feed and discharge by firefighting equipment, the building premises may have deep tube well with required flow, water wells, natural water sources or a ground (or underground) tank, roof top tank, swimming pools etc. The capacity of these facilities shall be sufficient to satisfy the flow requirement as specify in Table 4.4.1.

4.2.2.4 Water supply test

After installation of the hydrant system, a flow test shall be conducted to verify the capacity of the discharge system such that the installation can fulfill the minimum capacity (flow and time) as specified in Table 4.4.1. This system shall be periodically inspected, maintained and tested in accordance with NFPA 25.

4.2.2.5 Fire pump

The firefighting equipment shall be directly feed by automatic main fire pump. Centrifugal pump, turbine-type pump (submerged or with vertical shaft) or positive displacement pumps with adequate supply pressure and flow capacity shall be used for water supply during demand. Centrifugal pumps shall not be used where a static suction lift is required.

Once the pump starts, it shall run continuously until stopped manually. The pump shall be fully operational within 30 seconds after starting. There shall be provision for manual starting where priming is necessary. Automatic priming equipment is necessary to ensure priming at all times. The fire pump shall not be used for other purpose.

Fire pumps shall have the rated capacities as shown in Table 4.4.2. The pump shall be rated at net pressure of 272 kPa or more as per requirement of the firefighting system demand. For pump installation procedure and fittings NFPA 20 shall be followed.

The pump shall be housed in a readily accessible position in a building of non-combustible construction. The pump shall be adequately protected against mechanical damage.

There shall be a provision for secondary fire pump which can be operated by a dedicated diesel engine or by an alternate power supply source with adequate control system and incompliance with safety operation during fire. Quality of the pump assembly shall comply with the specification of International Association of Fire.

Figure 4.4.3 Typical diagram for gravity roof tank with adequate domestic and fire reserve.

4.2.3 Design Considerations for Standpipe and Hose System

4.2.3.1 The fire protection system shall be designed for their effective use either by amateur or trained firefighting personnel or both.

4.2.3.2 All standpipes in standpipe system shall be sized so that they will provide a minimum flow specified in Table 4.4.1. In standpipe system with more than one standpipe, the supply piping shall be sized for the minimum flow specified in Table 4.4.1 for the first standpipe plus 1000 litre per minute for each additional standpipe. The total number of such additional standpipes shall not be more than 8. All standpipe risers shall be interconnected through check valves of equivalent size to prevent recirculation.

Figure 4.4.4 Typical diagram for gravity roof tank with adequate domestic and fire reserve.

4.2.3.3 The minimum pressure for standpipes supplying a 50 mm or larger diameter hose shall be at least 300 kPa. For standpipe supplying first aid hose (38 mm nominal diameter) may have a minimum pressure of 200 kPa. The maximum pressure at any point of the system shall not exceed 2434 kPa, if the hose connection at 40 mm diameter outlet exceeds 700 kPa approved pressure regulating device shall be installed to maintain the above maximum limits.

4.2.3.4 Diameter of the standpipe termed as size shall comply with flow and capacity requirement of the pump shown in Table 4.4.2 or hydraulically design to provide required flow and pressure at the topmost hydrant point.

Table 4.4.2: Fire Pump Data

Pump Rating litre/min (gpm) Discharge,	**Minimum Pipe Sizes (Nominal) mm (inch)**
946 (250)	75 (3)
1136 (300)	100 (4)
1514 (400)	100 (4)
1703 (450)	125 (5)
1892 (500)	125 (5)
2839 (750)	150 (6)
3785 (1000)	150 (6)
4731 (1250)	200 (8)
5677 (1500)	200 (8)

4.2.3.5 The water supply required for combined system (for partial automatic sprinkler and Fire Department hose) shall be calculated in accordance with Table 4.4.1 plus an amount equal to the hydraulically calculated sprinkler demand.

4.2.3.6 The system for firefighting purpose may be designed with automatic fire pump with water tank at the ground as shown in Figure 4.4.5.

4.2.3.7 The water stored in storage tank for firefighting operation shall not be used for other purposes. Accordingly, separate water connections should be provided as shown in Figure 4.4.6.

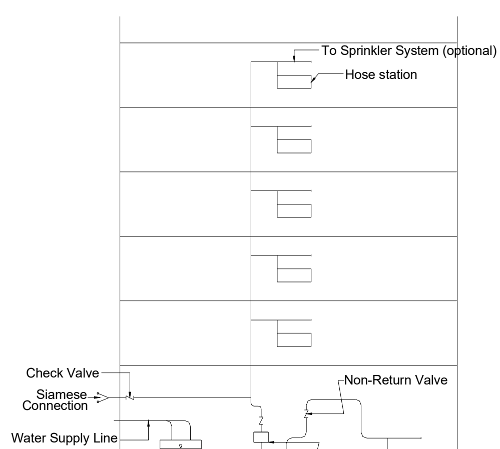


Figure 4.4.5 Typical diagram for fire protection with ground tank and automatic fire pump

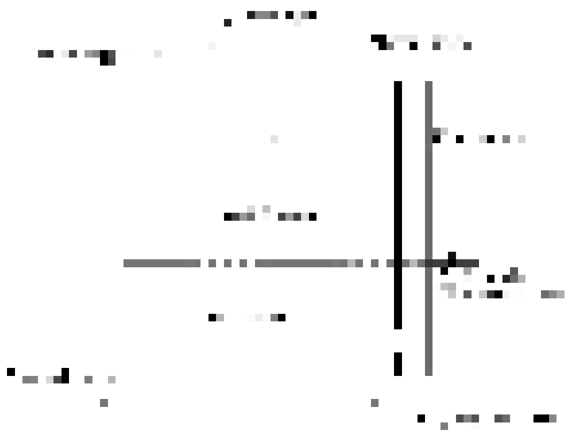


Figure 4.4.6 Typical diagram for storage tank (ground or overhead) with domestic and fire reserve.

4.2.3.8 The ground storage tank shall be easily accessible to fire engine of Fire Department. In absence of space available for fire engine, the cover slab of ground storage tank shall be designed to withstand a vehicular load of local fire engine.

4.2.3.9 The standpipe shall be located such as intermediate stair landing, vestibules or nearby in noncombustible enclosure such that it will be able to provide hose stream to the most remote area of the floor served.

4.2.3.10 The hose shall be connected to the standpipe within a height not more than 1.5 m from the finished floor level. The hose stations shall be easily accessible for inspection and testing.

4.2.3.11 The hose connection to a standpipe for large stream shall be at least 100 mm nominal and that of small stream may be 63 mm or 50 mm on each point. The size of first aid hose shall be 38 mm nominal. The hose length shall not be more than 30 m.

4.2.3.12 Different piping materials and fittings for standpipe system presented in Tables 4.4.3 and 4.4.4 shall conform to the standard or one of the standards cited against them. The standard requirements for other materials not provided in these tables shall be subject to the approval of the Authority.

Table 4.4.3: Piping for Standpipe System

Material	Standard
Copper Tube	ASTM B75, ASTM B88
Copper and Copper-Alloy Tube	ASTM B251
Steel Pipe	ASTM A55, ASTM A120, ASTM A135
Wrought Steel or Iron	ANSI B36.10

Table 4.4.4: Standpipe Fittings

Material	Standard
Cast Iron	ANSI 616.1, ANSI B16.4
Copper	ANSI B16.18, ANSI B16.22
Malleable Iron	ANSI B16.3
Steel	ANSI B16.5, ANSI B16.9, ANSI B16.11, ANSI B16.25, ASTM A234

4.2.3.13 The standpipe riser shall be supported at the top and at the lowest level. The riser shall also be provided with support at the alternate level in between top and bottom level of the standpipe riser. The support shall be of adequate strength to support the water-filled pipe load and an additional load of 110 kg.

4.2.3.14 The horizontal standpipe shall have hangers with a spacing not more than 5 m. The hanger shall be able to carry a load of five times the weight of the water-filled pipe and an additional load of 110 kg.

4.2.3.15 There shall be Siamese connection also termed as firemen connection to the standpipe or to the delivery pipe of the gravity roof storage tank. The location of Siamese connection shall be easily accessible from the street or means of access.

4.2.3.16 The system shall be provided with adequate drainage piping to discharge under pressure. The drain pipe shall not discharge into sanitary sewer.

4.2.3.17 All control valves shall be designed to withstand the pressure specified in Sec 4.2.3.3

4.2.4 Wet Riser

A wet riser is a vertical pipe of not less than 100 mm internal diameter, kept permanently charged with water which is then immediately available for use on any floor in the building at which a hydrant or landing valve is provided. The riser is connected to a booster pump or town main of suitable capacity so that they are capable to supply four 13 mm jet at 2.5 bars at the highest outlets.

4.2.5 Down Comer

A similar function to that of wet riser is performed by down comer which like a wet riser is constructed of vertical piping, with outlets at different levels, but is supplied with water from a tank in the roof through terrace pump, gate valve and non-return valve. It is also fitted with inlet connections at ground level and air release valve at roof level for being capable of charged with water by pumping from fire engines.

4.2.6 High Velocity Water Spraying Projector System

This system applies water in the form of conical spray consisting of droplets of water traveling at high velocity. The three principles of extinguishments are employed, namely emulsification, cooling and dilution. While the water droplets are passing through the flame zone, some of the water is turned into steam, diluting the oxygen feeding the fire. Addition of water to the burning oil also cools it and reduces the rate of vaporization. In addition to this droplets of water traveling at high velocity bombard the surface of the oil to form an emulsion of oil and water that will not support combustion.

4.2.7 Water Mist Technology

Fine water spray suppression system can extinguish fires using water and nitrogen from air. Nozzle is used to atomize water by nitrogen or other suitable media to generate mist or fog of finely controlled water droplets. The system operates at low pressure and produces droplets in a range of 80 to 200 microns. These droplets extinguish fire rapidly and efficiently even those involving highly volatile hydrocarbons. This system is an alternative to Halon and other gaseous system in many applications.

4.2.8 Drenchers

Drenchers are used for the external protection of the building against exposure hazard, or radiant heat. Drencher heads are similar to sprinkler heads and may be sealed or unsealed. Drenchers are of three types, roof drenchers, wall drenchers, window drenchers.

4.2.9 Dry Riser System

Dry riser stand pipe system shall be an equivalent alternative of wet riser stand pipe system. The water supply for an automatic or semi-automatic standpipe system shall be designed such that the system must be capable of supply the system during peak demand hour.

4.2.10 Design Consideration of Sprinkler System

4.2.10.1 A system of water pipes fitted with sprinkler heads as per manufacturers specification may be installed actuate automatically, control and extinguish a fire by the discharge of water.

4.2.10.2 The pipe schedule sizing to supply different number of sprinklers for their different uses may be in accordance with Tables 4.4.5 and 4.4.6

4.2.10.3 Each sprinkler shall serve a maximum ceiling area specified in Table 4.4.7 for different types of building according to their uses.

4.2.10.4 Water supply piping and fittings for sprinkler system shall conform to the standard or one of the standards cited against them in accordance with Tables 4.4.4 and 4.4.8. The standard requirements for other pipe materials not provided in these tables shall be subject to the approval of the Authority.

4.2.10.5 The sprinkler system shall be provided with adequate support or made flexible to prevent pipe breakage during earthquake.

4.2.10.6 The hanger in sprinkler system shall be designed to carry a load equal to five times the weight of the water-filled pipe plus an addition load of 110 kg. The support shall be designed to support a load equal to the weight-filled pipe plus and additional load of 110 kg.

4.2.11 Connection

4.2.11.1 There shall be Siamese connection to the sprinkler system located outside the building and accessible to the fire department connection.

4.2.11.2 All risers shall be connected through a gate valve with a main of size equal to that largest riser.

4.2.11.3 The sprinkler system shall be provided with adequate drainage arrangement. The drain pipe shall not discharge into sanitary sewer.

4.2.11.4 All control valves and fittings shall be able to withstand the pressure specified in Sec 4.2.3.3.

4.2.12 Inspection, Testing and Maintenance

4.2.12.1 Inspection

All piping and equipment shall be inspected for satisfactory supports in accordance with Sec 6.15 in Part 8 of this Code and protection from damage and corrosion. All outlets shall be free from obstruction.

Table 4.4.5: Size of Water Supply Steel Pipe to Sprinklers

Pipe Size mm (inch) nominal	No. of Sprinkler for Light Hazard*	No. of Sprinkler for Ordinary Hazard *	No. of Sprinkler for Ordinary Extra Hazard *
25(1)	2	2	1
32($1\frac{1}{4}$)	3	3	2
38($1\frac{1}{2}$)	5	5	5
50(2)	10	10	8
63($1\frac{1}{2}$)	30	20	15
75(3)	60	40	27
88($3\frac{1}{2}$)	100	65	40
100(4)	NL**	100	55
125(5)	-	160	90
150(6)	-	275	150
200(8)	-	400***	225***

* Definitions of these terms are given in Table 4.4.1.

** No limit.

*** One sprinkler system riser or combined system riser shall serve the floor area not more than 4850 m^2 for light and ordinary hazardous occupancy and 2325 m^2 for extra hazardous occupancy

Table 4.4.6: Size of Water Supply Copper Pipe to Sprinklers

Pipe Size mm (inch) nominal	No. of Sprinkler Connection for Light Hazard*	No. of Sprinkler Connection Ordinary Hazard *	No. of Sprinkler Connection Ordinary Extra Hazard *
25(1)	2	2	1
32($1\frac{1}{4}$)	3	3	2
38($1\frac{1}{2}$)	5	5	5
50(2)	12	12	8
63($1\frac{1}{2}$)	40	25	20
75(3)	65	45	30
88($3\frac{1}{2}$)	115	75	45
100(4)	NL**	115	65
125(5)	-	180	100
150(6)	-	300	170
200(8)	-	***	***

* Definition of these terms is given in Table 4.4.1.
** No limit.
*** One sprinkler system riser or combined system riser shall serve the floor area not more than 4850 m^2 for light and ordinary hazard occupancy and 2325 m^2 for extra hazard occupancy

Table 4.4.7: Ceiling Area for a Sprinkler

Construction Type	Light Hazard		Ordinary Hazard		Extra Hazard	
	Protected area	Spacing (Max)	Protected area	Spacing (Max)	Protected area	Spacing (Max)
	ft^2 (m^2)	ft (m)	ft^2 (m^2)	ft (m)	ft^2 (m^2)	ft (m)
Roof or Floor on Trusses, Girders or Beam With High Piling ***	200 (18.6)	15 (4.6)	130 (12.1)	15 (4.6)	100 (9.3)	12 (3.7)
Open Wood Joists With High Piling ***	225 (20.9)	15 (4.6)	130 (12.1)	15 (4.6)	100 (9.3)	12 (3.7)
Other Type of Construction With High Piling ***	168 (15.6)	15 (4.6)	130 (12.1)	15 (4.6)	100 (9.3)	12 (3.7)

* Maximum distance in m between sprinklers and between line of piping.

* * The definitions of these terms are given in Table 4.4.1.

** * Storage facilities which permit closely piled materials over 4.5 m or materials on rack over 3.6 m.

Table 4.4.8: Piping for Sprinkler System

Material	Standard
Copper and Copper-Alloy	ASTM B32, ASTM B75, ASTM B88, ASTM B25, ANSI B36
Steel	ASTM A53, ASTM A120, ASTM A135, ASTM A795

4.2.12.2 Testing

Fire protection plumbing system or part thereof shall be tested and approved after installation by the Authority.

(a) Testing of Standpipe System: The hydrant pipes shall be hydraulically tested to a pressure 1400 kPa or 150% of working pressure whichever is the higher for 2 hours without any leakage at any points. The system shall be able to maintain above test pressures. The system shall also be tested for the required flow at the highest outlet.

(b) Testing of Sprinkler System: This system shall be tested for at least 2 hours for a pressure of 1000 kPa or at 350 kPa in excess of normal working pressure when normal working pressure will be more than 650 kPa. The system shall be able to maintain above test pressures. The system shall also be tested for the required flow at the highest outlet.

(c) Testing of Sprinkler System Pump: The pump used for sprinkler system firefighting purpose shall be tested by approved authority for their performance characteristics and this test report must be submitted at the time of supply of pump. The pump shall be retested or repaired to its original condition if their performance characteristics fall below more than 10 percent of the supplier's test characteristic curve or as specified for the fire protection water supply system.

4.2.12.3 Maintenance

The system shall be maintained for safe operating conditions and tested at least once a year.

4.3 Fixed Installation Other Than Water

Other than water there are different types of fixed installation. These are of mainly two types. (a) Centrally fixed, (b) locally fixed.

4.3.1 Centrally Fixed Installation Discharging Extinguishing Agent other than Water

4.3.1.1 General

This installation can be of two types, one for zone coverage and the other for total coverage. For these system pipe circuits and exhaust manifold are required and shall have special discharging Alarm distinctly different than fire alarm. These fixed installations can be of different types, such as (a) Foam installation, (b) Vaporizing liquid installation, (c) Dry powder installation. (d) Gaseous installation (e) Dry chemical installation (f) Wet chemical installation.

4.3.1.2 Foam installation

Foam extinguishing system shall be of an approved type and shall be installed in accordance with the specification of the manufacturer. The foam extinguishing system is designed to discharge fire suppressive foam concentrates over the area to be protected.

(a) There are different types of foam installation, such as (i) Pump operated mechanical foam installation, (ii) Self-contained pressurized installation, (iii) Pre-Mixed Foam installation, (iv) High Expansion Foam installation.

(b) A foam extinguishing system shall be automatically actuated during a fire with provision of manual actuation.

(c) Warning sign and discharge alarm system shall be provided with the foam extinguishing system, which shall be actuated during the use of the system.

(d) The system provides protection of boiler rooms with its ancillary storage of furnace oils in basement and other areas where hazardous liquids are stored.

4.3.1.3 Vaporizing liquid installation

Liquefied compressed Halogenated hydrocarbon is fed through distribution pipe works and specially designed discharged nozzles to the area need to be extinguished. Upon discharge the liquid immediately vaporized to form a heavy vapour which achieves very rapid extinction.

There are two types of Vaporizing liquid installation, such as total flooding system and Local application system. This system shall be installed in accordance with the specification of the manufacturer. Safe guards are necessary to prevent injury or death of personnel in area where the atmosphere may be made hazardous by the discharge.

4.3.1.4 Dry powder installation

Dry powder of certain chemicals installation consist of pipe work and discharge nozzle and pressuring media. This installation can be operated automatically or manually. This can be designed for total coverage and for zone coverage.

Dry powder is a range of chemical agents available as extinguishing media. They are used on various flammable liquids where they are confined. This system shall be installed in accordance with the specification of the manufacturer.

4.3.1.5 Gaseous installation

(a) General: Gaseous extinguishing system shall be of an approved type and shall be installed as per provisions of this Code. The system supplies gas from a pressurized vessel through fixed pipes and nozzles.

(b) The system is used where water or foam cannot be used for fire extinguishing because of the special nature of the contents within the building or areas to be protected.

(c) The system shall be automatically actuated and shall be equipped with manual actuation devices as well.

(d) Warning signs and discharge alarm shall be provided where persons are likely to be trapped in an area made hazardous due to discharge of extinguishing gases.

(e) Halocarbon agents and inert gas system: Any approved Type of Halocarbon agents are chemicals in the liquid form at high pressure and vaporize readily leaving no residue. These are primarily to protect hazardous fire in enclosed room, vaults, machines, containers, storage tanks, engines, unattended computer server rooms, electrical appliances, liquid gas storage etc. Some example of these chemical is dichlorodifluoro ethane, chlorodifluoro methane. Inert gas system is also an alternative of Halocarbon agents. These are nitrogen and argon in pure form or in mixer at different proportion. These gases are identified as clean total folding fire suppression agents. They are stored in high pressure gas cylinders.

4.3.1.6 Dry chemical extinguishing system

(a) General: Dry chemical extinguishing system shall be of an approved type and shall be installed in accordance with the provisions of this Code and manufacture's instruction.

(b) The system shall be automatically actuated during a fire and shall be equipped with manual actuation device as well.

(c) Warning signs and discharge alarm shall be provided where persons are likely to be exposed to chemical discharge. Chemical agents of the system shall be nontoxic.

4.3.1.7 Wet chemical extinguishing system

(a) A wet chemical system is a solution of water and potassium carbonate or acetate based chemical which forms the extinguishing agent. The system shall be installed in accordance with the provisions of this Code and manufacturer's installation instruction.

(b) The system shall be automatically actuated during a fire and shall be equipped with manual actuation device as well.

(c) In case of wet chemical extinguishing system, label of the approved agent shall be affixed.

(d) Warning signs and discharge alarm shall be provided where persons are likely to be exposed to wet chemical discharge.

4.3.2 Localized Fixed

Containerized extinguishing agent are available in different shapes and size to be placed in different locations those are prone to fire hazard as for example at the top of cookers in the kitchen, electric connection box etc. Use of these containers shall be approved type and installation shall be as per specification of the manufacturer.

4.4 Portable Fire Extinguisher

4.4.1 Portable fire extinguishers shall readily available in different type. These are portable fire extinguisher are of carbon dioxide types, dry chemical types, water types, and Halon types, film-forming type, foam types and Halon carbon type. For proper operation persons with adequate knowledge and familiar with their operation must be available.

4.4.2 In accordance with the occupancy hazard, specification of the manufacturer and guide line set by NFPA 10, the minimum number of portable fire extinguishers for different class of fire shall be ascertained. As for example where the floor area of a building is less than 279 m^2 at least one fire extinguisher of the minimum size is recommended for Fire Class A.

4.4.3 Portable fire extinguishers shall be fully charged, operable at any time and conspicuously located where they will be readily accessible. Portable fire extinguishers shall not be obstructed or obscured from view. In large rooms, means shall be provided to indicate the extinguisher location.

4.4.4 Portable fire extinguishers shall be adequately protected from impact, vibration, and adverse environment and shall not be exposed to temperatures outside the listed temperature range shown on the fire extinguisher label.

4.4.5 Portable fire extinguishers mounted in cabinets or wall recesses shall be placed so that the fire extinguisher operating instructions face outward. The location of such fire extinguishers shall be marked conspicuously.

4.4.6 The owner or designated agent or occupant of a property in which fire portable extinguishers are located shall be responsible for inspection, maintenance, and recharging. The procedure for inspection and maintenance of fire extinguishers varies considerably. Monthly "quick check" or inspection in order to follow the inspection procedure as outlined in NFPA 10 shall be done.

4.4.7 Maintenance, servicing and recharging shall be performed by trained persons having available the appropriate servicing manual(s), the proper types of tools, recharge materials, lubricants, and manufacturer's recommended replacement parts or parts specifically listed for use in the fire extinguisher. These extinguishers shall be maintained as per NFPA 10, at intervals of not more than one (1) year.

4.4.8 All rechargeable-type fire extinguishers shall be recharged after any use or as indicated by an inspection or when performing maintenance or as per the recommendations of the manufacturer.

4.4.9 For personal safety during approach with extinguishing equipment it shall be remembered that most fires produce toxic decomposition products of combustion and some materials can produce highly toxic gases. Fires can also consume available oxygen or produce dangerously high heat. All of these can affect the degree to which a fire can be safely extinguished.

4.4.10 All extinguishing agents other than clean agents shall be approved by the authorities having jurisdiction.

4.5 Rate of Water Flow For Fire Protection In Tall Building

High rise building exceeding 80 meter height shall be termed as Tall Building. The quantity, sources and mode of water supply in tall building shall be in accordance with Sec 4.2. In high rise buildings fittings and equipment for firefighting may be subject to excessive pressure.

Pressure on firefighting equipment in Tall building shall be reduced by dividing the building into different zones. In this process the building shall be divided into different water supply zones so that the firefighting equipment will serve within their maximum allowable limit of pressure. Separate automatic fire pump or combination of tank and automatic pump shall be installed for supplying water to the firefighting equipment in each zone as per Figures 4.4.7 and 4.4.8.

4.6 Fire Detection and Alarm System

4.6.1 Fire Detection Shall be Done by the Following Ways

(a) Human surveillance:

Human surveillance shall be acceptable where the user and occupant are capable of maintaining surveillance for detecting fire and smoke when a person appointed and assigned to detect fire shall be termed as Fire watch.

(b) Automatic smoke or/and heat detection :

The installation of automatic fire and smoke detection system shall be a necessity when the size, arrangement and occupancy of a building become such that a fire itself cannot provide adequate warning to its occupants.

The automatic fire and smoke detection system shall include, spot or line type heat sensitive detectors and optical, ionized or chemical sensitive type of smoke detectors.

(c) Video surveillance :

Cameras capable of registering and transmitting real time images in to a monitoring device having display commonly termed as CCTV shall be installed systematically to cover an area for detecting any incision of smoke and fire. This CCTV will remain under either human surveillance or monitored by compatible software to transmit signal automatically to the fire alarm system and also to the authorized persons.

4.6.2 Fire Alarm System

4.6.2.1 In a fire incident, panic management shall be the prime concern for a successful relocation, delayed egress or evacuation of occupants from a building structure. Activation of alarm shall be sequential and compatible with all design scenarios. Means of egress system is so designed that all alarms of a building shall not be activated at a time. A general announcement of fire shall be done for the occupant or the word “Fire” shall be avoided but authorized persons responsible for evacuation shall be alerted through Password or Pass Phrase. As per design scenarios a systemic execution protocol shall be developed where a building shall be sub-divided into zones for installation alarms and for fight in place, relocation of occupants, delayed egress or immediate evacuation.

Alarm system can be of different types, such as audible alarm, visual alarm, vibration alarm, and display alarm.

(a) Audible alarm: Ringer, bell, horn, chime and voice command via public address system (PA system) are the examples of audible alarm system.

(b) Visual Alarm: A bright white light emitting device with specific intensity and cycle of emission is capable to draw attention of a person having limited hearing shall be termed as visual alarm. A visual alarm shall be installed where a person working alone in a room or a space having hearing limitations. In a public place or in any place more than two persons are present and one having normal hearing ability shall not require to install visual alarm.

(c) Vibration Alarm: Alarm activated through vibration can be used for alarm.

(d) Display Alarm: Textual, graphical or pictorial display on screens or monitors can be used as alarm.

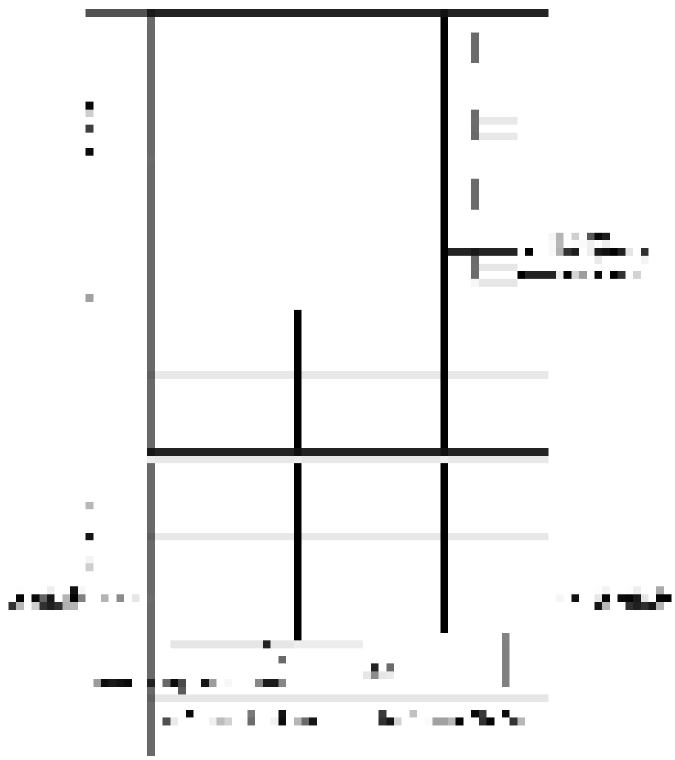


Figure 4.4.7 Typical diagram for fire protection in different water supply zones of a tall building

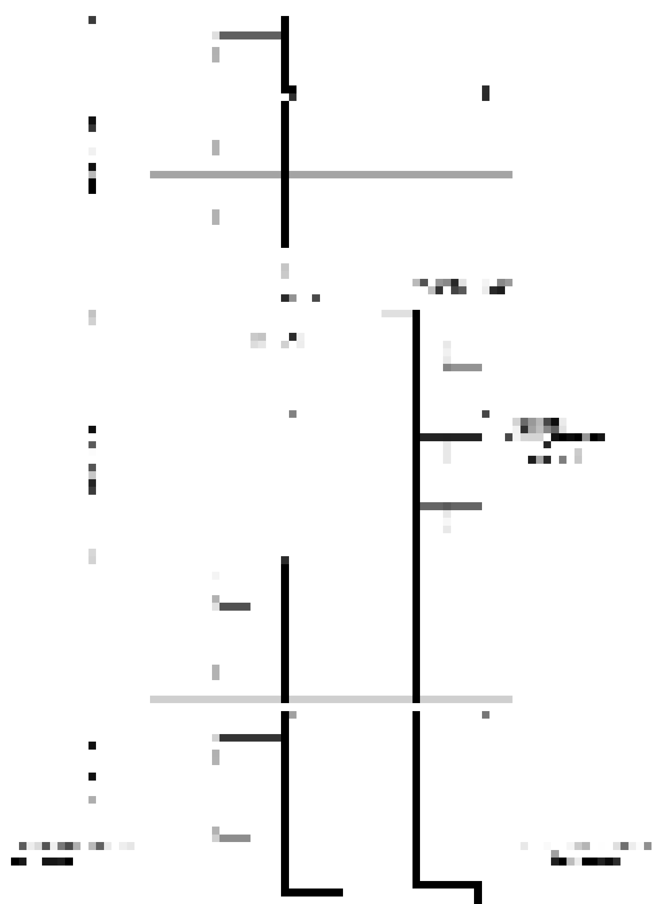


Figure 4.4.8 Typical diagram for fire protection in different water supply zones of a tall building

4.6.2.2 Each floor shall be separated as zone for the purpose of alarm annunciation.

4.6.2.3 A floor is subdivided by fire or smoke barriers and allows relocation of occupants from area of incident to another area on the same floor each area shall be considered as a zone and annunciated separately for the purpose of alarm location.

4.6.2.4 Notification zones shall be consistent with emergency response or evacuation plan for the protected premises. The boundaries of notification zones shall be coincident with building peripheral walls, fire or smoke compartment boundaries, floor separations or other fire safety subdivisions.

4.6.2.5 If required by the authorities having jurisdiction, the alarm system be allowed the application of alarm signal to one or more zones at the same time, shall allow voice paging to the other zones or in any combination.

4.6.2.6 Alarm annunciation at the fire command center shall be by means of audible and visible indicators.

4.6.2.7 Activation of fire extinguishment system shall have a supervisory alarm. An automatic extinguishment system capable of discharging other than water extinguishing agents shall have dedicated and distinct alarm system and shall be actuated before discharging such agents.

4.7 Related Appendix

Appendix C Detail Guidelines for Selection and Sitting of Fire Detection System

PART IV
Chapter 5
Requirements For Fire Detection and Extinguishing System

5.1 Scope

Installation of fire detection and firefighting equipment fixed centrally or localized or portable and their arrangement in the buildings shall be performance based. Construction type and occupancy classification of Buildings shall be as per provisions of this Code. Part 3 of this Code shall be determinant of construction type and the A-Z list for occupancy classification. Installation of fire detection and firefighting equipment shall comply with the Chapter 4 of Part 4 of this Code.

Intent of this Chapter is to reduce the probability of fire incident by confinement, extinguishment to reduce probability of injury or death from fire, structural failure due to fire and safety of building use.

Provisions of this Chapter shall be considered as minimum requirement and shall not be intended to prevent additional installation of higher standard of equipment.

5.1.1 Performance based fire protection system which includes "Passive" that is arrangement of building components and "Active" means detection, alarm, extinguishment devices and equipment which shall be incorporated in all buildings unless otherwise specified in this Code. Performance based design considerations shall be as follows:

(a) The starting of a fire incident shall be a single source to evaluate the fire protection system.

(b) The prime objective of a fire protection system to safe life and minimization of property damage shall be achieved by using required design scenarios and the performance criteria to be fulfilled. Each design scenario shall be challenging as realistic and the probability of occurrence is present in the building shall be reduced and protected.

(c) Design scenario shall include but not limited to those specified in Sections 5.1.2 to 5.1.4 and shall be documented and demonstrated to the satisfaction of the authorities having jurisdiction.

(d) Each design scenario used in the performance-based design shall be translated into input data specification as appropriate for calculation method or model.

(e) Input data of any design scenario did not analyzed and explicitly addressed or incorporated shall be omitted from input data specifications, shall be identified by a sensitivity analysis of the consequences of the modification for such omissions shall be performed.

5.1.2 Design Scenario I

Fire Class and Fire resistance rating shall be determined as per provision of this Code for the followings:

(a) All surface finish materials.

(b) Structural Members.

(c) Joints of Structural Members.

(d) All slabs.

(e) Roof Slab.

(f) Joints between Slabs.

(g) All Exterior Walls.

(h) All Interior Walls.

(i) Partitions.

(j) Suspended Ceiling.

Construction classification and the structural stability shall be concluded and documented.

5.1.3 Design Scenario II

Occupancy specific design scenario representative of a typical fire shall explicitly specify the following:

(a) Occupant activities.

(b) Number and location of occupants.

(c) Room size.

(d) Number of Control Area.

(e) Furnishings and contents.

(f) Fuel Properties represented by Fire Class and ignition sources.

(g) Ventilation conditions.

(h) First item ignited and its location.

5.1.4 Design Scenario III

(a) The largest possible fuel load characteristic of the normal operation of the building shall be considered regarding a rapid developing fire in presence of occupants.

(b) A slow-developing fire shielded from protection in the close proximity to a high occupancy area shall be considered a concern regarding a relatively small ignition source causing a significant fire.

(c) A concealed space or suspended ceiling space adjacent to a large occupied room shall be considered a concern regarding a fire originating in a concealed space that does not have either detection system or suppression system and then spreading into the room within holding the greatest number of occupants.

(d) An Ultrafast developing fire in the main exit access portion in a condition when interior doors are open but reduction in number of available of means of egress shall be considered.

(e) A room normally unoccupied from where a fire starts that can potentially endanger a large number of occupants in a room or other area shall be considered.

(f) The concern regarding exposure of fire outside of an area of incident started at a remote location either spreading from the area or bypassing barriers spread into another area and developed untenable condition thereof.

(g) The reliability and the design performance shall be considered for fire detection and protection system in such a way that a fire originating in ordinary combustibles in a room with each passive or active fire protection system or fire protection feature independently rendered ineffective shall be considered individually being unreliable or becoming unavailable. This scenario shall not be considered for a room or a space or a building where fire detection and protection systems or any independent features are absent.

5.1.5 Fire class shall be determined for all movables in each room and all control areas in the building.

5.1.6 Fire Protection Plan

A building or part thereof must have a fire protection plan for the following cases.

(a) High rise building or building sections 33 m and above in heights.

(b) Building or building sections classified in the occupancy groups G, H, J, K and M which are two or more storey in height with over 1858 m^2 per gross floor area or are two or more in height with total area exceeding 4717 m^2 gross floor area.

(c) Building classified as A3 containing 30 or more dwelling units; A4 and A5 having gross floor area of the building more than 1200 m^2.

(d) Part of a building used as mercantile, assembly, institutional or health care having gross floor area of the building over 930 m^2.

(e) Alteration to a building or a portion thereof listed in Sections 5.1.6(a) to 5.1.6(d) above, if cost of alteration equivalent to one third cost of new construction of the same or more or involves changes in occupancy classification.

(f) The plan shall include information where applicable building address, height in meter, occupancy classification, detail occupant load.

(g) Key Plan shows all floors, exits, corridors, partitions serving as fire separations or compartments, locations and ratings of required enclosures, windowless stair with pressurization, exit discharge, locations of frontage space including street width of abutting plot.

(h) Descriptions in narrative forms of safety systems and features where applicable, including:

- Communications systems
- Alarm system
- Detection systems
- Location of fire commend station
- Elevator recall
- Emergency lighting and power
- Extinguishing equipment
- Compartmentation
- Horizontal exits
- Mechanical ventilation and air conditioning

- Smoke control systems and equipment
- Furnishing type and materials
- Places of assembly
- Fire department access
- Other system, required or voluntary to be installed

(i) A fire protection plan shall be signed by the same architect who is signing on the proposed drawings for building approval and any person responsible for the Fire protection design.

5.2 Specific Recommendations

Specific recommendations applicable for buildings complied with the followings:

5.2.1 All building constructed monolithically as per provisions of this Code as an inherent full fire resistive construction type shall be termed as Type I-A.

5.2.2 All surface finishes shall be Class-I within the range of zero to twenty five flame spread index.

5.2.3 Any offsite construction, pre-stressed, pre-fabricated or steel structure encased with fire resistive assembly shall be termed as Modified Type I-A.

5.2.4 The following recommendations for fire protection system specified in Sections 5.3 to 5.14 are made based on construction type and surface finishes specified in Sections 5.2.1 and 5.2.2 respectively.

5.2.5 All buildings of any occupancy type and construction type as per provisions of this Code other than Sec 5.2.1 with all surface finish as per Sec 5.2.2 shall provide a performance based fire protection.

5.3 Occupancy A: Residential

The residential buildings complied with Sections 5.2.1 and 5.2.2 shall provide the following active fire protection:

5.3.1 Occupancy A1 and A2: Single Family Dwelling and Two Families Dwelling

(a) For buildings having total floor area less than 500 m^2, fire detection and fixed firefighting arrangements is not required.

(b) Buildings exceeding total floor area 500 m^2 shall have manual alarm system and portable extinguishers provided in the escape stairs route or in lift lobby and as per provision of this Code.

5.3.2 Occupancy A3: Flats and Apartments

(a) Up to 33 m height fire detection and fixed firefighting arrangement shall not be required.

(b) No protection is required within the dwelling units of high rise flats and apartments; manual alarm system and fixed hydrant system shall be provided in the landings of fire stairs or in the lift lobby as per the provisions of this Code.

5.3.3 Occupancy A4: Mess, Boarding House and Hostels

(a) For buildings up to 2 storey height, fire detection, fire alarm and fixed firefighting arrangements shall not be required.

(b) Buildings having 3 stories and having floor area less than 300 m^2 shall not require fire detection and fixed firefighting arrangements.

(c) The floor area of 3 stories building having more than 300 m^2 per floor and less than 33 m height having central corridor with rooms on both sides, manual fire alarm system shall be provided along with portable fire extinguishers. Instead of double loaded corridor a single loaded corridor having 3 m width shall not require any detection and fixed firefighting arrangements.

(d) High rise boarding house, mess and hostels manually operated electric fire alarm system shall be provided along with hydrant system.

5.3.4 Occupancy A5: Hotels and Lodging Houses

(a) For buildings up to 2 storey height, fire detection, fire alarm and fixed firefighting arrangements is not required.

(b) Buildings having 3 floors or above and having floor area less than 300 m^2 shall not require fire detection and fixed firefighting arrangements.

(c) The floor area of such building is more than 300 m^2 per floor and low rise building having central corridor with rooms on both sides, manually operated fire alarm system shall be provided along with portable fire extinguishers. For low rise buildings with other configurations performance based firefighting system shall be required as per the provisions of this Code.

(d) High rise hotels and lodging houses manually operated electric fire alarm system shall be provided along with hydrant system.

5.4 Occupancy B: Educational

The educational buildings complied with Sections 5.2.1 and 5.2.2 shall be provided with the following active fire protection:

5.4.1 Low rise buildings with open corridor of 3m width fire detection and fixed firefighting arrangements shall not be required.

5.4.2 High rise building or building having central corridor with classrooms on both sides, manual fire alarm and hydrant systems shall be required as per provisions of this Code. Single loaded open corridor having width of 3 m or more shall have detection and manual alarm systems.

5.4.3 Where hydrants cannot be used to extinguish fire in those areas appropriate portable firefighting appliances shall be installed as per standard.

5.5 Occupancy C: Institution For Care

The Institution for care buildings complied with Sections 5.2.1 and 5.2.2 shall be provided with the following active fire protection:

5.5.1 Occupancy C1: Institution for Care of Children:

Fire detection and fixed firefighting arrangements shall not be required. Portable firefighting appliances shall be installed as per the provisions of this Code.

5.5.2 Occupancy C2: Custodial Institution for the Physically Capable adults:

Fire detection and fixed firefighting arrangements shall not be required. Portable firefighting appliances shall be installed as per the provisions of this Code.

5.5.3 Occupancy C3, C4, C5: Custodial Institution for the Physically Incapable, Penal and mental institutions for children and Penal and mental institutions for adults:

Manually operated electric fire alarm system shall be installed. Portable firefighting appliances shall be installed as per the provisions of this Code.

5.6 Occupancy D: Health Care Facilities

The Health care facilities buildings complied with Sections 5.2.1 and 5.2.2 shall be provided with the following active fire protection:

5.6.1 Occupancy D1: Normal and Emergency Medical Facilities:

(a) Manually operated electric fire alarm system or automatic fire alarm system shall be installed in the duty room, so that the duty personnel receive the fire warning well in advance. Portable fire fighting appliances shall be installed as per the provisions of this Code.

(b) For low rise health care facility buildings with more the 300 m^2 per floor, performance based fire fighting system shall be required as per the provisions of this Code.

(c) For high rise health care facility buildings, manually operated electric fire alarm system shall be provided along with hydrant system.

5.7 Occupancy E: Business

The Business buildings complied with Sections 5.2.1 and 5.2.2 shall be provided with the following active fire protection:

	Buildings	Active Fire Protection
(i)	Office buildings up to 2 storey high and 500 m^2 single effective undivided space in a floor.	Portable fire extinguishers or hydrants.
(ii)	Office buildings more than 2 storey high or more than 500 m^2 single effective undivided space in a floor.	Manually operated electric fire alarm system shall be provided along with portable fire extinguishers or hydrants.
(iii)	Laboratories with precession instruments.	Automatic fire alarm system and performance based extinguishing system.
(iv)	Control areas of office buildings dealing with flammable liquids.	Automatic foam or gaseous or dry chemical fire extinguishing system required along with portable fire extinguishers.
(v)	Solvent storage in a control area of an office	Automatic fire alarm system and performance based foam or gaseous or dry chemical fire extinguishers or portable fire extinguishers.
(vii)	Telecommunication, Internet gateway equipment or computer installation in an unattended server room.	Automatic fire alarm system and performance based fixed gaseous or fixed vaporizing liquid extinguishers or portable fire extinguishers.
(viii)	Electrical low tension distribution panel room in a sub-station.	Automatic fire alarm system and performance based localized fixed gaseous or vaporizing liquid extinguisher or portable fire extinguishers.
(ix)	Space under one false ceiling more than 500 m^2	Automatic fire alarm system shall be installed for above and under the false ceiling.

	Buildings	Active Fire Protection
(x)	Essential Services (Occupancy E3)	Due to importance of services and the functionality of the building of this occupancy classification during any national or local emergency situation thus the fire protection system design shall be performance based (Sec 5.1.1).
(xi)	High rise office buildings	Manually operated electric fire alarm system shall be provided along with hydrant system.

5.8 Occupancy F: Mercantile

The Mercantile buildings complied with Sections 5.2.1 and 5.2.2 shall be provided with the following active fire protections:

5.8.1 Occupancy F1: Small Shops and Markets

	Mercantile	Active Fire Protection
(i)	Whole sale establishments, transport booking establishments.	Manual fire alarm system shall be provided along with portable fire extinguishers or hydrant.
(ii)	Other premises (other than shops, stores, markets etc.)	Manual fire alarm system shall be provided along with portable fire extinguishers or hydrant.

5.8.2 Occupancy F2: Large Shops and Markets

	Mercantile	Active Fire Protection
(i)	Shopping arcade with central corridors open to sky	Manual fire alarm system and portable fire extinguishers shall be provided or hydrant.
(ii)	Mercantile building under covered roof with single effective undivided space more than 500 m^2 on each floor	Manual fire alarm system and hydrant system with performance based portable fire extinguisher shall be installed.
(iii)	Underground mercantile structure	Automatic fire alarm system, sprinklers and standpipe with performance based portable fire extinguisher shall be installed.

5.8.3 Occupancy F3: Petrol and CNG Stations

	Mercantile	Active Fire Protection
(i)	Petrol pump and CNG station, automobile garages	Fixed automatic foam or gaseous or dry chemical fire extinguishing system shall be provided along with portable extinguisher.
(ii)	Aircraft hangars	Automatic foam or gaseous or dry chemical fire extinguishing system shall be provided along with portable extinguisher.

5.9 Occupancy G: Industrial

The Industrial buildings complied with Sections 5.2.1 and 5.2.2 shall be provided with the following active fire protection:

5.9.1 Occupancy G1: Low Hazard Industries

Manually operated electric fire alarm system shall be installed with portable fire extinguishers or hydrants when occupant loads are not more than 150.

Where occupant loads are more than 150 active fire protections shall be performance based.

5.9.2 Occupancy G2: Moderate Hazard Industries

Among the moderate hazard industries where large number of occupants are densely populated in a building, the active fire protections shall be performance based. Fire safety requirement for such type industry is elaborated as follows:

(a) Where occupancy load is more than 150 per production area shall have minimum 9.5 m^3 air volume per occupant.

(b) There shall have direct exits from the ground floor. This exit doors shall be used by only the occupant of the ground floor.

(c) Buildings less than 33 m in height shall have open stair and the interior stairs shall be protected by fire rated enclosures. Occupants located 33 m or above, all stair shall have smoke proof enclosures constructed as per provision of the Code.

(d) All windows or openings on exterior walls passable by occupant located above 3 m in height shall be protected by grills and all these grills shall be designed as such that a part or a portion having minimum 0.6 m height and minimum 0.75 m width framed and the grill within the frame shall be side hinged or pivoted so that it can swing. This swing type operable portion must be always locked and in case of emergency the firefighters can open by breaking the lock for rescue operation.

(e) The floor shall be constructed such that the travel path of the occupant shall not be exceeded as per Table 4.3.7 of this Code.

(f) As per general requirements, all exit access doors shall be of a side-swinging type. Fulfilling the conditions laid down by NFPA 101, edition 2015, article 7.2.1.4 horizontal sliding or vertical-rolling security grills or door assemblies that are part of the required means of egress shall be permitted.

(g) All raw materials, finished good and accessories shall be stored in control areas as per provision of part 3.

(h) Density of storage materials per control area shall not be exceeded the provision of this Code.

(i) During production that is feeding, checking for quality control rejects, waiting area for finishing, packing, cartooning etc. in every case dedicated area shall be defined as on process storages. The total volume of materials on process shall be such that in every four hour the material shall be used up and the finished goods shall be transferred to controlled area as finished goods store.

(j) From each end every work station shall be connected with a passage. The width of the passage shall comply with the provision of this Code Chapter 3 Part 4.

(k) Cargo lift and passenger lift shall have smoke proof lift lobby.

(l) Occupant load in a single effective undivided space shall not exceed 600. In case of existing building if the occupant load of a single effective undivided space exceeds 600, the space shall be compartmented complying with the horizontal exit provision of the Code.

(m) Where control areas and in process stores having materials may cause a fire classified as fire class A shall have hydrant system as per provision of this Code. In the utility occupancy areas fire extinguishing system shall be installed as per provision as specified for utilities of this Code.

(n) If there any change of fire classification due to the working condition or raw materials than appropriate extinguishing system shall be installed as per provision of this Code.

(o) Up to 750 m^2 single effective undivided space in a floor shall be installed with manual fire alarm system with portable fire extinguishers or as an alternate hydrants system shall be installed as per provisions of this Code.

(p) Above 750 m^2 single effective undivided space in a floor shall be fitted with manual fire alarms system with hydrants shall be installed.

5.10 Occupancy H: Storage

The Storage buildings complied with Sections 5.2.1 and 5.2.2 shall be provided with the following Active fire protection:

5.10.1 Occupancy H1: Low Fire Risk Storage

Manually operated electric fire alarm system shall be installed. Depending on the type of materials to be stored, performance based fire protection shall be installed as per provision of this Code.

5.10.2 Occupancy H2: Moderate Fire Risk Storage

Performance based fire protection system shall be installed as per provision of this Code.

5.11 Occupancy I: Assembly

The Assembly buildings complied with Sections 5.2.1 and 5.2.2 shall be provided with the following Active fire protection:

5.11.1 Occupancy I1: Large Assembly with Fixed Seats

All auditorium, corridor, green rooms and canteen attached to assembly buildings shall be fitted with manual fire alarm system and the performing stage should preferably be covered by an automatic sprinkler system. Portable firefighting appliances shall be installed as per specification of the manufacturer and provision of this Code.

5.11.2 Occupancy I2: Small Assembly with Fixed Seats

Requirements specified in Sec 5.6.1 shall be complied.

5.11.3 Occupancy I3: Large Assembly without Fixed Seats

Automatic fire alarm system shall be provided. Portable firefighting appliances shall be installed as per specification of the manufacturer and provision of this Code.

5.11.4 Occupancy I4: Small Assembly without Fixed Seats

Requirements specified in Sec 5.6.3 shall be complied.

5.11.5 Occupancy I5: Sports Facilities

Manually operated electric fire alarm system shall be provided. Portable firefighting appliances shall be installed as per specification of the manufacturer and provision of this Code.

5.12 Occupancy J: Hazardous

The Hazardous buildings complied with Sections 5.2.1 and 5.2.2 shall be provided with the following Active fire protection:

All hazardous occupancies shall be installed with automatic fire alarm and automatic fixed firefighting gaseous or foam or dry chemical extinguishing system as compatible with class of fire shall be installed as per provision of this Code.

5.13 Occupancy K: Garages

The parking buildings (garages) complied with Sections 5.2.1 and 5.2.2 shall provide the following fire protections:

(a) Where both parking and repair operations are conducted in the same building, the entire building shall comply with the requirement stated in this Code for Occupancy G1.

(b) Where the parking and repair sections are separated by not less than 1-hour fire-rated construction, the parking and repair sections shall be permitted to be treated separately.

(c) In areas where repair operations are conducted, the requirement of Occupancy G1 shall be fulfilled.

(d) The area used only for parking shall fulfill the requirement as laid down in chapter 42 of NFPA 101 edition 2015.

5.14 Occupancy L: Utilities

Fire protection system shall be as stated in Sec 2.12 of this Code.

5.15 Occupancy M: Miscellaneous

Performance based fire protection system shall be installed.

PART IV
Appendix A
Guidelines For Fire Drill and Evacuation Procedure

A.1 Introduction

The following provisions shall be applicable for emergency reporting, fire safety and evacuation plan of the occupants of different occupancies.

A.2 Fire Reporting

Any occupant within the occupancy discovering a fire or smoke shall immediately report the incident to the fire brigade directly or through the ground command station, if there is any. Reporting of this situation shall not be delayed by any person by way of making, issuing, posting or maintaining any regulation or order written or verbal to that effect.

A.3 Supervision of Fire Safety and Emergency Action and Plans

The owner shall designate competent persons to act as fire safety and evacuation plan staff, train the staff and conduct fire drill. Such persons shall possess such qualifications and/or hold such certificate of fitness as are required by the provisions of this Chapter. The owner shall ensure that adequate fire safety and evacuation plan staff is present on the premises during regular business hours and other time when the building is occupied, to perform the duties and responsibilities set forth in the fire safety and evacuation plan.

A.4 Fire Safety Staff

A.4.1 The fire safety and evacuation plan shall designate a fire safety director, a number of deputy fire safety directors and fire safety brigade members having following duties, authority and qualifications.

A.4.2 Fire Safety Director

(a) The fire safety plan shall contain the name of fire safety director, whether employed by a fire security firm or directly employed by the management.

(b) Depending on the size and complexity of the building, the Fire Safety director shall be a person of proven capability, having good training and schooling with adequate experience in dealing with fire.

(c) The fire safety director shall be present in the building during regular business hours. Duties of Fire Safety director shall primarily include but not be limited to the following :

(i) Shall be well conversant with the written fire safety plan for the fire drill and evacuation procedures.

(ii) Shall be in charge of selecting qualified building service employees for the fire command and engage in organizing, training and supervising the works of command crew.

(iii) Shall be responsible to conduct fire and evacuation drill.

(iv) Shall be responsible for the availability and state of preparedness of fire command crew during emergencies.

(v) Shall be responsible for the assignment and training of Fire fighters on floor supported by adequate number of deputies as detailed out in the fire safety plan.

(vi) Shall be responsible for the day to day supervision of the fire fighters and his deputies and the state of alertness of the fire fighters. When the number of fire fighters and deputies become such that it becomes impractical for the chief fire safety officer to check them directly during the working hours, he may provide substitute. Nonetheless the fire safety director shall spot check any number of floors as he wishes or time permits. An up to date organization chart shall be displayed at appropriate locations.

(vii) Cases of negligence to duties on the part of members of his crew shall be taken up by him and he shall rectify the situation by appropriate measures as far as he has been empowered under the fire safety plan, failing which he shall notify the matter to the owner or the management of the building. The owner or the management on their part shall take up the matter with the fire security firm or if employed directly shall deal with the matter directly. If the person/persons is/are employed by a firm, and the firm fails to correct the situation, the owner/management shall notify the matter to the Department of Fire Service and Civil Defence to take disciplinary action against the firm.

(viii) In the event of fire/emergency he/she shall be in charge of fire command station and shall supervise, guide and coordinate activities such as ensuring that the Department of Fire Service and Civil Defence has been notified of fire or fire alarm, direct the evacuation procedure as detailed in the fire safety plan, manning the fire command station, appraise the Department of Fire Service and Civil Defence about the spot of fire on their arrival, advise the Department of Fire Service and Civil Defence officer in charge of the operation.

A.4.3 Fire Safety Deputy Director

The fire safety plan shall contain the details of Deputy Fire Safety Director similar to the details mentioned under the fire safety director. Qualification and experience of Deputy Fire Safety Director shall also be similar to those of the Fire Safety director excepting that he shall be less experience than the Director.

Tenant or tenants of each floor upon request by the owner or in-charge of the building shall assign and make available dependable and trustworthy person/persons under their employee at the disposal of the Director to act as fire safety coordinator and fire fighter. They shall undergo basic firefighting and evacuation training by the Director or his deputy.

Duties of the Deputy Fire safety Director shall be similar to those mentioned under Sec A.4.2 except that he shall receive command from the Fire Safety director for execution and shall assume the role of Fire Safety director in his absence.

Each floor of a building shall be under the command of a deputy fire director for the safe evacuation of inmates in the case of fire. When the floor area of a tenant exceeds 700 m^2, a deputy fire director shall be assigned for each 700 m^2 or part thereof.

The deputy fire safety director shall be present in the building at all times. Duties of deputy Fire Safety director shall primarily include but not be limited to the following :

(a) Each Deputy Fire Safety director shall be conversant with the fire safety plan. They must be well acquainted with fire exits and location and operation of fire alarms.

(b) In case of fire or fire alarm, the deputy Fire Safety director shall ascertain location of fire and unfold evacuation procedure as directed from the command station and to the following general guides.

(i) The most critical area for immediate evacuation would be the fire floor and the floors above. Evacuation from other floors shall be initiated if so commanded by the ground command station or the situation indicates to be so. Evacuation should be carried out via stairs not influenced by fire and fire fighter shall try to carry out the operation using stair other than the ones used by the Department of Fire Service and Civil Defence personnel. If it becomes impossible, the fighters before opening door to the fire floor shall sought advice from the Department of Fire Service and Civil Defence personnel.

(ii) Evacuation from two or more floors below the fire floor should be adequate. He shall continuously keep the ground command station informed of his location.

(iii) Ensure that fire alarm has been transmitted.

(iv) Fire fighters shall ensure that all the inmates are intimated of the emergency and shall immediately proceed with the evacuation exercise detailed under Fire Safety Plan.

(v) Fire fighter shall keep the ground station informed of the step being taken by him/her.

(vi) Similarly fire fighter above fire floor shall notify the command station of the means being taken by him/her or any other special feature after unfolding Fire Safety Plan.

(vii) If and when stairways serving fire floor/floors above become useless by the presence of fire, smoke, fumes, in several floors above and when fire engulfs a considerable number of inmates then use of elevators shall be considered in accordance with the followings:

- If the elevator serving the floor to be evacuated also serves the fire floor, the lifts shall not be used if it is not fire lift. If there are more than one lift bank, however, the lift/lifts in the other bank may be used if notified by the ground command station that one may use such lift/lifts.
- If the lifts do not serve the fire floor or lift shaft has no opening on the fire floor, they may be used if not otherwise instructed by the command station.
- Elevators taken over by trained in-house person or Department of Fire Service and Civil Defence personnel may be used.
- In absence of unaffected available lift/lifts, Fire fighter shall decide to use the fire stair for evacuation based on considerations/ information available on the floor and any other instruction received from ground command. Before entering the fire stairway with the evacuees, the Fire fighter shall be sure about the

environment within the fire stairway by personal inspection and in case of adverse environment consider using an alternate stairway and shall notify the ground command accordingly.

- The Fire fighter shall keep the ground command informed of the means adopted by him during the evacuation process.

A fire safety coordinator and fire fighter shall be available at all times other than normal working hours when the Fire Safety Director or his Deputy is not available within the building.

Fire safety coordinator shall be a person capable of directing the evacuation procedure of occupants within the buildings as detailed in the Fire Safety Plan.

During fire/emergencies, primary function of fire safety coordinator shall be to take over command of the ground station and to direct and execute the evacuation process as laid down in the plan.

Fire safety coordinator shall be trained by the Director and shall be under his command for all evacuation purposes. His activities shall be controlled and governed by the clauses in Fire Safety Plan and shall be subject to scrutiny of the Department of Fire Service and Civil Defence.

Fire Party: If, in the opinion of the Fire Safety Director and endorsed by the Department of Fire Service and Civil Defence that the number of fighter, coordinator and Deputy are inadequate, a Fire Party shall be raised from among the employees of the tenants and the management who shall be acting as help to regular in-house fire fighting force in the event of fire and follow the same work schedule and function in the same manner as fighter, coordinator and Deputy fire safety director do.

A.5 Signs And Floor Plans

A.5.1 The lettering, arrows and other symbols of exit signs shall be written with vernacular alphabets high contrast background as per NFPA 170. Words on the signs shall be at least 150 mm high with a stroke of not less than 20 mm. For vernacular alphabets and numerics at least 150 mm high with stroke of not less than 20 mm. The sign/signs may be posted directly above the call button of the lift or any other conspicuous location securely attached to the surface of the wall. The top of the sign shall not be more than 2 m above floor level.

Sign shall be posted and maintained in front of the landing area of lifts on all floors that occupants may not miss, which shall direct the occupants to use stairs and not lifts during emergencies/fires, if not directed otherwise and shall also contain a floor plan with exact location of the stair and the relative position between the sign and the stair. Such posting in front of the landing area of lifts shall be omitted only if such signs are posted on all floors and some other area conspicuously located with the same message inscribed on it.

A.5.2 Sign Depicting Floor Number

A sign shall be posted and maintained on each stair enclosure preferably on the wall of the intermediate landing which in actual fact shall be half storey more or less than the actual indicating the floor number. The number shall be at least 75 mm square and in vernacular alphabets with contrast background as per NFPA 170.

A.5.3 Stairs and Elevators Identification

Each stair and Elevator shall be identified by a vernacular alphabet and posted with a sign, securely placed preferably on the wall of the stair side of the lift door from which egress is to be made.

A.5.4 Stair re-entry Provision

A sign shall be posted and maintained on each floor within stairway and on the occupancy side of the stairway where required, indicating whether re-entry is provided into the building and the floor where such re-entry is provided.

A.5.5 Command Station

Command station on the ground floor shall be provided with a detailed floor plan of the entire building including detailed locations of all first aid, firefighting equipment and other pertinent information. Command stations shall be adequately illuminated.

A.5.6 Two Way Communications and Fire Alarm

A two way communication system between each floor and the command station on the lobby of the entrance floor shall be provided and maintained by the owner of the building. Similarly fire alarm on each floor and the command station shall be fitted and maintained.

A.6 Fire Safety Plan

A.6.1 A fire safety plan shall be developed in line with the details elaborated as below and must have the approval of the local Department of Fire Service and Civil Defence regarding its adequacy.

A.6.2 Fire safety plan elaborates the purpose and objective of the plan with details of personnel and their duties and fire drilling and evacuation plan. In developing fire safety plan, evaluation of all individual floor layout, total occupancy load on each floor, number and kinds of exits available, zoning of the floor by area and occupancy shall be taken into consideration, careful evaluation of occupant movements and the most expeditions routes to exit and alternate routes shall be identified and taken into consideration.

A.6.3 Fire safety plan starts with the location, address of the building with telephone number and details of any other communication facilities available within the building.

A.6.4 Purpose of the plan is to delineate details of systematic safe and orderly evacuation of a part or whole of the building by its occupants in case of fire/emergency in the shortest possible time to a safe area through the safe means of egress. It also details out the use of in-built facilities of fire warning and firefighting like fire alarm, first aid hose etc. to safeguard the lives of the inmates of the building.

A.6.5 Objective of the plan is to provide continued education to the inmates and the fire command personnel and keep the people oriented to the in-built equipment in readiness to act in the event of fire. The plan shall be rehearsed through fire drill and the written plans containing instruction shall be updated if needed and use of the in-built equipment along with initiating fire safety procedure to safeguard life in case of fire until the fire brigade arrives.

A.6.6 Once the plan is accorded after approval by the Department of Fire Service and Civil Defence, the plan shall be distributed to all the tenants of the building by the building management, including the employees of the tenants and employees of the management.

A.6.7 If the building is owned by an individual or a single corporate body and the owner or right holding member/members of the corporate body are residing in the building shall be equally subject to fire safety plan applicable to other tenants.

A.6.8 All major changes in the safety plan shall be promptly reported to Department of Fire Service and Civil Defence for their approval.

A.7 Fire Drills

A.7.1 Fire drill shall be conducted as detailed under the fire safety plan. The frequency of fire drill shall be as per table shown below. All occupants of the buildings, building service employees including fire safety and evacuation plan staff shall participate in the fire drill. However, the very old, convalescent patients or otherwise incapacitated inmates are not obliged to actively take part in the exercise, except the fire man and his staff and family members of such person shall chalk out a clear plan as to how to evacuate in a real emergent situation with such incapacitated persons.

A.7.2 A record of such drills shall be kept in writing for at least 3 years for the inspection Department of Fire Service and Civil Defence whenever called for. The frequency of such fire drill shall be as mentioned in Table 4.A.1.

Table 4.A.1: Fire Drill Frequency

Occupancy	Frequency
Industry Having occupancy more than 150	Monthly
Industry Having occupancy less than 150	Quarterly
Mercantile occupancy more than 150	Quarter
Mercantile occupancy less than 150	Half yearly
School, College, Universities	Half yearly
High rise building	Half yearly
Tall building	Quarterly

A.8 Organization Chart For Fire Drill And Evacuation Assignment

(a) An organization chart clearly delineating assignment attributed to designated employees shall be prepared as per fire safety plan and posted to all tenants and in very conspicuous location/locations on each floor. A copy of the chart shall be in possession of the fire safety director.

(b) An updated list shall be continuously made available with the director, his deputy and coordinators and Fire fighter for all the disabled occupants unable to move without aid in the stairs. Arrangement shall be made in detail in the fire safety plan to have these inmates assigned in moving down the stairs two or more floors below fire floor. If it becomes necessary to move them still further down the stair, help may be sought of the elevator bank unaffected by fire and evacuated safely to ground floor. In case any extra assistance is needed, the director shall be notified.

(c) During fire or fire drill exercise, fire fighter shall be using arm band or such other identification.

(d) During fire on the fire floor it is to be ensured that all inmates are notified and are evacuated to safe area. A rush search shall be carried out including lavatories that all the inmates have been covered and the person in charge of this operation shall be trained in accomplishing this task fast and flawless.

(e) Persons not available on duty as per organization chart shall be promptly replaced as per contingency plan detailed in the fire safety plan.

(f) On completion of evacuation operation, a head count shall be carried out of all the regular occupants known to have occupied the floor evacuated.

(g) Immediately on receipt of the alarm, the fire fighter shall take position near the two way communication station on the floor, so that he/she can maintain continuous contact with the ground command and receive instructions.

A.9 Instruction to Inmates of The Building

Once the fire safety plan has been approved by the Department of Fire Service and Civil Defence, the applicable portion of the plan shall be distributed to all the tenants and the management of the building who in turn shall pass it on the their respective employees. All the occupants shall actively participate and cooperate in carrying out the provisions of fire safety plan.

A.9.1 Fire Prevention and Protection Program

A Plan for periodic formal inspection of each floor shall be developed in respect of exit facilities, fire extinguishers and good housekeeping. Reports of such inspection shall be carefully maintained for inspection of Department of Fire Service and Civil Defence. The Plan shall have provision for monthly testing of two way communication and fire alarm system.

A.9.2 Personal Fire Instruction Card

All the occupants of the building shall be supplied with a personal Fire Instruction Card containing details of the floor plan and exit routes as well as instruction to be followed during fire. Instructions may contain the following either in Bangla or both in Bangla and English.

A.9.3 Detailed Building Information

A form shall be maintained for the benefit of all concerned with fire hazard of the building and shall contain the following basic information :

(a) Building address in adequate details about its location.

(b) Name, Address and telephone number of the owner (corporate body or individual) and the person in charge of the building.

(c) Name address and telephone number if any, of the Fire Safety Director and his Deputy.

(d) Certificate of occupancy.

(e) Height, area, construction class (details of various load and non-load bearing elements).

(f) Number, type and location of fire stairs and/or fire towers.

(g) Number, type and location of horizontal exits or other refuge areas.

(h) Number, type location and operation of elevators and escalators (if any).

(i) Locations of fire alarm-floor wise and central.

(j) Communication System (telephone, mobile, walkie-talkie).

(k) Size and location of stand pipe system, gravity or pressure tank, fire pump and the name and qualifications of the person or persons in charge of the facilities.

(l) Automatic fire sprinkler system, primary and secondary water supply system and the area or areas being protected along with the name and qualification of the person or persons in charge.

(m) Any other fire extinguishing system, their location, efficacy and other pertinent details.

(n) Average number of employed persons by day and night.

(o) Average number of disabled persons visiting the building by day and night.

(p) Average number of outsiders visiting the building by day and night.

(q) Locations, types and capacities of other service facilities like primary and standby electric power, normal and emergency lighting arrangement, heating with fuel (if any), ventilation with fixed windows, other means of emergency exhaust facilities of smoke and heat, air-conditioning system including floor coverage and ducting, refuse disposal facilities, any other firefighting equipment, any other service facilities available.

(r) Measures taken or to be taken for addition, alteration and repair of any aspect within the buildings.

(s) Information on flammable solids, liquids and gases if used and stored within the building premises.

(t) In mixed occupancy, complete details of such occupancies and their special needs to be covered during fire or emergencies.

A.10 English Text of Instructions

(a) Safety First

- Push button fire alarm boxes (number is mentioned here) are provided on your floor. Please read the operating instruction posted on them.
- Please read the operating instructions on the body of the fire extinguisher provided in your floor.
- Nearest exit from your flat is shown in this plan (plan to be provided here).
- Assemble on the ground floor at the location indicated on the following plan. For clarification, contact the fire fighter or Deputy Safety Director. Plan of assembly point in ground floor to be provided here.

(b) For personal and collective safety, notify the fire fighter/Deputy Safety Director in case.

- Exit route and/or door are obstructed by dumping of boxes or such other loose materials.
- Staircase door, lift lobby doors do not close automatically or completely.
- Push button fire alarm or fire extinguisher are obstructed or damaged or seem to be out of order.

(c) If you discover a Fire

- Break the glass and push the button of the nearest fire alarm and call the fire service.
- With assistance from the floor fire fighter if needed, fight fire with the in-built facilities on your floor.
- Evacuate, if so instructed by the fire fighter

(d) When you hear Evacuation Instructions

- Immediately leave the floor taking the nearest staircase.
- Report to your fire fighter on reaching the predetermined assembly point outside the building.
- Try not to use lifts.
- Avoid going to cloak room.
- Refrain from running or shouting, do not get panicked.
- Do not waste a moment collecting personal belongings.
- Keep the lift lobby and staircase doors shut.

A.11 Bangla Text of Instructions (বাংলায় নির্দেশাবলী)

(ক) নিরাপত্তাই সর্বাগ্রে

- ভবনের প্রতি তলায় চাপ বোতাম বিশিষ্ট অগ্নি বিপদ সংকেত যন্ত্র দেয়া আছে। ব্যবহারের পূর্বে যন্ত্রের গায়ে মুদ্রিত নির্দেশাবলী পড়ুন।
- অগ্নিনির্বাপণ যন্ত্র ব্যবহারের পূর্বে অনুগ্রহ করে যন্ত্রের গায়ে মুদ্রিত নির্দেশাবলী পড়ুন।
- ভবনের যে স্থানে আপনি অবস্থান করছেন সেখান থেকে নির্গমণের নিকটতম/ সহজতম পথ খুঁজে পেতে নির্গমন নকশা অনুসরণ করুন।
- নির্গমনের সুবিধার্থে ভবনের নিচতলায় নির্গমন নকশা নির্দেশিত স্থানে সমবেত হউন ও অগ্নিনির্বাপণ কর্মীর নির্দেশনা অনুসরণ করুন।

(খ) আপনার ব্যক্তিগত ও সামগ্রিক নিরাপত্তার স্বার্থে নিম্নে উল্লিখিত বিষয়ে অগ্নিনির্বাপক কর্মীকে অবহিত করুন

- জরুরী নির্গমন পথে কোন প্রকার বাধা থাকলে।
- সিঁড়ি ঘরের দরজা, লিফ্‌ট লবির দরজা সম্পূর্ণভাবে বা স্বয়ংক্রিয়ভাবে বন্ধ না হলে।
- অগ্নিনির্বাপণ বিপদ সংকেত যন্ত্র ও অগ্নিনির্বাপণ যন্ত্র অকেজো বা ব্যবহার উপযোগী না থাকলে।

(গ) আগুনের উৎস খুঁজে পেলে

- বিপদ সংকেত যন্ত্রের কাঁচের আবরণ ভেঙ্গে ফেলুন, বোতামে চাপ দিন এবং ফায়ার সার্ভিসে খবর দিন।
- ভবনে রক্ষিত অগ্নি নির্বাপক যন্ত্রের সাহায্যে অগ্নি নির্বাপণে সহায়তা করুন।
- অগ্নিনির্বাপণ কর্মীর নির্দেশনা মেনে ভবন ত্যাগ করুন।

(ঘ) ভবন ত্যাগের নির্দেশনা পেলে

- নিকটতম সিঁড়ি দিয়ে দ্রুত ভবন ত্যাগ করুন।
- ভবনের বাইরে অবস্থিত নির্ধারিত সমাবেশস্থলে অগ্নিনির্বাপণ কর্মীকে আপনার উপস্থিতি অবহিত করুন।
- অগ্নিকাণ্ডের সময় লিফ্‌ট ব্যবহার করবেন না।
- অগ্নিকাণ্ডের সময় ভীতসন্ত্রস্ত হয়ে অহেতুক দৌড়াদৌড়ি বা চিৎকার করবেন না।
- ব্যক্তিগত জিনিস সংগ্রহের জন্য সময় নষ্ট করবেন না।
- সিঁড়ি ঘরের দরজা, লিফ্‌ট লবির দরজা বন্ধ রাখুন।
- প্রসাধন কক্ষ ব্যবহার করা থেকে বিরত থাকুন।
- গুজবে কান দেবেন না, গুজব ছড়াবেন না।

PART IV
Appendix B
Fire Protection Considerations For Venting In Industrial And Storage Buildings

B.1 Scope

B.1.1 This Appendix covers venting requirements in industrial buildings. Provisions contained herein shall be applicable to factory and storage facilities requiring large floor areas without dividing walls and enclosures.

B.1.2 This Appendix shall not apply to ventilation designed for personnel comfort, commercial cooking operation, regulating odor or humidity in toilet and bathing facilities, to regulate cooling equipment.

B.1.3 This Appendix shall apply to fire and smoke of two criteria: (a) Fire or smoke layer that does not enhance the burning rate and (b) Deflagration.

B.2 Venting of Fire And Smoke That Does Not Enhance the Burning Rate

B.2.1 Determination of precise venting requirements is difficult, as variables like rate of combustion, composition of the combustion product, shape, size and packaging of the combustible materials as well the size, height and disposition of the stacks of materials are involved with it.

B.2.2 Vent system designs shall be computed by calculating the vent area required to achieve a mass rate of flow through the vent that equals the mass rate of smoke production.

B.2.3 Venting devices are to be so designed and installed that they operate automatically at the earliest sign of fire or smoke.

B.2.4 The smoke and fire venting system shall be so designed and installed as to keep the temperature of the combustion product as low as possible, preferably below 150ºC.

B.2.5 To achieve full efficiency in vents total area of all vents must be more than the inlet area for cold air. Ideally the inlets should be as close to the ground as possible.

B.2.6 The area of unit vent shall not exceed $2d^2$, where d is the design depth of the smoke layer. For vents with length to width ratio more than two, the width shall not exceed the design depth of the smoke layer.

B.2.7 The center-to-center spacing of vents within a curtained area shall not exceed 2.8 H, where H is the ceiling height. For different shape of the roof the ceiling height can be calculated as per provision of NFPA 204.

B.2.8 The spacing of vents shall be such that the horizontal distance from any point on a wall or draft curtain to the center of the nearest vent, within a curtained area does not exceed 1.4 H.

B.2.9 The total vent area per curtained area shall be sized to meet the design objectives and the performance objectives relative to the design fire or smoke, determined in accordance with NFPA 204.

B.2.10 The design of venting for sprinkled building shall be based on performance analysis acceptable to the authority having jurisdiction, demonstrating that the established objectives are met.

B.2.11 Smoke and heat venting systems and mechanical exhaust systems shall be inspected and maintained in accordance with NFPA 204.

B.2.12 Venting systems are complement to fire extinguishing system. Where automatic sprinklers are installed as fire extinguishing system, the sprinklers shall operate before the vent system comes into operation.

B.2.13 In industrial buildings exterior wall windows alone shall not be accepted as satisfactory means of venting, but may be reckoned as additional means of venting when located close to the eaves and are provided with ordinary glass or movable section arranged for both manual and automatic operation.

B.2.14 Vents shall be automatic in operation unless where designed specifically for both manual and automatic operation.

B.2.15 Release mechanism of vent closure shall be simple in operation and shall not be dependent on electric power.

B.2.16 The automatic operation of vents can be achieved by actuation of fusible links or other heat or smoke detectors or by interlacing with the operation of sprinkler system or any other automatic fire extinguishing system covering the area. The vents can be so designed as to open by counterweights utilizing the force of gravity or spring loaded level following its release.

B.2.17 When vents and automatic sprinklers where installed together, sprinkler shall go into operation first before vents open, in order to avoid delay in sprinkler operation.

B.2.18 Materials used in hinges, hatches and other related parts in vents shall be noncorrosive in nature for long trouble free operation.

B.2.19 Vents shall be properly sited, at the highest point in each area to be covered.

B.2.20 If possible, vents shall be sited right on top of the probable risk area to be protected to ensure free and speedy removal of smoke and other combustion product.

B.2.21 Minimum vent opening shall not be less than 1250 mm in any direction.

B.2.22 Vent spacing shall be designed considering the fact that higher number of smaller vents is better than smaller number of large vents.

B.3 Deflagration Venting

B.3.1 Deflagration is the propagation of a fire or smoke at a velocity less than the sound wave. When this velocity of combustion increased beyond sound velocity then the combustion is said to be detonated and explosion occurred with the rupture of an enclosure or a container due to the increase of internal pressure from a deflagration.

B.3.2 The design of deflagration vents and vents closures necessitates consideration of many variables, only some of which have been investigated in depth. No Venting recommendations are currently available for fast-burning gases with fundamental burning velocities greater than 1.3 times that of propane, such as hydrogen. Recommendations are unavailable and no venting data have been generated that addresses condition that fast-burning gas deflagrations. The user is cautioned that fast-burning gas deflagrations can readily undergo transition to detonation.

B.3.3 Deflagration venting is provided for enclosures to minimize structural damage to the enclosure itself and to reduce the probability of damage to the other structures.

B.3.4 Venting shall be sufficient to prevent the maximum pressure that develops within the enclosure from exceeding enclosure strength.

B.3.5 The vent area shall be distributed as symmetrically and as evenly as possible.

B.3.6 The need for deflagration vents can be eliminated by the application of explosion prevention techniques described in NFPA 69.

B.3.7 The vent closure shall be designed to function as rapid as is practical. The mass of the closure shall be as low as possible to reduce the effects of inertia. The total mass of the moveable part of the vent closure assembly shall not be exceeded 12.2 kg/m^2.

B.3.8 When an enclosure is subdivided into compartments by walls, partitions, floors, or ceilings, then each compartment that contains a deflagration hazard should be provided with its own vent closure(s).

B.3.9 It is possible to isolate hazardous operations and equipment outside of buildings with a pressure resisting wall which will reduce risk of structural damage. Such operations and equipment may be housed in a single storey building having appropriate venting facilities and a device to absorb explosion shock from blowing through the duct back to the building.

B.3.10 Sometimes it may not be possible to house hazardous operations and equipment outside of the building, in which case the separation from other parts and equipment shall be achieved by pressure resisting walls and such units shall be ventilated outdoors. If suitable vents are integrated, external walls may be of heavy construction or of heavy panel which may be blown off easily.

B.3.11 Unobstructed vent opening is the most effective pressure release vent structures.

B.3.12 Explosion relief vents may be provided with open or unobstructed vents, louvers, roof vents, hanger type doors, building doors, windows, roof or wall panels or marble/fixed sash. Any or more than one of these may be adopted depending on individual situations and requirements.

B.3.13 Roof vents covered with weather hoods shall be as light as possible and attached lightly, so that it is easily blown off as and when an explosion occurs.

B.3.14 Doors and windows used as explosion vents shall be so fixed as to open outward. Doors shall be fitted with friction, spring or magnetic, latches that function automatically at the slight increase in internal pressure.

B.3.15 Placed at the top or bottom, the hinged or projected movable sash shall be equipped with latch or friction device to prevent accidental opening due to wind action or intrusion. Such latches or locks shall be well maintained.

B.3.16 Venting shall be so planned as to prevent injury to inmates and damage to enclosure. In populated locations, substantial ducts or diverts shall be provided to channelize the blast towards a pre-determined direction.

B.3.17 If explosion are probable within the duct, they shall be equipped with diaphragm to rupture at predetermined locations. The duct system shall not be physically connected to more than one collector.

B.3.18 Skylight with moveable sash that opens outward or fixed sash having panes of glass or plastic that blow out readily under pressure from within can be used to supplement wall vents or windows, provided their resistance to opening or displacement may be kept as low as possible consistent with structural requirement of the building.

B.3.19 For equivalent explosion pressure release, larger closed vents will be required compared to open vents.

B.3.20 As far as possible hazardous areas shall be segregated be means of fire walls or party walls to prevent spread of fire.

PART IV

Appendix C

Selection and Sitting of Fire Detection System

C.1 General

This Appendix provides information for selection and sitting of equipment for fire detection in buildings.

C.2 Choice of Fire Detectors

Fire detectors may respond to any one manifestation of combustions such as heat generation, smoke and flames.

Smoke detectors are not naturally suitable in places where the production process produces smokes.

Application of flame detectors are restricted due to the fact that all combustions do not necessarily accompany flame and that clear line of sight is desirable as radiation from flames travel in straight lines for actuation of sensitive element.

No single detector is able to meet the need of all types of fires and all types of occupancies. As such, based on needs arising out of various situations and occupancies, judicious selection is extremely important for the reduction of fire hazards.

C.2.1 Heat Detectors

"Point" or "Spot" type detectors are actuated by heat at layer adjacent to it over a limited area. "Line" type detectors are sensitive to the effect produced by heated gas along any portion of the detector line. Both the types operate on two broad principles: one, the heat sensitive elements is actuated by temperature rising beyond a predetermined level; while the second system is actuated by predetermined rate of rise of temperature.

C.2.2 Flame Detectors

Flame detectors are sensitive to radiation emitted by flames. Since heat, smoke and flame are produced during a fire, detectors responding to all these are accepted as general purpose detectors.

Fixed temperature heat detectors are suitable for use where ambient temperatures are high and or may rise and fall rapidly over a short period.

C.2.3 Rate of Rise Heat Detectors

These are suitable for use where ambient temperatures are low and/or may rise over a wide range slowly. Abnormally sharp rise in temperature during a fire actuates this alarm. As such it cannot be used with confidence where ambient temperatures reaches in the neighborhood of 40^{o}C, but are best used where ambient temperatures are in the range of about 40^{o}C.

C.2.4 Smoke Detectors

Three types of smoke detectors are commonly used. First type is actuated by absorption or scattering of visible or near-visible light by combustion product and known as "optical detector". The second type is actuated by the production on ionization current within the detector and referred to as "ionization detector". The third type is sensitive to carbon monoxide or other products of combustion and is known as "chemically sensitive detector". In general, these should be used at places where ambient temperature varies between 0^{o} to 35^{o}C.

C.2.4.1 Optical smoke detectors

Invisible smoke from a clear burning shall not actuate such detectors. But they respond quickly where smoke is optically dense and as such suitable for use in dust free clean atmosphere. Over a period of time, due to dust and dirt, the sensitive surface of photo sensitive element and/or executor lamp of optical detectors may loss its efficiency and as such optical detectors should be cleaned and maintained regularly.

C.2.4.2 Ionization chamber smoke detectors

These responds quickly to invisible smoke of clear burning, but may not respond to fire producing dense smoke. These can be used in dust free, humidity controlled area. Smoke and other fumes, dust including slow accumulated and disturbed aerial dust, fiber, steam and condensation produced by normal processes and vehicle engines may cause false alarm. Warehouses exposed to fast air flows can also cause false alarm. Burning of polyvinyl chloride will not sensitize the detector in time and may provide late warning or no warning at all.

C.2.4.3 Chemically sensitive smoke detectors

Chemically coated sensitive elements react to carbon monoxide or other products of combustion present in smoke. Dust or moisture adversely affects the sensitive elements and are not very suitable for residential use.

C.3 Siting Of Detectors

Considering the prevailing weather condition of the occupancies and the problem of false alarm, the type of detectors and the area of coverage shall be decided. Area of coverage of detectors is dependent on many factors. The following aspects shall be taken into considerations in the design of detectors.

- Various forms of overhead heating
- Exhaust air from air cooling equipment blowing out into the room or factory area
- Deep beams
- Roofs and ceiling of unusual shape
- Building with ground areas above 10 m and up to 30 m in height
- Staircases
- Canteen and Restaurants
- Plant Rooms
- Ambulant air currents

PART V
Chapter 1
Scope and Definitions

1.1 Scope

This Part specifies the minimum requirements of materials to be complied with in buildings and works under the provisions of the Code.

For each of the building materials the applicable standard specifications and test methods are listed. All materials shall conform to these Standards.

The list of standards given in this Part of the Code would be augmented from time to time by amendments, revisions and additions of which the Authority shall take cognizance. The latest version of a specification shall, as far as practicable, be applied in order to fulfil the requirements of this Part.

In view of the limited number of Bangladesh Standards (BDS) for building materials available at the present time, a number of standards of other countries have been referenced in this Code as applicable standards. As more standards of BDS regarding building materials become available and adopted by amendment of this Code, they shall supplement and/or replace the relevant standards listed in this Part.

1.2 Terminology

This Section provides an alphabetical list of the terms used in and applicable to this Part of the Code. In case of any conflict or contradiction between a definition given in this Section and that in Part 1, the meaning provided in this Part shall govern for interpretation of the provisions of this Part.

ACTUAL DIMENSIONS	Measured dimensions of a designated item.
ADMIXTURE	Material other than water, aggregate, or hydraulic cement used as an ingredient of concrete and added to concrete before or during its mixing to modify its properties.
AGGREGATE	Granular material, such as sand, gravel, crushed stone, crushed brick and iron blast-furnace slag, when used with a cementing medium that forms hydraulic cement concrete or mortar.
AGGREGATE, LIGHT WEIGHT	Aggregate with a dry, loose weight of 11.25 kN/m^3 or less.

CONCRETE	A mixture of Portland cement or any other hydraulic cement, fine aggregate, coarse aggregate and water, with or without admixtures.
CONCRETE, PLAIN	Concrete that does not conform to the definition of reinforced concrete.
CONCRETE, PRECAST	Plain or reinforced concrete element cast separately before they are fixed in position.
CONCRETE, PRESTRESSED	Reinforced concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.
CONCRETE, REINFORCED	Concrete containing adequate reinforcement, prestressed or non-prestressed, and designed on the assumption that the two materials act together in resisting forces.
FIBRE BOARD	A fibre-felted, homogenous panel made from lignocellulosic fibres (usually wood or cane) and having a unit weight between 1.6 kN/m^3 and 5 kN/m^3.
HARD BOARD	A fibre-felted homogenous panel made of lignocellulosic fibres consolidated under heat and pressure in a hot press to a density of 4.9 kN/m^3 or above.
MASONRY UNIT	Brick, tile, stone, glass-block or concrete-block used in masonry constructions.
MASONRY UNIT, GROUTED HOLLOW	Form of grouted masonry construction in which certain designated cells of hollow units are continuously filled with grout.
MASONRY UNIT, HOLLOW	A masonry unit whose net cross-sectional area in every plane parallel to the bearing surface is less than 75 percent of the gross cross-sectional area in the same plane.
MASONRY UNIT, SOLID	A masonry unit whose net cross-sectional area in every plane parallel to the bearing surface is 75 percent or more of the gross cross-sectional area in the same plane.
NOMINAL DIMENSIONS	Nominal dimensions of masonry units are equal to their specified dimensions plus the thickness of the joint with which the unit is laid.

PARTICLE BOARD	A manufactured panel product consisting of particles of wood or combinations of wood particles and wood fibres cemented together with synthetic resins or other suitable bonding system by an appropriate bonding process.
PLYWOOD	A built-up panel of laminated veneers.
REINFORCED MASONRY	Form of masonry construction in which reinforcement acting in conjunction with the masonry is used to resist designed forces.
REINFORCEMENT	Reinforcing bars, plain or deformed, excluding prestressing tendons, bar and rod mats, welded smooth wire fabric and welded deformed wire fabric used in concrete.
REINFORCEMENT, DEFORMED	Deformed reinforcing bars, bar and rod mats, deformed wire, welded smooth wire fabric and welded deformed wire fabric.
REINFORCEMENT, PLAIN	Reinforcement that does not conform to definition of deformed reinforcement.
REINFORCEMENT, SPIRAL	Continuously wound reinforcement in the form of a cylindrical helix.
STIRRUP	Reinforcement used to resist shear and torsion stresses in structural member; typically bars, wires, or welded wire fabric (smooth or deformed) bent into L, U or rectangular shapes and located perpendicular to or at an angle to longitudinal reinforcement. (The term "Stirrup" is usually applied to lateral reinforcement in flexural members and the term "ties" to those in compression members).
STRUCTURAL GLUED LAMINATED TIMBER	Any member comprising an assembly of laminations of lumber in which the grain of all laminations is approximately parallel longitudinally in which the laminations are bonded with adhesives.
TENDON	Steel element such as wire, cable, bar, rod or strand, or a bundle of such elements, used to impart prestress to concrete.
TIE	A loop of reinforcing bar or wire enclosing longitudinal reinforcement.
YIELD STRENGTH	The stress at which plastic deformation takes place under constant or reduced load.

PART V

Chapter 2

Building Materials

2.1 General

Materials used for the construction of buildings shall conform to standard specifications listed in this Part of the Code. Any deviation from the type design or architectural detail from those specified in these standards may be accepted by the Building Official as long as the materials standards specified therein are conformed with.

2.1.1 New or Alternative Materials

The provisions of this Part are not intended to prevent the use of any new and alternative materials. Any such material may be approved provided it is shown to be satisfactory for the purpose intended and at least equivalent of that required in this Part in quality, strength, effectiveness, fire resistivity, durability, safety, maintenance and compatibility.

Approval in writing shall be obtained by the owner or his agent before any new, alternative or equivalent materials are used. The Building Official shall base such approval on the principle set forth above and shall require that specified tests be made as per Sec 2.1.4 or sufficient evidence or proof be submitted, at the expense of the owner or his agent, to substantiate any claim for the proposed material.

2.1.2 Used Materials

The provisions of this Part do not preclude the use of used or reclaimed materials provided such materials meet the applicable requirements as for new materials for their intended use.

2.1.3 Storage of Materials

All building materials shall be stored at the building site(s) in such a way as to prevent deterioration or the loss or impairment of their structural and other essential properties (Part 7 of this Code).

2.1.4 Methods of Test

Every test of material required in this Part, or by the Building Official, for the control of quality and for the fulfillment of design and specification requirements, shall be carried out in accordance with a standard method of test issued by the Bangladesh Standards and Testing Institution (BSTI). In the absence of Bangladesh Standards, the Building Official shall determine the test procedures. Laboratory tests shall be conducted by recognized laboratories acceptable to the Building Official.

If, in the opinion of the Building Official, there is insufficient evidence of compliance with any of the provisions of the Code or there is evidence that any material or construction does not conform to the requirements of this Code, the Building Official may require tests to be performed as proof of compliance. The cost of any such test shall be borne by the owner.

The manufacturer or supplier shall satisfy himself that the materials conform to the relevant standards and if requested shall furnish a certificate or guarantee to this effect.

2.2 Masonry

2.2.1 Aggregates

Aggregates for masonry shall conform to the standards listed as follows: ASTM C144 Aggregates for Masonry Mortar; ASTM C404 Aggregates for Masonry Grout; ASTM C331 Lightweight Aggregates for Concrete Masonry Units (the applicable Standards for masonry are listed at the end of this Section).

2.2.2 Cement

Cement for masonry shall conform to the standards listed as follows: BDS EN 197-1: 2003 Cement Part-1 Composition, specifications and conformity criteria for common cements; or ASTM C150/C150M Portland Cement; ASTM C91 Masonry Cement; ASTM C595/C595M Blended Hydraulic Cements.

2.2.3 Lime

Limes for masonry shall conform to the standards listed as follows: ASTM C5, Quicklime for Structural Purposes; ASTM C207, Hydrated Lime for Masonry Purposes.

2.2.4 Masonry Units

(a) Clay: Masonry units of clay (or shale) shall conform to the standards listed as follows: BDS 208: 2009, Common building clay bricks; BDS 1249:1989, Acid resistant bricks; BDS 1250: 1990, Burnt clay facing bricks; BDS 1263: 1990, Burnt clay hollow bricks for walls and partitions; BDS 1264 : 1990, Glossary of terms relating to structural clay products; BDS 1432: 1993, Burnt clay perforated building bricks; BDS 1803: 2008, Specification for hollow clay bricks and blocks; ASTM C34 Structural Clay Load-Bearing Wall Tile; ASTM C212 Structural Clay Facing Tile; ASTM C56 Structural Clay Non-Load-Bearing Tile; and IS 7556 Burnt clay jallies.

(b) Concrete: Concrete masonry units shall conform to the standards listed as follows :

BDS EN 771-3	Specification for masonry units-Part: 3 Aggregate concrete masonry units (dense and lightweight aggregates).
BDS EN 772-1	Methods of test for masonry units-Part 1: Determination of compressive strength.
BDS EN 772-2	Methods of test for masonry units-Part 2: Determination of percentage area of voids in masonry units (by paper indentation).
BDS EN 772-6	Methods of test for masonry units-Part 6: Determination of bending tensile strength of aggregate concrete masonry units.
BDS EN 772-11	Methods of test for masonry units-Part 11: Determination of water absorption of aggregate concrete, autoclaved aerated concrete, manufactured stone and natural stone masonry units due to capillary action and the initial rate of water absorption of clay masonry units.
BDS EN 772-13	Methods of test for masonry units-Part 13: Determination of net and gross dry density of masonry units (except for natural stone).
BDS EN 772-14	Methods of test for masonry units-Part 14: Determination of moisture movement of aggregate concrete and manufactured stone masonry units.
BDS EN 772-16	Methods of test for masonry units-Part 16: Determination of dimensions.
BDS EN 772-20	Methods of test for masonry units-Part 20: Determination of flatness of faces of masonry units.
BDS EN 1052-3	Methods of test for masonry-Part 3: Determination of initial shear strength BDS EN 1745: 2009 Masonry and masonry products-Methods for determining design thermal values.
ASTM C55	Concrete Building Bricks.
ASTM C90	Specification for Load-Bearing Concrete Masonry Units.
ASTM C129	Non-Load Bearing Units.

(c) Others

Calcium Silicate	Calcium Silicate Face Brick (Sand-Lime Brick) shall conform to ASTM C73 Standard Specification.
Glazed Masonry Units	Glazed Masonry building units shall conform to the standards listed as follows: ASTM C126, Ceramic-Glazed Structural Clay Facing Tile, Facing Brick, and Solid Masonry Units; or ASTM C744 Prefaced Concrete and Calcium Silicate Masonry Units.
Glass Block	Glass block may be solid or hollow and contain inserts; all mortar contact surfaces shall be treated to ensure adhesion between mortar and glass.
Un-burnt Clay Masonry Units	Masonry of un-burnt clay units including cement stabilized and lime stabilized blocks shall not be used, in any building more than one storey in height.
Architectural Terra Cotta	All architectural terra cotta units shall be formed with a strong homogeneous body of hard-burnt weather-resistant clay which gives off a sharp metallic ring when struck. All units shall be formed to engage securely with and anchor to the structural frame or masonry wall.
Natural Stone	Natural stone for masonry shall be sound and free from loose friable inclusions. Natural stone shall have the strength and fire resistance required for the intended use.
Cast Stone	All cast stone shall be fabricated of concrete or other approved materials of required strength, durability and fire resistance for the intended use and shall be reinforced where necessary.
AAC Masonry	AAC (Autoclaved Aerated Concrete) masonry units shall conform to ASTM C1386 for the strength class specified.
Ceramic tile	Ceramic tile shall be as defined in, and shall conform to the requirements of ANSI A137.1.
Second Hand Units	Second hand masonry units shall not be used unless the units conform to the requirements for new units. The units shall be of whole, sound material and be free from cracks and other defects that would interfere with proper laying or use. All old mortar shall be cleaned from the units before reuse.

2.2.5 Mortar

Mortar shall consist of a mixture of cementitious material and aggregates to which sufficient water and approved additives, if any, have been added to achieve a workable, plastic consistency. Cementitious materials for mortar shall be one or more of the following: lime, masonry cement, Portland cement and mortar cement. Mortar for masonry construction other than the installation of ceramic tile shall conform to the requirements of BDS 1303: 1990 Chemical resistant mortars; BDS 1304:1990 Methods of test for chemical resistant mortars; ASTM C270, Mortar for Unit Masonry.

2.2.6 Grout

Grout shall consist of a mixture of cementitious materials and aggregates to which water has been added such that the mixture will flow without segregation of the constituents. Cementitious materials for grout shall be one or both of the following: Lime and Portland cement. Grout shall have a minimum compressive strength of 13 MPa. Grout used in reinforced and unreinforced masonry construction shall conform to the requirements of ASTM C476 Grout for Masonry.

2.2.7 Mortar for Ceramic Wall and Floor Tile

Portland cement mortars for installing ceramic wall and floor tile shall comply with ANSI A 108.1-2005 listed in Sec 2.2.11 and be of the composition specified in Table 5.2.1.

2.2.7.1 Dry-set leveling cement mortars

Premixed prepared leveling cement mortars, which require only the addition of water and are used in the installation of ceramic tile, shall comply with ANSI A118.1. The shear bond strength for tile set in such mortar shall be as required in accordance with ANSI A118.1. Tile set in dry-set Portland cement mortar shall be installed in accordance with ANSI A108.5.

2.2.7.2 Latex-modified leveling cement mortar

Latex-modified leveling cement thin-set mortars in which latex is added to dry-set mortar as a replacement for all or Part of the gauging water that are used for the installation of ceramic tile shall comply with ANSI A118.4. Tile set in latex-modified leveling cement shall be installed in accordance with ANSI A108.5.

2.2.7.3 Epoxy mortar

Ceramic tile set and grouted with chemical-resistant epoxy shall comply with ANSI A118.3. Tile set and grouted with epoxy shall be installed in accordance with ANSI A108.6.

2.2.7.4 Furan mortar and grout

Chemical-resistant furan mortar and grout that are used to install ceramic tile shall comply with ANSI A118.5. Tile set and grouted with furan shall be installed in accordance with ANSI A108.8.

2.2.7.5 Modified epoxy-emulsion mortar and grout

Modified epoxy-emulsion mortar and grout that are used to install ceramic tile shall comply with ANSI A118.8. Tile set and grouted with modified epoxy-emulsion mortar and grout shall be installed in accordance with ANSI A108.9.

2.2.7.6 Organic adhesives

Water-resistant organic adhesives used for the installation of ceramic tile shall comply with ANSI A136.1. The shear bond strength after water immersion shall not be less than 275 kPa (40 psi) for Type I adhesive and not less than 138 kPa (20 psi) for Type II adhesive when tested in accordance with ANSI A136.1. Tile set in organic adhesives shall be installed in accordance with ANSI A108.4.

2.2.7.7 Portland cement grouts

Portland cement grouts used for the installation of ceramic tile shall comply with ANSI A118.6. Portland cement grouts for tile work shall be installed in accordance with ANSI A108.10.

2.2.7.8 Mortar for Autoclaved Aerated Concrete (AAC) masonry

Thin-bed mortar for AAC masonry shall comply with Article 2.1 C.1 of TMS 602/ACI 530.1/ASCE 6. Mortar used for the leveling courses of AAC masonry shall comply with Article 2.1 C.2 of TMS 602/ACI 530.1/ASCE 6.

2.2.8 Metal Ties and Anchors

Metal ties and anchors shall conform to the standards listed as follows: ASTM A82/A82M, Wire Anchor and Ties; and ASTM A1008/A1008M, Sheet Metal Anchors and Ties.

Table 5.2.1: Ceramic Tile Mortar Compositions

Walls	Scratch coat	1 cement, 0.20 hydrated lime*, 4 dry or 5 damp sand
	Setting bed and leveling coat	1 cement, 0.50 hydrated lime, 5 damp sand to 1 cement, 1 hydrated lime; 7 damp sand
Floors	Setting bed	1 cement; 0.10 hydrated lime; 5 dry or 6 damp sand; or 1 cement; 5 dry or 6 damp sand
Ceilings	Scratch coat and sand bed	1 cement; 0.50 hydrated lime; 2.50 dry sand or 3 damp sand

* Lime may be excluded from the mortar if trial mixes indicate that the desired workability and performance are achieved without lime.

2.2.9 Reinforcement

Reinforcement in masonry shall conform to the standards listed as follows: ASTM A82/A82M, Cold Drawn Steel Wire for Concrete Reinforcement; ASTM A615/A615M, Deformed and Plain Billet Steel Bars; ASTM A996/A996M, Rail-Steel Deformed and Plain Bars; ASTM A996/A996M, Axle-Steel Deformed and Plain Bars; ASTM A706/A706M, Low-Alloy Steel Deformed Bars; ASTM A767/A767M, Zinc-Coated (Galvanized) Steel Bars; and ASTM A775/A775M, Epoxy-Coated Reinforcing Steel Bars.

2.2.10 Water

Water used in mortar or grout shall be clean and free of deleterious amounts of acid, alkalis or organic material or other harmful substances.

2.2.11 Applicable Standards for Masonry

The applicable standards for Masonry are listed below:

BDS EN 197-1	Cement Part-1 Composition, Specifications and Conformity Criteria for Common Cements.
BDS 208	Specification for Common Building Clay Bricks : Specifies the dimensions, quality and strength of common burnt clay bricks, methods of sampling, testing etc.

BDS 238	Fire Clay Refractory Bricks and Shapes for General Purposes: This Standard specifies the requirements for fireclay refractory bricks and shapes meant for general purpose; the products are classified in four grades according to the duty for which they are suitable.
BDS 1249	Acid Resistant Bricks: It specifies the requirements for acid-resistant bricks, dimensions, tolerances, test etc.
BDS 1250	Burnt Clay Facing Bricks: It specifies the dimensions, quality and strength of burnt clay facing bricks used in building and other structure, physical requirements etc.
BDS 1263	Burnt Clay Hollow Bricks for Walls and Partitions: It covers the dimensions, quality and strength for hollow bricks made from burnt clay and having perforations through and at right angle to the bearing surface tests.
BDS 1264	Glossary of Terms Relating to Structural Clay Products: It covers the definition of common terms applicable to structural clay products, used in building and civil engineering works.
BDS 1432	Burnt Clay Perforated Building Bricks: Specifies the requirements in regard to dimensions, perforations, quality, strength and also for quality of surface in case of special grade for facing bricks of perforated burnt clay building bricks for use in walls and partitions.
BDS 1433	Dimensions quantities in general construction work: Specifies the various dimensional values in SI units used in general construction work.
BDS 1803	Specification for hollow clay bricks and blocks.
BDS EN 1338	Concrete paving blocks-Requirements and test methods.
BDS EN 1339	Concrete paving flags-Requirements and test methods.
BDS EN 1340	Concrete kerb units-Requirements and test methods.
BDS EN 13369	Common rules for precast concrete products.
BDS EN 771-3	Specification for masonry units Part 3: Aggregate concrete masonry units (dense and lightweight aggregates).
BDS EN 772-1	Methods of test for masonry units Part 1: Determination of compressive strength.

BDS EN 772-2	Methods of test for masonry units Part 2: Determination of percentage area of voids in masonry units (by paper indentation).
BDS EN 772-6	Methods of test for masonry units Part 6: Determination of bending tensile strength of aggregate concrete masonry units.
BDS EN 772-11	Methods of test for masonry units Part 11: Determination of water absorption of aggregate concrete, autoclaved aerated concrete, manufactured stone and natural stone masonry units due to capillary action and the initial rate of water absorption of clay masonry units.
BDS EN 772-13	Methods of test for masonry units Part 13: Determination of net and gross dry density of masonry units (except for natural stone).
BDS EN 772-14	Methods of test for masonry units Part 14: Determination of moisture movement of aggregate concrete and manufactured stone masonry units.
BDS EN 772-16	Methods of test for masonry units Part 16: Determination of dimensions.
BDS EN 772-20	Methods of test for masonry units Part 20: Determination of flatness of faces of masonry units.
BDS EN 1052-3	Methods of test for masonry Part 3: Determination of initial shear strength.
BDS EN 1745	Masonry and masonry products: Methods for determining design thermal values.
ANSI A108.1A	Installation of Ceramic Tile in the Wet-Set Method, with Portland Cement Mortar.
ANSI A108.1B	Installation of Ceramic Tile, Quarry Tile on a Cured Portland Cement Mortar Setting Bed with Dry-set or Latex-Portland Mortar.
ANSI A108.1	Specifications for the Installation of Ceramic Tile with Portland Cement Mortar.
ASTM A82/ A82M	Specification for Steel Wire, Plain, for Concrete Reinforcement.

ASTM A1008/ A1008M	Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Bake Hardenable.
ASTM A615/A615M	Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement.
ASTM A996/A996M	Standard Specification for Rail-Steel and Axle-Steel Deformed Bars for Concrete Reinforcement.
ASTM A706/A706M	Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement.
ASTM A183	Standard Specification for Carbon Steel Track Bolts and Nuts.
ASTM A775/A775M	Standard Specification for Epoxy-Coated Steel Reinforcing Bars.
ASTM C5	Standard Specification for Quicklime for Structural Purposes.
ASTM C34	Standard Specification for Structural Clay Load-Bearing Wall Tile.
ASTM C55	Standard Specification for Concrete Building Brick.
ASTM C56	Standard Specification for Structural Clay Non load bearing Tile.
ASTM C73	Standard Specification for Calcium Silicate Brick (Sand-Lime Brick).
ASTM C90	Standard Specification for Load bearing Concrete Masonry Units.
ASTM C91	Standard Specification for Masonry Cement.
ASTM C126	Standard Specification for Ceramic Glazed Structural Clay Facing Tile, Facing Brick, and Solid Masonry Units.
ASTM C129	Standard Specification for Non-load bearing Concrete Masonry Units.
ASTM C144	Standard Specification for Aggregate for Masonry Mortar.
ASTM C90	Standard Specification for Load bearing Concrete Masonry Units.
ASTM C150/ C150M	Standard Specification for Portland Cement.

ASTM C207	Standard Specification for Hydrated Lime for Masonry Purposes.
ASTM C212	Standard Specification for Structural Clay Facing Tile.
ASTM C270	Standard Specification for Mortar for Unit Masonry.
ASTM C331	Standard Specification for Lightweight Aggregates for Concrete Masonry Units.
ASTM C404	Standard Specification for Aggregates for Masonry Grout.
ASTM C476	Standard Specification for Grout for Masonry.
ASTM C595/C595M	Standard Specification for Blended Hydraulic Cements.
ASTM C744	Standard Specification for Prefaced Concrete and Calcium Silicate Masonry Units.

2.3 Cement and Concrete

2.3.1 General

Materials used to produce concrete, and admixtures used for concrete shall comply with the requirements of this Section and those of Chapter 5 Part 6 of this Code.

2.3.2 Aggregates

Concrete aggregates shall conform to the following standards:

BDS 243: 1963, Coarse and Fine Aggregates from Natural Sources for Concrete; ASTM C33/C33M Concrete Aggregates; ASTM C330/C330M Lightweight Aggregates for Structural Concrete; ASTM C637 Aggregates for Radiation-Shielding Concrete; ASTM C332 Lightweight Aggregate for Insulating Concrete; IS: 9142 Artificial lightweight aggregates for concrete masonry units.

2.3.2.1 Special tests

Aggregates failing to meet the specifications listed in Sec 2.4.2 shall not be used unless it is shown by special test or actual service experience to produce concrete of adequate strength and durability and approved by the Building Official.

2.3.2.2 Nominal size

Nominal maximum size of coarse aggregate shall not be larger than:

(a) One-fifth of the narrowest dimension between sides of forms; or

(b) One-third the depth of slabs; or

(c) Three fourths the minimum clear spacing between individual reinforcing bars or wires, bundles of bars, or pre-stressing tendons or ducts.

Exception:

The above limitations regarding size of coarse aggregate may be waived if, in the judgment of the Engineer, workability and methods of consolidation are such that concrete can be placed without honeycomb or voids.

2.3.3 Cement

Cement shall conform to the following standards: BDS EN 197-1:2003 Cement Part-1 Composition, specifications and conformity criteria for common cements, BDS 612 Sulphate resisting Portland cement-type A, ASTM C150/C150M Standard Specification for Portland Cement, BDS 232 Portland cement, ASTM C595/C595M Blended Hydraulic Cements, and to other such cements listed in ACI 318.

2.3.4 Water

Water used in mixing concrete shall be clean and free from injurious amounts of oils, alkalies salts, organic materials or other substances that may be deleterious to concrete or reinforcement. Water shall conform to the following standards: BDS ISO 12439:2011 Mixing water for concrete.

2.3.4.1 Chloride ions

Mixing water for pre-stressed concrete or for concrete that will contain aluminium embedment, including the portion of mixing water contributed in the form of free moisture on aggregates shall not contain deleterious amounts of chloride ion. The maximum water-soluble chloride ion concentration in concrete shall not exceed the limitations specified in Sec 5.5.3 Part 6.

2.3.4.2 Potability

Nonpotable water shall not be used in concrete unless the following are satisfied:

(a) Selection of concrete proportions shall be based on concrete mixes using water from such source.

(b) Mortar test cubes made with non-potable mixing water shall have 7 days and 28 days strengths equal to at least 90 percent of strengths of similar specimens made with potable water.

2.3.5 Admixtures

Admixtures to be used in concrete shall be subject to prior approval by the Building Official and shall comply with Sections 2.4.5.1 to 2.4.5.5.Admixtures shall conform following standards:

BDS EN 934-1	Admixtures for Concrete, Mortar and Grout-Part 1: Common Requirements.
BDS EN 934-2	Admixtures for Concrete, Mortar and Grout-Part 2: Concrete Admixtures Definitions, Requirements, Conformity, Marking and Labelling.

2.3.5.1 Chloride

Calcium chloride or admixtures containing chloride from admixture ingredients shall not be used in prestressed concrete, concrete containing embedded aluminium in concrete cast against permanent galvanized metal forms, or in concrete exposed to severe or very severe sulphate-containing solutions (Sec 5.5.2.1 Part 6).

2.3.5.2 Standards

Air-entraining admixtures shall conform to ASTM C260 Standard Specification for Air-entraining Admixtures for Concrete. Water-reducing admixtures, retarding admixtures, accelerating admixtures, water-reducing and retarding admixtures, and water-reducing and accelerating admixtures shall conform to ASTM C494/C494M Chemical Admixtures for Concrete, or ASTM C1017/C1017M Chemical Admixtures for Use in Producing Flowing Concrete.

2.3.5.3 Pozzolanas

Fly ash (Pulverized Fuel Ash) or other Pozzolanas used as admixtures shall conform to ASTM C618 Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolanas for Use in Concrete.

2.3.5.4 Blast furnace slag

Ground granulated blast-furnace slag used as an admixture shall conform to ASTM C989 Standard Specification for Slag Cement for Use in Concrete and Mortars.

2.3.5.5 Pigment for coloured concrete

Pigment for integrally coloured concrete shall conform to ASTM C979 Standard Specification for Pigments for Integrally Colored Concrete.

2.3.6 Metal Reinforcement

Reinforcement and welding of reinforcement to be placed in concrete shall conform to the requirements of this Section.

(a) Deformed Reinforcement: Deformed reinforcing bars shall conform to the following Standards; BDS ISO 6935-2:2010, Steel for the reinforcement of concrete-Part-2: Ribbed bars; Reinforcement conforming to the ASTM, Standards: A615/A615M Deformed and Plain Billet-Steel Bars; A616M, Rail-Steel Deformed and Plain Bars; A617M Axle-Steel Deformed and Plain Bars; A706M Low-Alloy Steel Deformed Bars; A767M Zinc Coated (Galvanized) Steel Bars; and A775M Epoxy-Coated Reinforcing Steel.

Deformed reinforcing bars with a specified yield strength $f_y$exceeding 410 MPa may be used, provided f_y shall be the stress corresponding to a strain of 0.35 percent and the bars otherwise conform to ASTM standards noted above. Fabricated deformed steel bar mats conforming to ASTM A184/A184M and deformed steel wire complying with ASTM A496/A496M may be used. Deformed wire for concrete reinforcement shall not be smaller than size D4 (nominal diameter: 5.72 mm), and for wire with a specified yield strength, f_y exceeding 410 MPa, f_y shall be the stress corresponding to a strain of 0.35 percent.

Welded deformed steel wire fabric conforming to ASTM A497/A497M may be used; for a wire with specified yield strength f_y exceeding 410 MPa, f_y shall be the stress corresponding to a strain of 0.35 percent. Welded intersections shall not be spaced farther apart than 400 mm in direction of calculated stress, except for wire fabric used as stirrups.

(b) Plain Reinforcement: Plain reinforcement shall conform to the following BDS and ASTM Standards. BDS ISO 6935-1:2010; ASTM A615/A615M; ASTM A996/A996M and ASTM A996/A996M. Steel welded wire, fabric plain reinforcement conforming to ASTM A185/A185M may be used, except that for wire with specified yield strength f_y exceeding 410 MPa, f_y shall be the stress corresponding to a strain of 0.35 percent. Welded intersections shall not be spaced farther apart than 300 mm in direction of calculated stress, except for wire fabric used as stirrups.

Smooth steel wire conforming to ASTM A182/A182M may be used in concrete; except that for a wire with specified yield strength f_y exceeding 410 MPa, f_y shall be the stress corresponding to a strain of 0.35 percent.

(c) Cold-worked Steel Reinforcement: Cold-worked steel high strength bars shall conform to IS 1786 or BS 4461: 1978.

(d) Pre-stressing Tendons: Wire, strands and bars for tendons in pre-stressed concrete shall conform to BDS: 240 Plain cold drawn steel wire; ASTM A416/A416M Steel Strand Uncoated Seven-Wire Stress Relieved; ASTM A421/A421M: Uncoated Stress Relieved Steel Wire; and ASTM A722/A722M: Uncoated High-Strength Steel Bar.

Wires, strands and bars not specifically listed in the above standards may be used, provided they conform to minimum requirements of these specifications and do not have properties that make them less satisfactory than those listed.

(e) Structural Steel, Steel Pipe or Tubing: Structural steel used with reinforcing bars in composite compression members meeting the requirements of the Code shall conform to ASTM A36/A36M Structural Steel; ASTM A242/A242M High Strength Low-Alloy Structural Steel; ASTM A572/A572M High-Strength Low-Alloy Columbium-Vanadium Steel; and ASTM A588/A588M High-Strength Low-Alloy Structural Steel.

Steel pipe or tubing for composite compression members composed of a steel-encased concrete core meeting the requirements of this Code shall conform to ASTM A53/A53M Pipe, Steel, Black and Hot Dipped Zinc Coated Welded and Seamless; ASTM A500/A500M Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes; and ASTM A501 Hot-Formed Welded and Seamless Carbon Steel Structural Tubing.

2.3.7 Applicable Standards

Materials used in concrete shall comply with the applicable standards listed below.

BDS 279	Specification for Abrasion of Coarse Aggregates by Use of Los Angeles Machine (under revision).
BDS 281	Specification for Organic Impurities in Sands for Concrete (under revision).
BDS 921	Specification for Standard Sand for Testing of Cement.
BDS 240	Specification for Plain Cold Drawn Steel Wire for Pre-stressed Concrete.
BDS 243	Specification for Coarse and Fine Aggregates from Natural Sources for Concrete.
BDS ISO 1920-8	Testing of Concrete-Part 8: Determination of Drying Shrinkage of Concrete for Samples Prepared in the Field or in the Laboratory.
BDS ISO 1920-9	Testing of Concrete-Part 9: Determination of Creep of Concrete Cylinders in Compression.
BDS ISO 1920-10	Testing of Concrete-Part 10: Determination of Static Modulus of Elasticity in Compression.
BDS ISO 22965-1	Concrete-Part 1: Methods of Specifying and Guidance for the Specifier.

BDS ISO 22965-2	Concrete-Part 2: Specification of Constituent Materials, Production of Concrete and Compliance of Concrete.
ASTM C31/C31M	Standard Practice for Making and Curing Concrete Test Specimens in the Field.
ASTM C39/C39M	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.
ASTM C42/C42M	Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.
ASTM C78	Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)
ASTM C94/C94M	Standard Specification for Ready-Mixed Concrete.
ASTM C172	Standard Practice for Sampling Freshly Mixed Concrete.
ASTM C192/C192M	Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory.
ASTM C317/C317M	Standard Specification for Gypsum Concrete.
ASTM C496/C496M	Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens.
ASTM C617	Standard Practice for Capping Cylindrical Concrete Specimens.
ASTM C685/C685M	Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing.
ASTM C989	Standard Specification for Slag Cement for Use in Concrete and Mortars.

2.3.8 Concrete Pipe and Precast Sections

Concrete pipes and precast sections shall conform to the Standards listed below:

BDS 1626	Concrete pipes (with and without) reinforcement.
ASTM C14M	Standard Specification for Non-reinforced Concrete Sewer, Storm Drain, and Culvert Pipe (Metric).
ASTM C76M	Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe (Metric).

ASTM C361M	Standard Specification for Reinforced Concrete Low-Head Pressure Pipe (Metric).
ASTM C444M	Standard Specification for Perforated Concrete Pipe (Metric).
ASTM C478M	Standard Specification for Precast Reinforced Concrete Manhole Sections (Metric).
ASTM C507M	Standard Specification for Reinforced Concrete Elliptical Culvert, Storm Drain, and Sewer Pipe (Metric).
ASTM C654M	Standard Specification for Porous Concrete Pipe (Metric).
ASTM C655M	Standard Specification for Reinforced Concrete D-Load Culvert, Storm Drain, and Sewer Pipe (Metric).
ASTM C1433M	Standard Specification for Precast Reinforced Concrete Monolithic Box Sections for Culverts, Storm Drains, and Sewers (Metric).
ASTM C858	Standard Specification for Underground Precast Concrete Utility Structures.
ASTM C891	Standard Practice for Installation of Underground Precast Concrete Utility Structures.
ASTM C913	Standard Specification for Precast Concrete Water and Wastewater Structures.
ASTM C924M	Standard Practice for Testing Concrete Pipe Sewer Lines by Low-Pressure Air Test Method (Metric).
IS 458	Specification for precast concrete pipes with and without reinforcement.
IS 784	Specification for pre-stressed concrete pipes.
IS 1916	Specification for steel cylinder pipe with concrete lining and coating.
IS 3597	Methods of test for concrete pipes.
IS 4350	Specification for concrete porous pipes for under drainage.
IS 7319	Specification for perforated concrete pipes.
IS 7322	Specification for specials for steel cylinder reinforced concrete pipes.

2.4 Pre-Stressed Concrete

2.4.1 Concrete for Pre-stressed Concrete

Cement and concrete required for pre-stressed concrete are elaborately described in Sec 2.3 of this Part. BDS and other standards for concrete as a material are also contained in the same section.

2.4.2 Steel for Pre-stressed Concrete

Steel and tendons for pre-stressed concrete along with the BDS and other standard requirements are included in Sec 2.8 of this Part.

Steel material for pre-stressed concrete shall also conform following Standards.

BDS ISO 6934-1	Steel for the prestressing of concrete-Part 1: General requirements.
BDS ISO 6934-2	Steel for the prestressing of concrete-Part 2: Cold-drawn wire.
BDS ISO 6934-3	Steel for the prestressing of concrete-Part 3: Quenched and tempered wire.
BDS ISO 6934-4	Steel for the prestressing of concrete-Part 4: Strand.
BDS ISO 6934- 5	Steel for the Prestressing of concrete-Part 5: Hot-rolled steel bars with or without subsequent processing.
BDS ISO 6935 (Part-1)	Steel for the reinforcement of concrete-Part-1: Plain bars.
BDS ISO 6935 (Part-2)	Steel for the reinforcement of concrete-Part-2: Ribbed bars.
BDS ISO 6935 (Part-3)	Steel for the reinforcement of concrete-Part-3: Welded fabric. Specifies technical requirements for factory made sheets or rolls welded fabric manufacture from steel wires or bars with diameters from 4 mm to 16 mm and designed for reinforcement in ordinary concrete structured and for non-prestressed reinforcement in prestressed concrete structures.
BDS ISO 10065	Steel bars reinforcement of concrete bend and re-bend tests.
BDS ISO 15835-1	Steel for the reinforcement of concrete-Reinforcement couplers for mechanical splices of bars-Part 1: Requirements.

BDS ISO 15835-2	Steel for the reinforcement of concrete-Reinforcement couplers for mechanical splices of bars-Part 2: Test methods.
BDS ISO 10144	Certification scheme for steel bars and wires for the reinforcement of concrete structures.
BDS ISO 15630-1	Steel for the reinforcement and Prestressing of concrete-Test methods-Part 1: Reinforcing bars, wire rod and wire.
BDS ISO 15630-2	Steel for the reinforcement and prestressing of concrete-Test methods-Part 2: Welded fabric.
BDS ISO 15630-3	Steel for the Reinforcement and prestressing of concrete-Test methods-Part 3: Prestressing steel.
BDS ISO 16020	Steel for the reinforcement and prestressing of concrete-Vocabulary.

2.5 Building Limes

2.5.1 Types of Lime

According to the degree of calcinations, slaking and setting actions and depending upon the nature and amount of foreign matters associated with, the limes are classified as: (i) High calcium, fat, rich, common or pure lime; (ii) Lean, meager or poor lime; and (iii) Hydraulic or water lime

2.5.2 Properties of Lime

A good lime should slake readily in water, dissolve in soft water, free from fuel ashes and unburnt particles and have good setting power under water.

Building limes shall comply with the following ASTM standard specifications: ASTM C206 Finishing Hydrated Lime; ASTM C207 Hydrated Lime for Masonry Purposes; ASTM C141/C141M Hydraulic Hydrated Lime for Structural Purposes; ASTM C977 Quicklime and Hydrated Lime for Soil Stabilization; and ASTM C5 Quicklime for Structural Purposes.

The following Indian Standards may also be accepted for lime concrete and testing of building limes:

IS712	Specification for building limes.
IS1624	Method of field testing of building lime.
IS 2686	Specification for cinder aggregates for use in lime concrete.

IS 3068	Specification for broken brick (burnt clay) coarse aggregates for use in lime concrete.
IS 3115	Specification for lime-based blocks.
IS 3182	Specification for broken brick (burnt clay) fine aggregates for use in lime mortar.
IS 4098	Specification for lime-pozzolana mixture.
IS 4139	Specification for sand-lime bricks.
IS 6932 (Parts I to XI)	Method of tests for building limes.
IS 10360	Specification for lime-pozzolana concrete blocks for paving.
IS 10772	Specification for quick setting lime pozzolana mixture.
IS12894	Specification for pulverized fuel ash lime bricks.

2.6 Gypsum Based Materials and Plaster

2.6.1 Gypsum Board

Gypsum wallboard, gypsum sheathing, gypsum base for gypsum veneer plaster, exterior gypsum soffit board, pre-decorated gypsum board or water resistant gypsum backing board complying with the standards listed below.

2.6.2 Gypsum Plaster

A mixture of calcined gypsum or calcined gypsum and lime and aggregate and other approved materials as specified in this Code.

2.6.3 Gypsum Veneer Plaster

Gypsum plaster applied to an approved base in one or more coats normally not exceeding 1/4 inch (6.4 mm) in total thickness.

2.6.4 Cement Plaster

A mixture of Portland or blended cement, Portland cement or blended cement and hydrated lime, masonry cement or plastic cement and aggregate and other approved materials as specified in this Code.

Gypsum building materials shall conform to the Standards listed below.

ASTM C22/C22M	Standard Specification for Gypsum.
ASTM C28/C28M	Standard Specification for Gypsum Plasters.
ASTM C35	Standard Specification for Inorganic Aggregates for Use in Gypsum Plaster.
ASTM C59/C59M	Standard Specification for Gypsum Casting Plaster and Gypsum Molding Plaster.
ASTM C317/ C317M	Standard Specification for Gypsum Concrete.
ASTM C471M	Standard Test Methods for Chemical Analysis of Gypsum and Gypsum Products.
ASTM C472	Standard Test Methods for Physical Testing of Gypsum, Gypsum Plasters and Gypsum Concrete.
ASTM C473	Standard Test Methods for Physical Testing of Gypsum Panel Products.
ASTM C474	Standard Test Methods for Joint Treatment Materials for Gypsum Board Construction.
ASTM C587	Standard Specification for Gypsum Veneer Plaster.
ASTM C1396/C1396M	Standard Specification for Gypsum Board.
IS 2849-1983	Specification for non-load bearing gypsum partition blocks (solid and hollow types).

2.7 Flooring Materials

2.7.1 General

Flooring materials are generally of two types; precast systems like tiles, bricks and cast in-situ.

2.7.2 Concrete/Terrazzo Tiles

Concrete/Terrazzo tiles shall have good abrasion and impact resistance properties. Factors such as the type of cement and the type and grading of aggregate used, influence the resistance of such tiles to chemicals including cleaning agents. Terrazzo tiles shall have a wear layer after grinding at least 6 mm composed of graded marble chipping in white, tinted or grey Portland cement on a layer of fine concrete. They may be ground after manufacture to expose the marble aggregate and subsequently grouted. Slip resisting grits may be incorporated. These tiles shall conform to BDS EN 13748-1:2008 Terrazzo tiles-Part 1: Terrazzo tiles for internal use; BDS EN 13748-2:2008 Terrazzo tiles-Part 2: Terrazzo tiles for external use; BDS 1262: 1990 Clay flooring tiles; BDS 1248: 1989 Ceramic unglazed vitreous acid resistant tiles or IS: 1237, Specification for cement concrete flooring tile.

2.7.3 Asphalt Tiles/Flooring

Asphalt tiles/floorings are suitable for industrial flooring in areas where they will not be exposed to solvents, grease, oil, corrosive chemicals and excessive heat. Bitumen mastic for flooring shall conform to IS: 1195; IS: 8374 Bitumen Mastic, Anti-static and Electrically Conducting Grade and IS: 9510 Bitumen Mastic Acid Resisting Grade.

2.7.4 Mosaic Tiles

Mosaic tiles of a variety of shapes and sizes may be used. Thickness of the wear layer is dependent on the sizes of marble chips but shall not be less than 6 mm thick. The tiles shall be wet cured for sufficient time before laying so that their surfaces are not damaged during grinding and polishing.

2.7.5 Clay Tile

Clay floor tiles shall have sufficient strength and abrasion resistant characteristics to withstand the impact and abrasion they are likely to be subject to. When glazed earthenware tiles are used in flooring they shall conform to IS: 777 Glazed Earthenware Tiles.

2.7.6 Vinyl Tiles

The vinyl tiles shall consist of a thoroughly blended composition of thermoplastic binder, asbestos fibre, fillers and pigments. The thermoplastic binder shall consist substantially of either or both of the following:

(a) Vinyl chloride polymer

(b) Vinyl chloride copolymers.

The polymeric material shall be compounded with suitable plasticizers and stabilizers. The tiles may be plain, patterned or mottled. The thickness shall not be less than 1.5 mm.

2.7.7 Rubber Tiles

These tiles are composed of natural, synthetic or reclaimed rubber, or a combination of these, with reinforcing fibres, pigments, and fillers, vulcanized and molded under pressure. The tiles shall have excellent resilience and resistance to indentation, and good resistance to grease, alkali and abrasion. The thickness shall not be less than 2 mm.

2.7.8 Cast In-situ Floor Coverings

(a) Terrazzo: Terrazzo is a marble mosaic with Portland cement matrix and is generally composed of two parts marble chips to one part Portland cement. Color pigments may be added. The thickness of terrazzo topping may vary from 13 mm to 19 mm and may be applied to green concrete of the floor or bonded with neat Portland cement, or over a sand cushion placed on the concrete floor.

(b) Concrete: A concrete topping may be applied to a concrete structural slab before or after the base slab has hardened. Integral toppings may generally be 25 mm to 40 mm thick; independent toppings about 25 mm to 50 mm thick. Aggregate sizes shall not exceed 6 mm.

2.7.9 Other Flooring Materials

Other flooring materials i.e. bricks, natural stone, etc. showing satisfactory performance in similar situations may be allowed. Plastic flooring tile and ceramic unglazed vitreous acid resistant tiles, if used, shall conform to IS: 3464 and IS: 4357 respectively.

Flooring compositions complying with IS: 657, Materials for use in the manufacture of magnesium oxychloride flooring composition; and IS: 9197, Epoxy resin composition for floor topping may be allowed. Linoleum sheets and tiles shall conform to IS: 653.

Flooring materials shall also conform to the standards listed below.

BDS 1248	Ceramic unglazed vitreous acid resistant tiles seat covers the requirements for ceramic unglazed vitreous acid resistant tiles used in lying of floors & lining of tanks subjected to corrosive conditions. Manufacture, Finish, Tests etc.
BDS 1262	Clay flooring tiles. Specifies the requirements for dimensions, quality & strength for clay flooring tiles & different types of tests.

BDS ISO 10545 - 1	Ceramic tiles, Sampling and basis for acceptance. Specifies rules for batching, sampling, inspection and acceptance/ rejection of ceramic tiles.
BDS ISO 10545 - 2	Ceramic tiles, Determination of dimensions and surface quality. Specifies methods for determining the dimensional characteristics (length, width, thickness, straightness of sides, rectangularity, and surface flatness) and the surface of ceramic tiles.
BDS ISO 10545 - 3	Ceramic tiles, Determination of water absorption, apparent porosity, apparent relative density and bulk density. Specifies methods for determining water absorption, apparent porosity, apparent relative density and bulk density of ceramic tiles.
BDS ISO 10545 - 4	Ceramic tiles, Determination of modulus of rupture and breaking strength Defines a test method for determining the modulus of rupture and breaking strength of all ceramic tiles.
BDS ISO 10545 - 5	Ceramic tiles, Determination of impact resistance by measurement of coefficient of restitution Specifies methods for determining the impact resistance of ceramic tiles by measuring the coefficient of restitution.
BDS ISO 10545 - 6	Ceramic tiles, Determination of resistance to deep abrasion for unglazed tiles.
BDS ISO 10545 - 7	Ceramic tiles, Determination of resistance to surface abrasion for glazed tiles. Specifies a method for determining the resistance to surface abrasion of all glazed ceramic tiles used for floor covering.
BDS ISO 10545 - 8	Ceramic tiles, Determination of linear thermal expansion Defines a test method for determining the coefficient of linear thermal expansion of ceramic tiles.
BDS ISO 10545 - 9	Ceramic tiles, Determination of resistance to thermal shock. Defines a test method for determining the resistance to thermal shock of all ceramic tiles under normal conditions of use.

BDS ISO 10545 - 10	Ceramic tiles, Determination of moisture expansion.
	Specifies a method for determining the moisture expansion of all ceramic tiles.
BDS ISO 10545 - 11	Ceramic tiles, Determination of crazing resistance for glazed tiles.
	Defines a test method for determining the crazing resistance of all glazed ceramic tiles except when the crazing is an inherent decorative feature of the product.
BDS ISO 10545 - 12	Ceramic tiles, Determination of frost resistance.
	Specifies a method for determining the frost resistance of all ceramic tiles intended for use in freezing conditions in the presence of water.
BDS ISO 10545 - 13	Ceramic tiles, Determination of chemical resistance.
	Specifies a test method for determining the chemical resistance of all ceramic tiles at room temperature. The method is applicable to all types of ceramic tiles.
BDS ISO 10545 - 14	Ceramic tiles, Determination of resistance to stains.
	Specifies a method for determining the resistance to stains of the proper surface of ceramic tiles.
BDS ISO 10545 - 15	Ceramic tiles, Determination of lead and cadmium given off by glazed tiles.
	Specifies a method for the determination of lead and cadmium given off by the glaze of ceramic tiles.
BDS ISO 10545 - 16	Ceramic tiles, Determination of small color differences.
	Describes a method for utilizing color measuring instruments for quantifying the small color differences between plain colored glazed ceramic tiles, which are designed to be uniform and consistent color. It permits the specification of a maximum acceptable value which depends only on the closeness of match and not on the nature of the color difference.

BDS EN 490	Concrete roofing tiles and fittings for roof covering and all cladding-Product specifications.
BDS ISO 13006	Ceramic tiles - Definitions, classification, characteristics and marking. This Standard defines terms and establishes classifications characteristics and marking requirements for ceramic tiles of the best commercial quality (first quality).
BDS EN 491	Concrete roofing tiles and fittings for roof covering and wall cladding-Test methods.
BDS EN 538	Clay roofing tiles for discontinuous laying-Flexural strength test.
BDS EN 539 - 1	Clay roofing tiles for discontinuous laying. Determination of physical characteristics-Part 1: Impermeability test.
BDS EN 1024	Clay roofing tiles for discontinuous laying-Determination of geometric characteristics.
BDS EN 1304	Clay roofing tiles and fittings-Product definitions and specifications.
BDS EN 13748 - 1	Terrazzo tiles-Part 1: Terrazzo tiles for internal use.
BDS EN 13748 - 2	Terrazzo tiles-Part 2: Terrazzo tiles for external use.

2.8 Steel

2.8.1 Reinforcing Steel

Reinforcing steel shall comply with the requirements specified in Sec 2.4.6 in this Part.

2.8.2 Structural Steel

Structural steel shall conform to Bangladesh Standards BDS 878: 1978, Specification for weld able structural steels; BDS 1355: 1992, Dimensions and properties of hot rolled steel beam, column, channel and angle sections. Where Bangladesh standards are not available, the relevant standards listed below shall be applicable.

BDS 1429	Light gauge steel sections.
BDS ISO 2566-1	Steel-Conversion of elongation values-Part 1: Carbon and low alloy steels.

BDS ISO 2566-2	Steel-Conversion of elongation values-Part 2: Austenitic steels.
BDS ISO 657-1	Hot-rolled steel sections-Part 1: Equal-leg angles-Dimensions.
BDS ISO 657-2	Hot-rolled steel sections-Part 2: Unequal-leg angles-Dimensions.
BDS ISO 657-5	Hot-rolled steel sections-Part V Equal-leg angles and unequal leg angles-Tolerances for metric and inch series.
BDS ISO 657-11	Hot-rolled steel sections-Part 11: Sloping flange channel sections (Metric series)-Dimensions and sectional properties.
BDS ISO 657-15	Hot-rolled steel sections-Part 15 Sloping flange beam sections (Metric series)-Dimensions and sectional properties.
BDS ISO 657-16	Hot-rolled steel sections-Part 16: Sloping flange column sections (metric series)-Dimensions and sectional properties.
BDS ISO 657-18	Hot-rolled steel sections-Part 18: L sections for shipbuilding (metric series) 104-Dimensions, sectional properties and tolerances.
BDS ISO 657-19	Hot-rolled steel sections-Part 19: Bulb flats (metric series)-Dimensions, sectional properties and tolerances.
BDS ISO 657-21	Hot-rolled steel sections-Part 21 T-sections with equal depth and flange width-Dimensions.
BDS ISO 10474	Steel and steel products-Inspection documents.
BDS ISO 14284	Steel and iron-Sampling and preparation of samples for the determination of chemical composition.
BDS ISO 9769	Steel and iron-Review of available methods of analysis.
BDS ISO 6929	Steel products-Definition and classification.
BDS ISO 20723	Structural steels-Surface condition of hot-rolled sections-Delivery requirements.
BDS ISO 24314	Structural steels-Structural steels for building with improved seismic resistance-Technical delivery conditions.

BDS ISO 404	Steel and steel products-General technical delivery requirements.
BDS ISO 1127	Stainless steel tubes-Dimensions, tolerances and conventional masses per unit length.
BDS ISO 4200	Plain end steel tubes, welded and seamless-General tables of dimensions and masses per unit length.
BDS ISO 6761	Steel tubes-Preparation of ends of tubes and fittings for welding.
ASTM A27/A27M	Standard Specification for Steel Castings, Carbon, for General Application.
ASTM A36/A36M	Standard Specification for Carbon Structural Steel.
ASTM A48/A48M	Standard Specification for Gray Iron Castings.
ASTM A53/A53M	Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless.
ASTM A148/A148M	Standard Specification for Steel Castings, High Strength, for Structural Purposes.
ASTM A242/A242M	Standard Specification for High-Strength Low-Alloy Structural Steel.
ASTM A252	Standard Specification for Welded and Seamless Steel Pipe Piles.
ASTM A283/A283M	Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates.
ASTM A307	Standard Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength.
ASTM A325	Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength.
ASTM A325M	Standard Specification for Structural Bolts, Steel, Heat Treated 830 MPa Minimum Tensile Strength [Metric].
ASTM A336/A336M	Standard Specification for Alloy Steel Forgings for Pressure and High-Temperature Parts.

ASTM A653/A653M	Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process.
ASTM A449	Standard Specification for Hex Cap Screws, Bolts and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use.
ASTM A490	Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength.
ASTM A500/A500M	Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes.
ASTM A501	Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing.
ASTM A514/A514M	Standard Specification for High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding.
ASTM A529/A529M	Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality.
ASTM A563	Standard Specification for Carbons and Alloy Steel Nuts.
ASTM A563M	Standard Specification for Carbon and Alloy Steel Nuts [Metric].
ASTM A1011/A1011M	Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength.
ASTM A572/A572M	Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel.
ASTM A588/A588M	Standard Specification for High-Strength Low-Alloy Structural Steel, up to 50 ksi [345 MPa] Minimum Yield Point, with Atmospheric Corrosion Resistance.
ASTM A606/A606M	Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance.

ASTM A1008/A1008M	Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Bake Harden able.
ASTM A618/A618M	Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing.
ASTM A666	Standard Specification for Annealed or Cold-Worked Austenitic Stainless Steel Sheet, Strip, Plate, and Flat Bar.
ASTM A668/A668M	Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use.
ASTM A690/A690M	Standard Specification for High-Strength Low-Alloy Nickel, Copper, Phosphorus Steel H-Piles and Sheet Piling with Atmospheric Corrosion Resistance for Use in Marine Environments.
ASTM A852/A852M	Standard Specification for Quenched and Tempered Low-Alloy Structural Steel Plate with 70 ksi [485 MPa] Minimum Yield Strength to 4 in. [100 mm] Thick.

2.8.3 Steel Plate, Sheet and Strips

These shall conform to the following standards.

BDS 868 : 1978	Code of practice for galvanized corrugated sheet roof and wall coverings.
BDS 1122: 1985	Specification for hot-dip galvanized steel sheet and coil.
BDS ISO 9328-1	Steel flat products for pressure purposes-Technical delivery conditions-Part 1: general requirements.
BDS ISO 9328-2	Steel flat products for pressure purposes-Technical delivery conditions-Part 2: Non-alloy and alloy steels with specified elevated temperature properties.
BDS ISO 9328-3	Steel flat products for pressure purposes-Technical delivery conditions -Part 3: Weldable fine grain steels, normalized.
BDS ISO 9328-4	Steel flat products for pressure purposes-Technical delivery conditions-Part 4: Nickel-alloy steels with specified low temperature properties.

BDS ISO 9328-5	Steel flat products for pressure purposes-Technical delivery conditions-Part 5: Weldable fine grain steels, thermo mechanically rolled.
BDS ISO 9328-6	Steel flat products for pressure purposes-Technical delivery conditions-Part 6: Weldable fine grain steels, quenched and tempered.
BDS ISO 9328-7	Steel flat products for pressure purposes-Technical delivery conditions-Part 7: Stainless steels.
BDS ISO 4995	Hot-rolled steel sheet of structural quality.
BDS ISO 7452	Hot-rolled structural steel plates-Tolerances on dimensions and shape.
BDS ISO 7778	Steel plate with specified through-Thickness characteristics.
BDS ISO 7788	Steel-Surface finish of hot-rolled plates and wide flats-Delivery requirements.
BDS ISO 9034	Hot-rolled structural steel wide flats-Tolerances on dimensions and shape.
BDS ISO 9364	Continuous hot-dip aluminum/zinc coated steel sheet of commercial, drawing and structural qualities.
BDS ISO 16160	Continuously hot-rolled steel sheet products-Dimensional and shape tolerances.
BDS ISO16162	Continuously cold-rolled steel sheet products-Dimensional and shape tolerances.
BDS ISO 16163	Continuously hot-dipped coated steel sheet products-Dimensional and shape tolerances.
IS 412	Specification for expanded metal steel sheets for general purposes.
IS 1079	Specification for hot rolled carbon steel sheet and strip.
IS 4030	Specification for cold-rolled carbon steel strip for general engineering purposes.
IS 7226	Specification for cold-rolled medium, high carbon and low-alloy steel strip for general engineering purposes.

IS 3502	Specification for steel chequered plates.
ASTM A109/A109M	Standard Specification for Steel, Strip, Carbon (0.25 Maximum Percent), Cold-Rolled.
ASTM A123/A123M	Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products.
ASTM A167	Standard Specification for Stainless and Heat-Resisting Chromium-Nickel Steel Plate, Sheet, and Strip.
ASTM A176	Standard Specification for Stainless and Heat-Resisting Chromium Steel Plate, Sheet, and Strip.
ASTM A240/A240M	Standard Specification for Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Pressure Vessels and for General Applications.
ASTM A263	Standard Specification for Stainless Chromium Steel-Clad Plate.
ASTM A264	Specification for Stainless Chromium-Nickel Steel-Clad Plate.
ASTM A285/A285M	Standard Specification for Pressure Vessel Plates, Carbon Steel, Low- and Intermediate-Tensile Strength.
ASTM A328/A328M	Standard Specification for Steel Sheet Piling.
ASTM A1008/A1008M	Standard Specification for Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, Solution Hardened, and Bake Harden able.
ASTM A414/A414M	Standard Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy for Pressure Vessels.
ASTM A424/A424M	Standard Specification for Steel, Sheet, for Porcelain Enameling.
ASTM A929/A929M	Standard Specification for Steel Sheet, Metallic-Coated by the Hot-Dip Process for Corrugated Steel Pipe.
ASTM A463/A463M	Standard Specification for Steel Sheet, Aluminum-Coated, by the Hot-Dip Process.

ASTM A480/A480M	Standard Specification for General Requirements for Flat-Rolled Stainless and Heat-Resisting Steel Plate, Sheet, and Strip.
ASTM A505	Standard Specification for Steel, Sheet and Strip, Alloy, Hot-Rolled and Cold-Rolled, General Requirements for.
ASTM A506	Standard Specification for Alloy and Structural Alloy Steel, Sheet and Strip, Hot-Rolled and Cold-Rolled.
ASTM A507	Standard Specification for Drawing Alloy Steel, Sheet and Strip, Hot-Rolled and Cold-Rolled.
ASTM A568/A568M	Standard Specification for Steel, Sheet, Carbon, Structural, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for.
ASTM A577/A577M	Standard Specification for Ultrasonic Angle-Beam Examination of Steel Plates.
ASTM A578/A578M	Standard Specification for Straight-Beam Ultrasonic Examination of Rolled Steel Plates for Special Applications.
ASTM A879/A879M	Standard Specification for Steel Sheet, Zinc Coated by the Electrolytic Process for Applications Requiring Designation of the Coating Mass on Each Surface.
ASTM A599/A599M	Standard Specification for Tin Mill Products, Electrolytic Tin-Coated, Cold-Rolled Sheet.
ASTM A606/A606M	Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance.
ASTM A635/A635M	Standard Specification for Steel, Sheet and Strip, Heavy-Thickness Coils, Hot-Rolled, Alloy, Carbon, Structural, High-Strength Low-Alloy, and High-Strength Low-Alloy with Improved Formability, General Requirements for.
ASTM A653/A653M	Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process.
ASTM A666	Standard Specification for Annealed or Cold-Worked Austenitic Stainless Steel Sheet, Strip, Plate, and Flat Bar.

ASTM A690/A690M	Standard Specification for High-Strength Low-Alloy Nickel, Copper, Phosphorus Steel H-Piles and Sheet Piling with Atmospheric Corrosion Resistance for Use in Marine Environments.
ASTM A775/A775M	Standard Specification for Epoxy-Coated Steel Reinforcing Bars.
ASTM A792/A792M	Standard Specification for Steel Sheet, 55 % Aluminum-Zinc Alloy-Coated by the Hot-Dip Process.
ASTM A857/A857M	Standard Specification for Steel Sheet Piling, Cold Formed, Light Gage.
ASTM A875/A875M	Standard Specification for Steel Sheet, Zinc-5% Aluminum Alloy-Coated by the Hot-Dip Process.

2.8.4 Steel Pipe, Tube and Fittings

These items shall conform to the following Standards:

BDS ISO 49	Malleable cast iron fittings threaded to ISO 7-1.
BDS ISO 3419	Non-alloy and alloy steel butt-welding fittings.
BDS ISO 3545-3	Steel tubes and fittings-Symbols for use in specifications-Part 3: Tubular fittings with circular cross-section.
BDS ISO 4144	Pipe work-Stainless steel fittings threaded in accordance with ISO 7-1.
BDS ISO 4145	Non-alloy steel fittings threaded to ISO 7-1.
BDS ISO 5251	Stainless steel butt-welding fittings.
ASTM A53/A53M	Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless.
ASTM A105/A105M	Standard Specification for Carbon Steel Forgings for Piping Applications.
ASTM A106/A106M	Standard Specification for Seamless Carbon Steel Pipe for High-Temperature Service.
ASTM A134	Standard Specification for Pipe, Steel, Electric-Fusion (Arc)-Welded (Sizes NPS 16 and Over).

ASTM A139/A139M	Standard Specification for Electric-Fusion (Arc)-Welded Steel Pipe (NPS 4 and Over).
ASTM A181/A181M	Standard Specification for Carbon Steel Forgings, for General-Purpose Piping.
ASTM A182/A182M	Standard Specification for Forged or Rolled Alloy and Stainless Steel Pipe Flanges, Forged Fittings, and Valves and Parts for High-Temperature Service.
ASTM A234/A234M	Standard Specification for Piping Fittings of Wrought Carbon Steel and Alloy Steel for Moderate and High Temperature Service.
ASTM A252	Standard Specification for Welded and Seamless Steel Pipe Piles.
ASTM A254	Standard Specification for Copper-Brazed Steel Tubing.
ASTM A268/A268M	Standard Specification for Seamless and Welded Ferritic and Martensitic Stainless Steel Tubing for General Service.
ASTM A269	Standard Specification for Seamless and Welded Austenitic Stainless Steel Tubing for General Service.
ASTM A270	Standard Specification for Seamless and Welded Austenitic Stainless Steel Sanitary Tubing.
ASTM A312/A312M	Standard Specification for Seamless, Welded, and Heavily Cold Worked Austenitic Stainless Steel Pipes.
ASTM A333/A333M	Standard Specification for Seamless and Welded Steel Pipe for Low-Temperature Service.
ASTM A334/A334M	Standard Specification for Seamless and Welded Carbon and Alloy-Steel Tubes for Low-Temperature Service.
ASTM A403/A403M	Standard Specification for Wrought Austenitic Stainless Steel Piping Fittings.
ASTM A420/A420M	Standard Specification for Piping Fittings of Wrought Carbon Steel and Alloy Steel for Low-Temperature Service.
ASTM A423/A423M	Standard Specification for Seamless and Electric-Welded Low-Alloy Steel Tubes.

ASTM A450/A450M	Standard Specification for General Requirements for Carbon and Low Alloy Steel Tubes.
ASTM A500/A500M	Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes.
ASTM A50	Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing.
ASTM A522/A522M	Standard Specification for Forged or Rolled 8 and 9% Nickel Alloy Steel Flanges, Fittings, Valves, and Parts for Low-Temperature Service.
ASTM A524	Standard Specification for Seamless Carbon Steel Pipe for Atmospheric and Lower Temperatures.
ASTM A530/A530M	Standard Specification for General Requirements for Specialized Carbon and Alloy Steel Pipe.
ASTM A589/A589M	Standard Specification for Seamless and Welded Carbon Steel Water-Well Pipe.
ASTM A618/A618M	Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing.
ASTM A632	Standard Specification for Seamless and Welded Austenitic Stainless Steel Tubing (Small-Diameter) for General Service.
ASTM A707/A707M	Standard Specification for Forged Carbon and Alloy Steel Flanges for Low-Temperature Service.
ASTM A733	Standard Specification for Welded and Seamless Carbon Steel and Austenitic Stainless Steel Pipe Nipples.
ASTM A778	Standard Specification for Welded, Un-annealed Austenitic Stainless Steel Tubular Products.
ASTM A807/A807M	Standard Practice for Installing Corrugated Steel Structural Plate Pipe for Sewers and Other Applications.
ASTM A865/A865M	Standard Specification for Threaded Couplings, Steel, Black or Zinc-Coated (Galvanized) Welded or Seamless, for Use in Steel Pipe Joints.

2.8.5 Steel Bars, Wire and Wire Rods

These shall conform to the following Standards.

BDS ISO 1035-1	Hot-rolled steel bars-Part 1: Dimensions of round bars.
BDS ISO 1035-2	Hot-rolled steel bars-Part 2: Dimensions of square bars.
BDS ISO 1035-3	Hot-rolled steel bars-Part 3: Dimensions of flat bars.
BDS ISO 1035-4	Hot-rolled steel bars-Part 4: Tolerances.
BDS ISO 4951-1	High yield strength steel bars and sections-Part 1: General delivery requirements.
BDS ISO 4951-2	High yield strength steel bars and sections-Part 2: Delivery conditions for normalized, normalized rolled and as-rolled steels.
BDS ISO 4951-3	High yield strength steel bars and sections-Part 3: Delivery conditions for thermo mechanically-rolled steels.
ASTM A29/A29M	Standard Specification for Steel Bars, Carbon and Alloy, Hot-Wrought, General Requirements for.
ASTM A49	Standard Specification for Heat-Treated Carbon Steel Joint Bars, Micro alloyed Joint Bars, and Forged Carbon Steel Compromise Joint Bars.
ASTM A108	Standard Specification for Steel Bar, Carbon and Alloy, Cold-Finished.
ASTM A116	Standard Specification for Metallic-Coated, Steel Woven Wire Fence Fabric.
ASTM A185/A185M	Standard Specification for Steel Welded Wire Reinforcement, Plain, for Concrete.
ASTM A227/A227M	Standard Specification for Steel Wire, Cold-Drawn for Mechanical Springs.
ASTM A228/A228M	Standard Specification for Steel Wire, Music Spring Quality.
ASTM A229/A229M	Standard Specification for Steel Wire, Oil-Tempered for Mechanical Springs.
ASTM A276	Standard Specification for Stainless Steel Bars and Shapes.

ASTM A311/A311M	Standard Specification for Cold-Drawn, Stress-Relieved Carbon Steel Bars Subject to Mechanical Property Requirements.
ASTM A322	Standard Specification for Steel Bars, Alloy, Standard Grades.
ASTM A108	Standard Specification for Steel Bar, Carbon and Alloy, Cold-Finished.
ASTM A368	Standard Specification for Stainless Steel Wire Strand.
ASTM A434	Standard Specification for Steel Bars, Alloy, Hot-Wrought or Cold-Finished, Quenched and Tempered.
ASTM A475	Standard Specification for Zinc-Coated Steel Wire Strand.
ASTM A478	Standard Specification for Chromium-Nickel Stainless Steel Weaving and Knitting Wire.
ASTM A479/A479M	Standard Specification for Stainless Steel Bars and Shapes for Use in Boilers and Other Pressure Vessels.
ASTM A492	Standard Specification for Stainless Steel Rope Wire.
ASTM A499	Standard Specification for Steel Bars and Shapes, Carbon Rolled from “T” Rails.
ASTM A510	Standard Specification for General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel.
ASTM A575	Standard Specification for Steel Bars, Carbon, Merchant Quality, M-Grades.
ASTM A576	Standard Specification for Steel Bars, Carbon, Hot-Wrought, Special Quality.
ASTM A580/A580M	Standard Specification for Stainless Steel Wire.
ASTM A586	Standard Specification for Zinc-Coated Parallel and Helical Steel Wire Structural Strand.
ASTM A603	Standard Specification for Zinc-Coated Steel Structural Wire Rope.
ASTM A627	Standard Test Methods for Tool-Resisting Steel Bars, Flats, and Shapes for Detention and Correctional Facilities.

ASTM A663/A663M	Standard Specification for Steel Bars, Carbon, Merchant Quality, Mechanical Properties.
ASTM A666	Standard Specification for Annealed or Cold-Worked Austenitic Stainless Steel Sheet, Strip, Plate, and Flat Bar.
ASTM A706/A706M	Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement.
ASTM A764	Standard Specification for Metallic Coated Carbon Steel Wire, Coated at Size and Drawn to Size for Mechanical Springs.
ASTM C933	Standard Specification for Welded Wire Lath.

2.8.6 Steel Fasteners

Steel fasteners shall conform to the following Standards:

BDS 1373	Slotted countersunk flat head tapping screws.
BDS 1374	Slotted raised counter.
BDS 1375	Fasteners hexagon products widths across flats.
BDS 1405	Bolts, screws, nuts and accessories terminology and nomenclature.
BDS 1406	Hexagon nuts style 2 products grades A and B.
BDS 1407	Hexagon nuts style 3 products grades A and B.
BDS 1408	General purpose screw threads general plan.
BDS 1409	General purpose screw threads selected sizes for screws, bolts and nuts.
BDS 1410	Thread run-outs for fasteners thread of BDS 1408: 1995 and BDS 1409: 1993.
BDS 1411	Tapping screws thread.
BDS 1412	Thread undercuts of external metric thread fasteners.
BDS 1413	Head configuration and gauging of countersunk head screws.
BDS 1428	Fasteners-bolts, screws, studs and nuts-symbols and designations of dimensions.

ASTM A31	Standard Specification for Steel Rivets and Bars for Rivets, Pressure Vessels.
ASTM A183	Standard Specification for Carbon Steel Track Bolts and Nuts.
ASTM A193/A193M	Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High Temperature or High Pressure Service and Other Special Purpose Applications.
ASTM A194/A194M	Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High Temperature Service, or Both.
ASTM A307	Standard Specification for Carbon Steel Bolts and Studs, 60000 psi Tensile Strength.
ASTM A320/A320M	Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for Low-Temperature Service.
ASTM A325	Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength.
ASTM A354	Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners.
ASTM A437/A437M	Standard Specification for Stainless and Alloy-Steel Turbine-Type Bolting Specially Heat Treated for High-Temperature Service.
ASTM A449	Standard Specification for Hex Cap Screws, Bolts and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use.
ASTM A489	Standard Specification for Carbon Steel Lifting Eyes.
ASTM A490	Standard Specification for Structural Bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength.
ASTM A502	Standard Specification for Rivets, Steel, Structural.
ASTM A540/A540M	Standard Specification for Alloy-Steel Bolting for Special Applications.
ASTM A563	Standard Specification for Carbons and Alloy Steel Nuts.

ASTM A574	Standard Specification for Alloy Steel Socket-Head Cap Screws.
ASTM C514	Standard Specification for Nails for the Application of Gypsum Board.
ASTM C954	Standard Specification for Steel Drill Screws for the Application of Gypsum Panel Products or Metal Plaster Bases to Steel Studs from 0.033 in. (0.84 mm) to 0.112 in. (2.84 mm) in Thickness.
ASTM C955	Standard Specification for Load-Bearing (Transverse and Axial) Steel Studs, Runners (Tracks), and Bracing or Bridging for Screw Application of Gypsum Panel Products and Metal Plaster Bases.
ASTM C1002	Standard Specification for Steel Self-Piercing Tapping Screws for the Application of Gypsum Panel Products or Metal Plaster Bases to Wood Studs or Steel Studs.
ASTM F436	Standard Specification for Hardened Steel Washers.
ASTM F593	Standard Specification for Stainless Steel Bolts, Hex Cap Screws, and Studs.
ASTM F594	Standard Specification for Stainless Steel Nuts.
ASTM F844	Standard Specification for Washers, Steel, Plain (Flat), Unhardened for General Use.
ASTM F959	Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners.

2.8.7 Welding Electrodes and Wires

Welding electrodes and wires shall conform to the following Standards:

BDS 239	Specification for soft solder.
BDS 1442-1	Filler rods and wire for gas shielded arc-welding-ferric steel.
BDS 1442-2	Filler rods and wire for gas shielded arc-welding-austenitic

	stainless steel.
BDS 1442-3	Filler rods and wires for gas shielded arc welding-copper and copper alloy.
BDS 1442-4	Filler rods and wires for gas shielded arc welding-aluminum and aluminum alloy and magnesium alloys.
BDS 1442-5	Filler rods and wires for gas shielded arc welding-nickel and nickel alloys.
IS 814	Specification for covered electrodes for manual metal arc welding of carbon and carbon manganese steel.
IS 815	Classification and coding of covered electrodes for metal arc welding of structural steels.
IS 1278	Specification for filler rods and wires for gas welding.
IS 1395	Specification for low and medium alloy steel covered electrodes for manual metal arc welding.
IS 3613	Acceptance tests for wire flux combinations for submerged-arc welding of structural steel.
IS 4972	Specification for resistance spot-welding electrodes.
IS 6419	Specification for welding rods and bare electrodes for gas shielded arc welding of structural steel.
IS 6560	Specification for molybdenum and chromium-molybdenum low alloy steel welding rods and bare electrodes for gas shielded arc welding.
IS 7280	Specification for base wire electrodes for submerged-arc welding of structural steels.
IS 8363	Specification for bare wire electrodes for electro slag welding of steels.
ISO 9453	Soft solder alloys-chemical compositions and forms.
ISO 9454	Soft soldering fluxes-classification and requirements.
	Part 1: Classification, labeling and packaging.
ISO 9455-1	Soft soldering fluxes- test methods.

	Part 1: Determination of non-volatile matter, gravimetric method.
ISO 9455-8	Soft soldering fluxes-test methods.
	Part 8: Determination of zinc content.
ISO 9455-11	Soft soldering fluxes-test methods.
	Part 11: Solubility of flux residues.
ISO 9455-14	Soft soldering fluxes-test methods.
	Part 14: Assessment of tackiness of flux residues.

2.9 Timber & Wood Products

2.9.1 Timber Types and Properties

Timber types for the structural purpose with their engineering characteristics are contained in Table 6.11.1 Part 6 of this Code. Details of the uses of timber in structures or elements of structures including terminology, material requirements, and moisture content preferred cut sizes of sawn timbers, grading, permissible defects, suitability in respect of durability and treatability, design criteria, and details of joints are also given in Chapter 11 Part 6. Timber and timber constructions shall satisfy the requirements of that Chapter and conform to the following Standards:

BDS 142	Specification for wood doors.
BDS 173	Specification for wood windows.
BDS 230	Glossary of terms applicable to timber, plywood and joinery.
BDS 803	Trade names and abbreviated symbols for timber species.
BDS 819	Code of practice for preservation of timber.
BDS 820	Recommendation for maximum permissible moisture content of timber used for different purposes in Bangladesh.
BDS 857	Specification for grading rules for logs and sawn timbers.
BDS 1090	Methods of test for plywood.

BDS 1256	Classification of commercial timber.
BDS 1311	Key for identification of commercial timber.

2.9.2 Plywood

A wood structural panel comprised of plies of wood veneer arranged in cross-aligned layers. The plies are bonded with waterproof adhesive that cures on application of heat and pressure.

Plywood shall conform to the following Standards:

BDS 799	Specification for plywood for general purposes.
BDS 1158	Specification for veneered decorative plywood.

For sampling and testing of plywood, the following Standards are applicable:

BDS 1087	Specification for method of sampling of plywood.
BDS 1090	Methods of test of plywood.
IS 4990	Specification for plywood for concrete shattering work.
IS 5509	Specification for fire retardant plywood.
IS 5539	Specification for Preservative Treated Plywood.

2.9.3 Particle Boards and Fibre Boards

A panel primarily composed of cellulosic materials (usually wood), generally in the form of discrete pieces or particles, as distinguished from fibers. The cellulosic material is combined with synthetic resin or other suitable bonding system by a process in which the inter-particle bond is created by the bonding system under heat and pressure.

Fiber boards are fibrous, homogeneous panel made from lingo-cellulosic fibers (usually wood or cane) and having a density of less than 497 kg per cubic meter but more than 160 kg per cubic meter.

These materials shall conform to the following standards:

BDS 619	Specification for particle board (medium density).

BDS 620	Specification for hardboard.
BDS EN 316	Wood fiberboards-Definition, classification and symbols.
ISO 820	Particle boards-Definition and classification.
ISO 821	Particle boards-Determination of dimensions of test pieces.
ISO 822	Particle boards-Determination of density.
ISO 823	Particle boards-Determination of moisture content.
ISO 766	Fibre building boards-Determination of dimensions of test pieces.
ISO 767	Fibre building boards-Determination of moisture content.
ISO 768	Fibre building boards-Determination of bending strength.
ISO 769	Fibre building boards-Hard and medium boards-determination of.

Water Absorption and of Swelling in Thickness after Immersion in Water;

ISO 818	Fibre building boards-Definition-Classification.
ISO 819	Fibre building boards-Determination of density.
ISO 2695	Fibre building boards-Hard and medium boards for general.

Purposes-Quality Specifications-Appearance, Shape and Dimensional Tolerances;

ISO 2696	Fibre building boards-Hard and medium boards-Quality.

Specifications- Water Absorption and Swelling in Thickness;

ISO 3340	Fibre building boards-Determination of sand content.
ISO 3346	Fibre building boards-Determination of surface finish (roughness).
ISO 3729	Fibre building boards-Determination of surface stability.
ISO/TR 7469	Dimensional stability of hardboards.

Wood based Laminates

Laminated boards having a core of strips, each not exceeding 7 mm in thickness, glued together face to face to form a slab which in turn is glued between two or more veneers, with the direction of the grain of the core strips running at right angles to that of the adjacent outer veneers.

Wood based laminates shall conform to the following Standards:

IS 3513	Specification for resin treated compressed wood laminates (compregs).
	Part 3 For general purposes.
IS 3513	Specification for resin treated compressed wood laminates (compregs).
	Part 4 Sampling and Tests.
IS 9307 (Parts I to VIII)	Methods of tests for wood-based structural sandwich construction.
	Part I Flexure test.
	Part II Edgewise compression test.
	Part III Flatwise compression test.
	Part IV Shear test.
	Part V Flatwise tension test.
	Part VI Flexure creep test.
	Part VII Cantilever vibration test.
	Part VIII Weathering test.

2.9.4 Adhesives and Glues

Adhesives and glues are used to join two or more parts so as to form a single unit. Adhesives shall conform to the following Standards:

IS 848	Specification for synthetic resign adhesives for plywood (phenolic and aminoplastic).
IS 849	Specification for cold setting case in glue for wood.
IS 851	Specification for synthetic resin adhesives for construction work (nonstructural) in wood.

IS 852	Specification for animal glue for general wood-working purposes.
IS 4835	Specification for polyvinyl acetate dispersion-based adhesives for wood.
IS 9188	Specification for adhesive for structural laminated wood products for use under exterior exposure condition.

2.10 Doors, Windows and Ventilators

2.10.1 Wooden Doors, Windows and Ventilators

These shall conform to the following Standards:

BDS 142	Specification for wood door.
BDS 173	Specification for wood windows.
BDS 820	Recommendation for maximum permissible moisture content of timber used for different purposes in Bangladesh.
BDS 1504	Timber door window and ventilator frames
IS 1003	Specification for timber panelled and glazed shutters.
	Part 1- 2003 Door shutters.
	Part 2- 1994 Window and ventilator shutters.
IS 1826	Specification for venetian blinds for windows.
IS 2191	Specification for wooden flush door shutters (cellular and hollow core type).
	Part 1 Plywood face panels.
	Part 2 Particle board face panels and hardboard face panels.
IS 2202	Specification for wooden flush door shutters (solid core type).
	Part 1 Plywood face panels.
	Part 2 Particle board face panels and hardboard face panels.
IS 4020	Method of tests for door shutters.
	(Part 1): 1998 General.

(Part 2): 1998	Measurement of dimensions and squareness.
(Part 3): 1998	Measurement of general flatness.
(Part 4): 1998	Local planeness test.
(Part 5): 1998	Impact indentation test.
(Part 6): 1998	Flexure test.
(Part 7): 1998	Edge loading test.
(Part 8): 1998	Shock resistance test.
(Part 9): 1998	Buckling resistance test.
(Part 10):1998	Slamming test.
(Part 11):1998	Misuse test.
(Part 12):1998	Varying humidity test.
(Part 13):1998	End immersion test.
(Part 14):1998	Knife test.
(Part 15):1998	Glue adhesion test.
(Part 16):1998	Screw withdrawal resistance test.

IS 4021	Specification for timber door, window and ventilator frames.
IS 4962	Specification for wooden side sliding doors.
IS 6198	Specification for ledged, braced and battened timber shutters.

2.10.2 Metal Doors, Windows Frames and Ventilators

These shall conform to the following Standards:

BDS 1270	Specification for strong room door
BDS 1273	Specification for vault doors.
IS 1038	Specification for steel doors, windows and ventilators.
IS 1361	Specification for steel windows for industrial buildings.
IS 1948	Specification for aluminum doors, windows and ventilators.
IS 1949	Specification for aluminum windows for industrial buildings.

IS 4351	Specification for steel door frames.
IS 6248	Specification for metal rolling shutters and rolling grills.
IS 7452	Specification for hot rolled steel sections for doors, windows and ventilators.
IS 10451	Specification for steel sliding shutters (top hung type).
IS 10521	Specification for collapsible gates.

2.10.3 Plastic Doors and Windows

These shall conform to the following Standards:

BDS EN 477	Unplasticized polyvinylchloride (PVC-U) profiles for the fabrication of windows and doors-Determination of the resistance to impact of main profiles by falling mass.
BDS EN 478	Unplasticized polyvinylchloride (PVC-U) profiles for the fabrication of windows and doors-Determination of appearance after exposure at 150ºC.
BDS EN 479	Unplasticized polyvinylchloride (PVC-U) profiles for the fabrication of windows and doors-Determination of heat reversion.
BDS EN 513	Unplasticized polyvinylchloride (PVC-U) profiles for the fabrication of windows and doors-Determination of the resistance to artificial weathering.
BDS EN 514	Unplasticized polyvinylchloride (PVC-U) profiles for the fabrication of windows and doors-Determination of the strength of welded corners and T-joints.
BDS EN 12608	Unplasticized polyvinylchloride (PVC-U) profiles for the fabrication of windows and doors-Classification, requirements and test methods.
BDS ISO 1163-1	Plastics-Unplasticized polyvinylchloride (PVC-U) molding and extrusion materials-Part 1: Designation system and basis for specifications.

BDS ISO 1163-2	Plastics-Unplasticized polyvinylchloride (PVC-U) molding and extrusion materials-Part 2: Preparation of test specimens and determination of properties.
IS 14856	Specification for glass fibre reinforced (GRP) panel type door shutters for internal use.
IS 15380	Specification for molded raised high density fibre (HDF) panel doors.

2.11 Aluminium and Aluminium Alloys

Aluminum used for structural purposes in buildings and structures shall comply with AA ASM 35 and AA ADM 1.

Aluminium and Aluminium Alloys shall also conform to the following Standards:

BDS EN 755-9	Aluminum and aluminum alloys-Extruded rod/bar, tube and profiles-Part 9: Profiles, tolerances on dimensions and form.
BDS EN 755-2	Aluminum and aluminum alloys-Extruded rod/bar, tube and profiles-Part 2: Mechanical properties.
BDS EN 755-1	Aluminum and aluminum alloys-Extruded rod/bar, tube and profiles-Part 1: Technical conditions for inspection and delivery.
BDS EN 755-3	Aluminum and aluminum alloys-Extruded rod/bar, tube and profiles-Part 3: Round bars, tolerances on dimensions and form.
BDS EN 755-4	Aluminum and aluminum alloys-Extruded rod/bar, tube and profiles-Part 4: Square bars, tolerances on dimensions and form.
BDS EN 755-5	Aluminum and aluminum alloys-Extruded rod/bar, tube and profiles-Part 5: Rectangular bars, tolerances on dimensions and form.
BDS EN 755-6	Aluminum and aluminum alloys-Extruded rod/bar, tube and profiles-Part 6: Hexagonal bars, tolerances on dimensions and form.
BDS EN 755-7	Aluminum and aluminum alloys-Extruded rod/bar, tube and profiles-Part 7: Seamless tubes, tolerances on dimensions and form.

BDS EN 755-8	Aluminum and aluminum alloys-Extruded rod/bar, tube and profiles-Part 8: Porthole tubes, tolerances on dimensions and form.
BDS EN 12020-1	Aluminum and aluminum alloys- Extruded precision profiles in alloys EN AW-6060 and EN AW-6063- Part 1: Technical conditions for inspection and delivery.
BDS EN 12020-2	Aluminum and aluminum alloys-Extruded precision profiles in alloys EN AW-6060 and EN AW-6063-Part 2: Tolerances on dimensions and form.
BDS EN 515	Aluminum and aluminum alloys-Wrought products-Temper designations.
ASTM B26/B26M	Standard Specification for Aluminum-Alloy Sand Castings.
ASTM B85/B85M	Standard Specification for Aluminum-Alloy Die Castings.
ASTM B108/B108M	Standard Specification for Aluminum-Alloy Permanent Mold Castings.
ASTM B209	Standard Specification for Aluminum and Aluminum-Alloy Sheet and Plate.
ASTM B210	Standard Specification for Aluminum and Aluminum-Alloy Drawn Seamless Tubes.
ASTM B211	Standard Specification for Aluminum and Aluminum-Alloy Bar, Rod, and Wire.
ASTM B221	Standard Specification for Aluminum and Aluminum-Alloy Extruded Bars, Rods, Wire, Profiles, and Tubes.
ASTM B241/B241M	Standard Specification for Aluminum and Aluminum-Alloy Seamless Pipe and Seamless Extruded Tube.
ASTM B308/B308M	Standard Specification for Aluminum-Alloy 6061-T6 Standard Structural Profiles.
ASTM B313/B313M	Standard Specification for Aluminum and Aluminum-Alloy Round Welded Tubes.
ASTM B316/B316M	Standard Specification for Aluminum and Aluminum-Alloy Rivet and Cold-Heading Wire and Rods.
ASTM B429/B429M	Standard Specification for Aluminum-Alloy Extruded Structural Pipe and Tube.

ASTM B483/B483M	Standard Specification for Aluminum and Aluminum-Alloy Drawn Tube and Pipe for General Purpose Applications.
ASTM B547/B547M	Standard Specification for Aluminum and Aluminum-Alloy Formed and Arc-Welded Round Tube.
ASTM B632/B632M	Standard Specification for Aluminum-Alloy Rolled Tread Plate.
ASTM B745/B745M	Standard Specification for Corrugated Aluminum Pipe for Sewers and Drains.
ASTM E34	Standard Test Methods for Chemical Analysis of Aluminum and Aluminum-Base Alloys.

2.12 Builders Hardware

The applicable Standards are listed below:

BDS 113	Specification for latches and locks for doors in buildings.
IS 204	Specification for tower bolts.
	Part 1 Ferrous metals.
	Part 2 Nonferrous metals.
IS 205	Specification for nonferrous metal butt hinges.
IS 206	Specification for tee and strap hinges.
IS 208	Specification for door handles.
IS 281	Specification for mild steel sliding door bolts for use with padlock.
IS 362	Specification for parliament hinges.
IS 363	Specification for hasps and staples.
IS 364	Specification for fanlight catch.
IS 452	Specification for door springs, rat-tail type.
IS 453	Specification for double acting spring hinges.
IS 729	Specification for drawer locks, cupboard locks and box locks.
IS 1019	Specification for rim latches.
IS 1341	Specification for steel butt hinges.

IS 1823	Specification for floor door stoppers.
IS 1837	Specification for fanlight pivots.
IS 2209	Specification for mortise locks (vertical type).
IS 2681	Specification for nonferrous metal sliding door bolts for use with padlocks.
IS 3564	Specification for door closers (hydraulically regulated).
IS 3818	Specification for continuous (piano) hinges.
IS 3828	Specification for ventilator chains.
IS 3843	Specification for steel back-flap hinges.
IS 3847	Specification for mortise night latches.
IS 4621	Specification for indicating bolts for use in public baths and lavatories.
IS 4948	Specification for welded steel wire fabric for general use.
IS 4992	Specification for door handles for mortise locks (vertical type).
IS 5187	Specification for flush bolts.
IS 5899	Specification for bathroom latches.
IS 5930	Specification for mortise latch (vertical type).
IS 6315	Specification for floor springs (hydraulically regulated) for heavy doors.
IS 6318	Specification for plastic window stays and fasteners.
IS 6343	Specification for door closers (pneumatically regulated) for light doors weighing up to 40 kg.
IS 6602	Specification for ventilator poles.
IS 6607	Specification for rebated mortise locks (vertical type).
IS 7196	Specification for hold fast.
IS 7197	Specification for double action floor springs (without oil check) for heavy doors.
IS 7534	Specification for sliding locking bolts for use with padlocks.

IS 7540	Specification for mortise dead locks.
IS 8756	Specification for ball catches for use in wooden almirah.
IS 8760	Specification for mortise sliding door locks, with lever mechanism.
IS 9106	Specification for rising butt hinges.
IS 9131	Specification for rim locks.
IS 9460	Specification flush drop handle for drawer.
IS 9899	Specification for hat, coat and wardrobe hooks.
IS 10019	Specification for steel window stays and fasteners.
IS 10090	Specification for numerical.
IS 10342	Specification for curtain rail system.
IS 12817	Specification for stainless steel butt hinges.
IS 12867	Specification for PVC hand rails covers.
IS 14912	Specification for door closers concealed type (hydraulically regulated)

2.13 Roof Coverings

2.13.1 Scope

The provisions of this Section shall govern the materials used for roof coverings.

2.13.2 Compatibility of Materials

All roofs and roof coverings shall be of materials that are compatible with each other and with the building or structure to which the materials are applied.

2.13.3 Material Specifications and Physical Characteristics

All materials to be used in the construction of roofs and roof coverings shall conform to the applicable standards listed in this Section. In the absence of applicable standards or when materials are of questionable suitability, testing by an approved testing agency may be required by the building official to determine the character, quality and limitations of use of the materials.

2.13.4 Weather Protection

All roofs shall be covered with approved roof coverings properly secured to the building or structure to resist wind and rain. Roof coverings shall be designed, installed and maintained in accordance with approved manufacturer's recommendations such that the roof covering shall serve to protect the building or structure.

2.13.5 Wind Resistance

All roofs and roof coverings shall be secured in place to the building or structure to withstand the wind loads.

2.13.6 Structural and Construction Loads

The structural roof components shall be capable of supporting the roof covering system and the material and equipment loads that will be encountered during installation of the roof covering system.

2.13.7 Impact Resistance

Roof coverings shall resist impact damage based on the results of tests conducted in accordance with ASTM D4272 or ASTM D3746.

2.13.8 Metal-Sheet Roof Coverings

Metal-sheet roof coverings installed over structural framing and decking shall comply with BDS 868, Galvanized corrugated sheet roof and wall coverings; BDS 1122, Hot-dip galvanized steel sheet and coil; ASTM A755/A755M or ASTM B101. Metal-sheet roof coverings shall be installed in accordance with approved manufacturer's installation instructions.

2.13.9 Interlocking Clay or Cement Tile

Interlocking clay or cement tile shall be installed only over solid sheathing or spaced structural sheathing boards. Interlocking clay or cement tile shall not be installed on roof slopes below one unit vertical in three units horizontal (1:3). Horizontal battens shall be required on roof slopes over one unit vertical in two units horizontal (1:2). Single layer underlayment is required over solid sheathing on all roof slopes. Reinforced underlayment shall be required when spaced sheathing is used. Regardless of roof slope, the first three tile courses and all tiles within 900 mm of roof edges, tiles at changes in roof slope or changes in slope direction, shall be fastened to the roof. For the field of the roof, fastening is not required on roof slopes below one unit vertical in two units horizontal (1:2). Every other tile course shall be fastened on roof slopes 1:2 to less than

1:1; and every tile shall be fastened on roof slopes 1:1 and over. Tile overlap shall be in accordance with approved manufacturer's installation instructions.

2.13.10 Non-interlocking Clay or Cement Tile

Non-interlocking clay or cement tile shall not be installed on roof slopes below one unit vertical in five units horizontal (1:5). Double layer underlayment is required on roof slopes below one unit vertical in four units horizontal (1:4). Single layer underlayment is required on all other roof slopes. Non-interlocking clay or cement tile shall be secured to the roof with two fasteners per tile. The minimum tile overlap shall be 75 mm.

2.13.11 Roof Insulation

Rigid combustible roof insulation shall be permitted, provided the insulation is covered with approved roof coverings directly applied thereto. In-situ lime concrete may be used on flat roofs of buildings. Minimum compacted thickness of such a layer shall be 75 mm and have adequate slope for drainage. The materials used in lime concrete shall conform to the standards specified in Sec 2.5 of this Part.

2.13.12 Recovering and Replacement of Roof Coverings

New roof coverings shall not be installed without first removing existing roof coverings when the existing roof or roof covering is water soaked or has deteriorated to the point that the existing roof or roof covering is not acceptable as a base for additional roofing.

2.13.13 Reuse of Materials

Existing slate, clay or cement tile shall be permitted for reuse, except that damaged, cracked or broken slate or tile shall not be reused. Existing vent flashings, metal edgings, drain outlets, collars and metal counter flashings shall not be reused where rusted, damaged or deteriorated. Aggregate surfacing materials shall not be reused.

2.13.14 Applicable Standards

The applicable Standards for materials used in roofs and roof coverings are listed below:

BDS 868	Code of practice for galvanized corrugated sheet roof and wall coverings.
BDS 1122	Specification for hot-dip galvanized steel and coil.
BDS EN 490	Concrete roofing tiles and fittings for roof covering and all cladding-Product specifications.
BDS EN 491	Concrete roofing tiles and fittings for roof covering and wall cladding-Test methods.

BDS EN 538	Clay roofing tiles for discontinuous laying-Flexural strength test.
BDS EN 539-l	Clay roofing tiles for discontinuous laying Determination of physical characteristics-Part 1: Impermeability test.
BDS EN 1024	Clay roofing tiles for discontinuous laying-Determination of geometric characteristics.
BDS EN 1304	Clay roofing tiles and fittings-Product definitions and specifications.
ASTM A755/A755M	Standard Specification for Steel Sheet, Metallic Coated by the Hot-Dip Process and Pre-painted by the Coil-Coating Process for Exterior Exposed Building Products.
ASTM B101	Standard Specification for Lead-Coated Copper Sheet and Strip for Building Construction.
ASTM C406	Standard Specification for Roofing Slate.
ASTM C836/C836M	Standard Specification for High Solids Content, Cold Liquid-Applied Elastomeric Waterproofing Membrane for Use with Separate Wearing Course.
ASTM C1029	Standard Specification for Spray-Applied Rigid Cellular Polyurethane Thermal Insulation.
ASTM D225	Standard Specification for Asphalt Shingles (Organic Felt) Surfaced With Mineral Granules.
ASTM D226/D226M	Standard Specification for Asphalt-Saturated Organic Felt Used in Roofing and Waterproofing.
ASTM D227	Standard Specification for Coal-Tar-Saturated Organic Felt Used in Roofing and Waterproofing.
ASTM D312	Standard Specification for Asphalt Used in Roofing.
ASTM D450	Standard Specification for Coal-Tar Pitch Used in Roofing, Damp proofing, and Waterproofing.
ASTM D1227	Standard Specification for Emulsified Asphalt Used as a Protective Coating for Roofing.
ASTM D1863	Standard Specification for Mineral Aggregate Used on Built-Up

	Roofs.
ASTM D2178	Standard Specification for Asphalt Glass Felt Used in Roofing and Waterproofing.
ASTM D2626	Standard Specification for Asphalt-Saturated and Coated Organic Felt Base Sheet Used in Roofing.
ASTM D2898	Standard Practice for Accelerated Weathering of Fire-Retardant-Treated Wood for Fire Testing.
ASTM D3161	Standard Test Method for Wind-Resistance of Asphalt Shingles (Fan-Induced Method).
ASTM D3747	Standard Specification for Emulsified Asphalt Adhesive for Adhering Roof Insulation.
ASTM D3909	Standard Specification for Asphalt Roll Roofing (Glass Felt) Surfaced With Mineral Granules.
ASTM D4272	Standard Test Method for Total Energy Impact of Plastic Films By Dart Drop.
ASTM D4434/D4434M	Standard Specification for Poly (Vinyl Chloride) Sheet Roofing.
ASTM D4601	Standard Specification for Asphalt-Coated Glass Fiber Base Sheet Used in Roofing.
ASTM D4637	Standard Specification for EPDM Sheet Used In Single-Ply Roof Membrane.
ASTM D4897/D4897M	Standard Specification for Asphalt-Coated Glass-Fiber Venting Base Sheet Used in Roofing.
ASTM D6380	Standard Specification for Asphalt Roll Roofing (Organic Felt).
ASTM E108	Standard Test Methods for Fire Tests of Roof Coverings.
ASTM G90	Standard Practice for Performing Accelerated Outdoor Weathering of Nonmetallic Materials Using Concentrated Natural Sunlight.
ASTM G154	Standard Practice for Operating Fluorescent Light Apparatus for UV Exposure of Nonmetallic Materials.
ASTM G155	Standard Practice for Operating Xenon Arc Light Apparatus for Exposure of Non-Metallic Materials.

CGSB (Canadian General Standards Board)	Membrane, modified bituminous, prefabricated, and reinforced for roofing.
37-GP-56M-80	
FM 447-86	Approval standard for class I roof coverings.
FM (Factory Manual) 4450-89	Standard laboratories department approved standard for class I insulated steel deck roofs.
RMA (Rubber Manufacturer Association, USA)	Wind design guide for ballasted single-ply roofing systems.
RP-4-88	
SPRI (Single Ply Roofing Institute, USA) -86	Wind design guide for ballasted single-ply roofing systems.

2.14 Paints and Varnishes

2.14.1 Water Based Paints and Pigments

Water based paints shall conform to the following Standards:

BDS 500	Specification for distemper dry.
BDS 1097	Specification for plastic emulsion paint.
	Part 1 for Interior use.
	Part 2 for Exterior use.
IS 427	Specification for distemper, dry, color as required.
IS 428	Specification for distemper, washable.
IS 5410	Specification for cement paint, color as required.
IS 5411	Specification for plastic emulsion paint.
	Part 1: For interior use.
	Part 2: For exterior use

2.14.2 Ready Mixed Paints, Enamels and Powder Coatings

Ready mixed paints and enamels shall conform to the following Standards:

BDS 13	Specification for ready mixed paints, varnish, lacquers and related products.
BDS 14	Specification for black bituminous paint, brushing for general purposes.
BDS 397	Specification for ready mixed paint, brushing, red oxide zinc chrome, priming.
BDS 398	Specification for ready mixed paint, spraying, red oxide zinc chrome, priming.
BDS 399	Specification for aluminum paint, spraying for general purposes, in dual container.
BDS 400	Specification for aluminum paint, brushing, for general purposes in dual container.
BDS 401	Specification for varnish, finishing, exterior, type-I, (synthetic).
BDS 402	Specification for ready mixed paint, brushing, finishing, semi-gloss, for general purposes.
BDS 499	Specification for ready mixed paints, brushing, for road marking (white, yellow and black).
BDS 616	Specification for enamel, brushing, exterior (i) undercoating, (ii) finishing, color as required.
BDS 617	Specification for enamel, brushing, interior (i) undercoating, (ii) finishing, color as required.
BDS 926	Specification for ready mixed paint, brushing, petrol resisting, air drying, for exterior painting of containers, color as required.
BDS 927	Specification for ready mixed paint, brushing, petrol resisting, air drying, for interior painting of tanks and containers, red oxide (color unspecified).
BDS 928	Specification for ready mixed paint, brushing, acid resisting, for protection against acid fumes, color as required.
BDS 973	Specification for specification and methods of test for linseed stand oil for paints and varnishes.

BDS 974 Specification and methods of test for raw tung oils for paints and varnishes.

BDS 1005 Specification for ready mixed paint, brushing, finishing, stoving, enamel, color as required.

BDS 1141 Specification for ready mixed aluminum priming paints for woodwork.

BDS 1151 Specification for pavement marking paints.

IS 101 Methods of sampling and test for paints, varnishes and related products:

(Part l/Sec 1): Test on liquid paints (general and physical), Section 1 Sampling.

(Part l/Sec 2): Test on liquid paints (general and physical), Section 2 Preliminary examination and preparation of samples for testing.

(Part l/Sec 3): Test on liquid paints (general and physical), Section 3 Preparation of panels.

(Part l/Sec 4): Test on liquid paints (general and physical), Section 4 Brushing test.

(Part l/Sec 5): Test on liquid paints (general and physical), Section 5 Consistency.

(Part l/Sec 6): Test on liquid paints (general and physical), Section 6 Flash point.

(Part l/Sec 7): Test on liquid paints (general and physical), Section 7 Mass per 10 Iitres.

(Part 2/Sec 1): Test on liquid paints (chemical examination), Section 1 Water content.

(Part 2/Sec 2): Test on liquid paints (chemical examination), Section 2 Volatile matter.

(Part 3/Sec 1): Tests on paint film formation, Section 1 Drying time.

(Part 3/Sec 2): Tests on paint film formation, Section 2 Film thickness.

(Part 3/Sec 4): Tests on paint film formation, Section 4 Finish.

(Part 3/Sec 5): Tests on paint film formation, Section 5 Fineness of grind

(Part 4/Sec 1): Optical test, Section 1 Opacity.

(Part 4/Sec 2): Optical test, Section 2 Color.

(Part 4/Sec 3): Optical test, Section 3 Light fastness test.

(Part 4/Sec 4): Optical test, Section 4 Gloss.

(Part 5/Sec 1): Mechanical test on paint films, Section 1 Hardness tests.

(Part 5/Sec 2): Mechanical test on paint films, Section 2 Flexibility and adhesion.

(Part 5/Sec 3): Mechanical test on paint films, Section 3 Impact resistance.

(Part 5/Sec 4): Mechanical test on paint films, Section”4 Print free test.

(Part 6/Sec 1): Durability tests, Section 1 Resistance to humidity under conditions of condensation.

(Part 6/Sec 2): Durability tests, Section 2 Keeping properties.

(Part 6/Sec 3): Durability tests, Section 3 Moisture vapour permeability.

(Part 6/Sec 4): Durability tests, Section 4 Degradation of coatings (pictorial aids for evaluation).

(Part 6/Sec 5): Durability tests, Section 5 Accelerated weathering test.

(Part 7/Sec 1): Environmental tests on paint films, Section 1 Resistance to water.

(Part 7/Sec 2): Environmental tests on paint films, Section 2 Resistance to liquids.

(Part 7/Sec 3): Environmental tests on paint films, Section 3 Resistance to heat.

(Part 7/Sec 4): Environmental tests on paint films, Section 4 Resistance to bleeding of pigments.

(Part 8/Sec 1): Tests for pigments and other solids, Section 1 Residue on sieve.

(Part 8/Sec 2): Tests for pigments and other solids, Section 2 Pigments and nonvolatile matter.

(Part 8/Sec 3): Tests for pigments and other solids, Section 3 Ash content.

(Part 8/Sec 4): Tests for pigments and other solids, Section 4 Phthalic anhydride.

(Part 8/Sec 5): Tests for pigments and other solids, Section 5 Lead restriction test.

(Part 8/Sec 6): Tests for pigments and other solids, Section 6 Volume solids.

(Part 9/Sec 1): Tests for lacquers and varnish, Section 1 Acid value.

(Part 9/Sec 2): Tests for lacquers and varnish, Section 2 Rosin test.

IS 104	Specification for ready mixed paint, brushing, zinc chrome, priming.
IS 109	Specification for ready mixed paint, brushing, priming, plaster to Indian Standard colors No. 361 and 631.
IS 123	Specification for ready mixed paint, brushing, finishing, semi-gloss, for general purposes, to Indian Standard colors No. 445, 446, 448, 449, 451 and 473; and red oxide (color unspecified).
IS 133	Specification for enamel, interior (a) undercoating, (b) finishing.
IS 137	Specification for ready mixed paint, brushing, matt or egg-shell flat, finishing, interior, to Indian Standard color, as required.
IS 158	Specification for ready mixed paint, brushing, bituminous, black, lead-free, acid, alkali, and heat resisting.
IS 168	Specification for ready mixed paint, air-drying semi-glossy/matt, for general purposes.
IS 341	Specification for black Japan, Types A, B and C.
IS 2074	Specification for ready mixed paint, air drying red oxide-zinc chrome, priming.
IS 2075	Specification for ready mixed paint, stoving, red oxide-zinc chrome, priming.

IS 2339	Specification for aluminum paint for general purposes, in dual container.
IS 2932	Specification for enamel, synthetic, exterior, (a) undercoating, (b) finishing.
IS 2933	Specification for enamel, exterior, (a) undercoating, (b) finishing.
IS 3536	Specification for ready mixed ‘paint, brushing, wood primer.
IS 3537	Specification for ready mixed paint, finishing, interior for general purposes, to Indian Standard colors No. 101, 216, 217, 219, 275, 281, 352, 353, 358 to 361, 363, 364, 388, 410, 442, 444, 628, 631, 632, 634, 693, 697, white and black.
IS 3539	Specification for ready mixed paint, undercoating, for use under oil finishes, to Indian Standard colors, as required.
IS 3585	Specification for ready mixed paint, aluminum, brushing, priming, water resistant, for wood work.
IS 3678	Specification for ready mixed paint, thick white, for lettering.
IS 8662	Specification for enamel, synthetic, exterior, (a) undercoating, (b) finishing, for railway coaches.
IS 9862	Specification for ready mixed paint, brushing, bituminous black lead free, acid, alkali, water and chlorine resisting.
IS 11883	Specification for ready mixed paint, brushing, red oxide, priming for metals.
IS 13183	Specification for aluminum paints, heat resistant.
IS 13213	Specification for polyurethane full gloss enamel (two pack).
IS 13607	Specification for ready mixed paint, finishing, general purposes, synthetic.
IS 13871	Specification for powder coatings.

2.14.3 Thinners and Solvents

These shall conform to the following Standards:

IS 324	Specification for ordinary denatured spirit.
IS 82	Methods of sampling and test for thinners and solvents for paints.

IS 324	Specification for ordinary denatured spirit.
IS 533	Specification for gum spirit of turpentine (oil of turpentine).
IS 14314	Specification for thinner general purposes for synthetic paints and varnishes.

2.14.4 Varnishes and Lacquers

These materials shall conform to the following Standards:

BDS 401	Specification for varnish, finishing, exterior, type-I, (synthetic).
BDS 1064	Specification for varnish, stoving.
BDS 1065	Specification for varnish, acid resisting.
BDS 1066	Specification for varnish, finishing, interior.
IS 337	Specification for varnish, finishing, interior.
IS 347	Specification for varnish, shellac for general purposes.
IS 348	Specification for french polish.
IS 524	Specification for varnish, finishing, exterior, synthetic.
IS 525	Specification for varnish, finishing, exterior and general purposes.
IS 642	Specification for varnish medium for aluminum paint.

2.15 Sanitary Appliances and Water Fittings

2.15.1 Sanitary Appliances

Sanitary appliances shall conform to the following Standards:

ASHRAE 90A	Energy conservation in new building design.
ASHRAE 90B	Energy Conservation in New Building Design.
AWWA C700	Cold-Water Meters-Displacement type, bronze main case.
AWWA C701	Cold-Water Meters-Turbine type, for customer service.
AWWA C702	Cold-Water Meters-Compound type.
BDS 1162	Ceramic wash basin and pedestal, ceramic wash basin and pedestals dimension, design & construction, type, permissible deviation

BDS 1163	Specification for Vitreous Sanitary Appliances,
	Part-1, General requirements.
	Part-2, Specific requirements for water closets;
	Part-3, Specification requirements for urinal (bowl type).
	Part-4, Specific requirements for foot rest.
	Part-5, Specific requirements for integrated squatting pans.
BDS 1361	Faucets.
BDS 1593	Plastic sanitary squatting pan.
BS 1125	Specification for WC flushing cisterns (including dual flash cisterns and flush pipes).
BS 1244	Metal Sink for domestic purposes.
BS 1254	Specification for C seats (plastics).
BS 1329	Specification for metal hand rinse basins.
BS 1876	Specification for automatic flushing cistern for urinals.

2.15.2 Pipes and Pipe Fittings for Water Supply and Sanitation

Pipes and pipe fittings for water supply and sanitation shall comply with the following Standards :

BDS 1111	Centrifugally cast (spun) iron pressure pipes for water, gas and sewage.
BDS 1356	Specification for ferrules for water services.
BDS 1357	Specification for washers with fittings for water service.
BDS 1361	Faucets.
	This standard specifies the technical requirements of various types of Faucets.
BDS 1562	Solvent cements for polyvinylchloride (PVC) plastic pipe and fitting.

BDS 1593	Plastic sanitary squatting pan. Pan lays down the requirement for material, dimension physical requirements and testing for power flush type injection molded high density polyethylene (HDPE) or polypropylene (PP) squatting pan.
BDS EN 1254-2	Copper and copper alloys - Plumbing fittings-Part 2: Fittings with compression ends for use with copper tubes.
BDS EN 1717	Protection against pollution of potable water in water installations and general requirements of devices to prevent pollution by backflow.
BDS EN 14506	Devices to prevent pollution by backflow of potable water-Automatic diverter-Family H, type C.
BDS ISO 3419	Non-alloy and alloy steel butt-welding fittings.
BDS ISO 5251	Stainless steel butt-welding fittings.
BDS ISO 6761	Steel tubes-Preparation of ends of tubes and fittings for welding.
BDS ISO 3822-1	Acoustics: Laboratory tests on noise emission from appliances and equipment used in water supply installations-Part 1: Method of measurement.
BDS ISO 3822 -2	Acoustics: Laboratory tests on noise emission from appliances and equipment used in water supply installations-Part 2: Mounting and operating conditions for draw-off taps and mixing valves.
BDS ISO 3822 -4	Acoustics: Laboratory tests on noise emission from appliances and equipment used in water supply installations-Part 4: Mounting and operating conditions for special appliances.
BDS ISO 161-1	Thermoplastics pipes for the conveyance of fluids Nominal outside diameters and Nominal Pressures- Part 1: Metric series.

BDS ISO 161-2	Thermoplastics pipes for the conveyance of fluids- Nominal outside diameters and Nominal Pressures- Part 2: Inch-based series.
BDS ISO 265-1	Pipes and fittings of plastics materials- fittings for domestic and industrial waste pipes- Basic dimensions: Metric series- Part 1: Un-plasticized Poly (Vinyl chloride) (PVC-U).
BDS ISO 1167-1	Thermoplastics pipes fittings and assemblies for the conveyance of fluids-Determination of the resistance to internal pressure- Part 1: General method.
BDS ISO 1167-2	Thermoplastics pipes fittings and assemblies for the conveyance of fluids- Determination of the resistance to internal pressure- Part 2: Preparation of pipe test pieces.
BDS ISO 1746	Rubber or Plastics hoses and tubing-bending tests.
BDS ISO 2505	Thermoplastics pipes- Longitudinal reversion-Test method and parameters.
BDS ISO 2507-2	Thermoplastics pipes and fittings-Vista softening temperature- Part 2: Test conditions for Un-plasticized polyvinylchloride (PVC-U) or chlorinated polyvinylchloride (PVC-C) pipes and fittings and for high impact resistance polyvinylchloride (PVC-HI) pipes.
BDS ISO 3114	Unplasticized polyvinylchloride (PVC) pipes for potable water supply-Extractability of lead and tin- Test method.
BDS ISO 3126	Plastics piping systems-Plastics components- Determination of dimensions.
BDS ISO 3127	Thermoplastics pipes-Determination of resistance to external blows-round-the-clock method.
BDS ISO 3501	Assembled joints between fittings and polyethylene (PE) pressure pipes-Test of resistance to pull-out.
BDS ISO 3503	Assembled joints between fittings and polyethylene (PE) pressure pipes-Test of leak proofness under internal pressure when subjected to bending.

BDS ISO 3633	Plastics piping systems for soil and waste discharge (low and high temperature) inside buildings-Specifications.
BDS ISO 6964	Polyolefin pipes and fittings-Determination of carbon black content by calcinations and pyrolysis-Test method and basic specification.
BDS ISO 4065	Thermoplastics pipes- Universal wall thickness table.
BDS ISO /TR 4191	Unplasticized polyvinylchloride (PVC-U) pipes for water supply-Recommended practice for laying.
BDS ISO 4422-1	Pipes and fittings made of unplasticized polyvinylchloride (PVC-U) for water supply-Specifications-Part 1: General.
BDS ISO 4422-2	Pipes and fittings made of unplasticized polyvinylchloride (PVC-U) for water supply-Specifications-Part 2: Pipes (with or without integral sockets).
BDS ISO 4422-3	Pipes and fittings made of unplasticized polyvinylchloride (PVC-U) for water supply- Specifications-Part 3: Fittings and joints.
BDS ISO 4422-4	Pipes and fittings made of unplasticized polyvinylchloride (PVC-U) for water supply- Specifications- Part 4: Valves and ancillary equipment.
BDS ISO 4422-5	Pipes and fittings made of unplasticized polyvinylchloride (PVC-U) for water supply- Specifications- Part 5: Fitness for purpose of the system.
BDS ISO 4433-3	Thermoplastics pipes- Resistance to liquid chemicals-Classification-Part 3: Unplasticized polyvinylchloride (PVC-U), high- impact polyvinylchloride (PVC-HI) and chlorinated polyvinylchloride (PVC -C) pipes.
BDS ISO 4435	Plastic piping systems for non- pressure underground drainage and sewerage- Unplasticized polyvinylchloride (PVC-U).
BDS ISO 4439	Unplasticized polyvinylchloride (PVC) pipes and fittings-Determination and specification of density.
BDS ISO 6259-1	Thermoplastics pipes-Determination of tensile properties-Part 1: General test method.

BDS ISO 6259-2	Thermoplastics pipes-Determination of tensile properties- Part 2: Pipes made of unplasticized polyvinylchloride (PVC-U), Chlorinated polyvinylchloride (PVC-C), and high-impact polyvinylchloride (PVC-HI).
BDS ISO 6992	Unplasticized polyvinylchloride (PVC-U) pipes for drinking water supply-Extractability of cadmium and mercury occurring as impurities.
BDS ISO 9624	Thermoplastics pipes for fluids under pressure-Mating dimensions of flange adapters and loose backing flanges.
BDS ISO 11413	Plastics pipes and fittings-Preparation of test piece assemblies between a polyethylene (PE) pipe and an electro fusion fitting BDS ISO 11414, Plastics pipes and fittings-Preparation of polyethylene (PE) pipe/pipe or pipe/fitting test piece assemblies by butt fusion.
BDS ISO 12176-2	Plastics pipes and fittings-Equipment for fusion jointing polyethylene systems-Part 2: Electro fusion
BDS ISO 12176-3	Plastics pipes and fittings-Equipment for fusion jointing polyethylene systems-Part 3: Operator's badge.
BDS ISO 12176-4	Plastics pipes and fittings-Equipment for fusion jointing polyethylene systems-Part 4: Traceability coding.
BDS ISO 13479	Polyolefin pipes for the conveyance of fluids-Determination of resistance to crack propagation-Test method for slow crack growth on notched pipes (notch test).
BDS ISO 13761	Plastics pipes and fittings-Pressure reduction factors for polyethylene pipeline systems for use at temperatures above 20°C.
BDS ISO 13951	Plastics piping systems-Test method for the resistance of polyolefin pipe/pipe or pipe/fitting assemblies to tensile loading.
BDS ISO 13953	Polyethylene (PE) pipes and fittings-Determination of the tensile strength and failure mode of test pieces from a butt-fused joint.
BDS ISO 13954	Plastics pipes and fittings-Peel de-cohesion test for polyethylene

	(PE) electro fusion assemblies of nominal outside diameter greater than or equal to 90 mm.
BDS ISO 13955	Plastics pipes and fittings-Crushing de-cohesion test for polyethylene (PE) electro fusion assemblies.
BDS ISO 13957	Plastics pipes and fittings-Polyethylene (PE) tapping tees-Test method for impact resistance.
BDS ISO 14236	Plastics pipes and fittings-Mechanical-joint compression fittings for use with polyethylene pressure pipes in water supply systems.
BDS ISO 18553	Method for the assessment of the degree of pigment or carbon black dispersion in polyolefin pipes, fittings and compounds.
BDS ISO 18553	Method for the assessment of the degree of pigment or carbon black dispersion in polyolefin pipes, fittings and compounds Amendment 1:2010.
BDS ISO 4427-1	Plastics piping systems-Polyethylene (PE) pipes and fittings for water supply-Part 1: General.
BDS ISO 4427-2	Plastics piping systems-Polyethylene (PE) pipes and fittings for water supply-Part 2: Pipes.
BDS ISO 4427-3	Plastics piping systems-Polyethylene (PE) pipes and fittings for water supply-Part 3: Fittings.
BDS ISO 4427-5	Plastics piping systems-Polyethylene (PE) pipes and fittings for water supply-Part 5: Fitness for purpose of the system.
BDS ISO 4427-1	Plastics piping systems-Polyethylene (PE) pipes and fittings for water supply-Part 1: General Technical corrigendum 1: 2010.
BDS ISO 3458	Assembled joints between fittings and polyethylene (PE) pressure pipes-Test of leak proofness under internal pressure.
BDS ISO 3459	Polyethylene (PE) pressure pipes-Joints assembled with mechanical fittings-Internal under pressure test method and requirement.

ASTM A53/A53M	Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless.
ASTM A74	Standard Specification for Cast Iron Soil Pipe and Fittings.
ASTM A377	Standard Index of Specifications for Ductile-Iron Pressure Pipe.
ASTM B42	Standard Specification for Seamless Copper Pipe, Standard Sizes.
ASTM B43	Standard Specification for Seamless Red Brass Pipe, Standard Sizes.
ASTM B75	Standard Specification for Seamless Copper Tube.
ASTM B88	Standard Specification for Seamless Copper Water Tube.
ASTM B251	Standard Specification for General Requirements for Wrought Seamless Copper and Copper-Alloy Tube.
ASTM B302	Standard Specification for Thread-less Copper Pipe, Standard Sizes.
ASTM B306	Standard Specification for Copper Drainage Tube (DWV).
ASTM B429/B429M	Standard Specification for Aluminum-Alloy Extruded Structural Pipe and Tube.
ASTM B447	Standard Specification for Welded Copper Tube.
ASTM B745/B745M	Standard Specification for Corrugated Aluminum Pipe for Sewers and Drains.
ASTM C14	Standard Specification for Non-reinforced Concrete Sewer, Storm Drain, and Culvert Pipe.
ASTM C76	Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe.
ASTM C654	Standard Specification for Porous Concrete Pipe.
ASTM C700	Standard Specification for Vitrified Clay Pipe, Extra Strength, Standard Strength, and Perforated.
ASTM D1527	Standard Specification for Acrylonitrile-Butadiene-Styrene (ABS) Plastic Pipe, Schedules 40 and 80.

ASTM D1785	Standard Specification for Poly(Vinyl Chloride) (PVC) Plastic Pipe, Schedules 40, 80, and 120.
ASTM D2239	Standard Specification for Polyethylene (PE) Plastic Pipe (SIDR-PR) Based on Controlled Inside Diameter.
ASTM D2241	Standard Specification for Poly(Vinyl Chloride) (PVC) Pressure-Rated Pipe (SDR Series).
ASTM D2321	Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications ASTM D2464 Standard Specification for Threaded Polyvinyl Chloride (PVC) Plastic Pipe Fittings, Schedule 80.
ASTM D2466	Standard Specification for Polyvinyl Chloride (PVC) Plastic Pipe Fittings, Schedule 40ASTM D2467 Standard Specification for Poly(Vinyl Chloride) (PVC) Plastic Pipe Fittings, Schedule 80.
ASTM D2609	Standard Specification for Plastic Insert Fittings for Polyethylene (PE) Plastic Pipe.
ASTM D2661	Standard Specification for Acrylonitrile-Butadiene-Styrene (ABS) Schedule 40 Plastic Drain, Waste, and Vent Pipe and Fittings.
ASTM D2665	Standard Specification for Poly(Vinyl Chloride) (PVC) Plastic Drain, Waste, and Vent Pipe and Fittings.
ASTM D2672	Standard Specification for Joints for IPS PVC Pipe Using Solvent Cement.
ASTM D2729	Standard Specification for Poly(Vinyl Chloride) (PVC) Sewer Pipe and Fittings.
ASTM D2737	Standard Specification for Polyethylene (PE) Plastic Tubing.
ASTM D2751	Standard Specification for Acrylonitrile-Butadiene-Styrene (ABS) Sewer Pipe and Fittings.
ASTM D2846/D2846M	Standard Specification for Chlorinated Poly(Vinyl Chloride) (CPVC) Plastic Hot- and Cold-Water Distribution Systems.
ASTM D2949	Standard Specification for 3.25-in. Outside Diameter Poly(Vinyl

	Chloride) (PVC) Plastic Drain, Waste, and Vent Pipe and Fittings.
ASTM D3034	Standard Specification for Type PSM Poly(Vinyl Chloride) (PVC) Sewer Pipe and Fittings.
ASTM F405	Standard Specification for Corrugated Polyethylene (PE) Pipe and Fittings.
ASTM F409	Standard Specification for Thermoplastic Accessible and Replaceable Plastic Tube and Tubular Fittings.
ASTM F437	Standard Specification for Threaded Chlorinated Poly(Vinyl Chloride) (CPVC) Plastic Pipe Fittings, Schedule 80.
ASTM F438	Standard Specification for Socket-Type Chlorinated Poly(Vinyl Chloride) (CPVC) Plastic Pipe Fittings, Schedule 40.
ASTM B209	Standard Specification for Aluminum and Aluminum-Alloy Sheet and Plate.
ASTM F441/F441M	Standard Specification for Chlorinated Poly(Vinyl Chloride) (CPVC) Plastic Pipe, Schedules 40 and 80.
ASTM F442/F442M	Standard Specification for Chlorinated Poly(Vinyl Chloride) (CPVC) Plastic Pipe (SDR-PR).
ASTM F628	Standard Specification for Acrylonitrile-Butadiene-Styrene (ABS) Schedule 40 Plastic Drain, Waste, and Vent Pipe With a Cellular Core.
ASTM F891	Standard Specification for Coextruded Poly(Vinyl Chloride) (PVC) Plastic Pipe With a Cellular Core.
IS 404 (Part-I)	Specification for lead pipes Part I for other than chemical purpose.
ISO 2531	Ductile Iron pipes, fittings and accessories for pressure pipelines.
ASME/ANSI B16.3	Malleable iron threaded fittings: Classes 150 and 300.
ASME/ANSI	Cast Iron threaded fittings.

B16.485	
ASME/ANSI B16.9	Factory made wrought steel butt welding fittings.
ASME/ANSI B16.11	Forged Steel Fittings, Socket-Welding and Threaded.
ASME/ABSI B16.12	Cast-Iron Threaded Drainage Fittings;
ASME/ANSI B16.15	Cast Copper Alloy Threaded Fittings: Classes 125 and 250.
ASME/ANSI B16.18	Cast Copper Alloy Solder Joint Pressure Fittings.
ASME/ANSI B16.22	Wrought Copper and Copper Alloy Solder Joint Pressure Fittings.
ASME/ANSI B16.23	Cast Copper Alloy Solder Joint Drainage Fittings (DWV).
ASME/ANSI B16.28	Wrought Steel Butt welding Short radius Elbows and Returns.
ASME/ANSI B16.29	Wrought Copper and Wrought Copper Alloy Solder Joint Fittings for solvent Drainage Systems.
ASME/ANSI B16.32	Cast Copper Alloy Solder Joint Fittings for Solvent Drainage Systems.
AWWA C110	Standard for Grey Iron and Ductile Iron Fittings, 76 mm to 1220 mm (3 in. through 48 inches), for Water and Other Liquids.

2.15.3 Joints and Connections Between Pipes and Fittings

Applicable standards for joints and connections between pipes and fittings are listed below:

BDS EN 681-1	Elastomeric seals-Materials requirements for pipe joint seals used in water and drainage applications-Part 1: Vulcanized rubber.
BDS EN 681-2	Elastomeric seals-Materials requirements for pipe joint seals used

	in water and drainage applications-Part 2: Thermoplastic elastomers.
ASTM B42	Standard Specification for Seamless Copper Pipe, Standard Sizes.
ASTM C425	Standard Specification for Compression Joints for Vitrified Clay Pipe and Fittings.
ASTM C443	Standard Specification for Joints for Concrete Pipe and Manholes, Using Rubber Gaskets.
ASTM C564	Standard Specification for Rubber Gaskets for Cast Iron Soil Pipe and Fittings.
ASTM D2235	Standard Specification for Solvent Cement for Acrylonitrile-Butadiene-Styrene (ABS) Plastic Pipe and Fittings.
ASTM D2564	Standard Specification for Solvent Cements for Poly(Vinyl Chloride) (PVC) Plastic Piping Systems.
ASTM D2657	Standard Practice for Heat Fusion Joining of Polyolefin Pipe and Fittings.
ASTM D2661	Standard Specification for Acrylonitrile-Butadiene-Styrene (ABS) Schedule 40 Plastic Drain, Waste, and Vent Pipe and Fittings.
ASTM D2846/D2846M	Standard Specification for Chlorinated Poly(Vinyl Chloride) (CPVC) Plastic Hot- and Cold-Water Distribution Systems.
ASTM D2855	Standard Practice for Making Solvent-Cemented Joints with Poly(Vinyl Chloride) (PVC) Pipe and Fittings.
ASTM D3139	Standard Specification for Joints for Plastic Pressure Pipes Using Flexible Elastomeric Seals.
ASTM D3212	Standard Specification for Joints for Drain and Sewer Plastic Pipes Using Flexible Elastomeric Seals.
ASTM F402	Standard Practice for Safe Handling of Solvent Cements, Primers, and Cleaners Used for Joining Thermoplastic Pipe and Fittings.
ASTM F493	Standard Specification for Solvent Cements for Chlorinated Poly(Vinyl Chloride) (CPVC) Plastic Pipe and Fittings.

ASTM F628	Standard Specification for Acrylonitrile-Butadiene-Styrene (ABS) Schedule 40 Plastic Drain, Waste, and Vent Pipe With a Cellular Core.
ASTM F656	Standard Specification for Primers for Use in Solvent Cement Joints of Poly(Vinyl Chloride) (PVC) Plastic Pipe and Fittings.
ASME/ANSI B1.20.1	Pillar taps used in water supply.

2.15.4 Taps and Valves

Taps and valves shall conform to the following Standards:

BDS 987	Sand cast brass screw-down bib taps and stop taps for water services. It covers the requirements regarding materials, dimensions, constructions, workmanship, finish and testing of tapes for water services.
BDS 1507	Bib taps used in water supply.
BDS 1508	Stop taps used in water supply specifies the requirements, dimensions construction, materials and test methods of stop taps used in water supply.
BDS 1509	Pillar taps used in water supply.
BDS EN 200	Sanitary tapware-Single taps and combination taps for water supply systems of type 1 and type 2-General technical specification.
BDS EN 246	Sanitary tapware-General specifications for flow rate regulators.
BDS EN 248	Sanitary tapware-General specification for electrodeposited coatings of Ni-Cr.
BDS EN 1112	Sanitary tapware - Shower outlets for sanitary tapware for water supply systems of type 1 and type 2-General technical specification.

BDS EN 1113	Sanitary tapware-Shower hoses for sanitary tapware for water supply systems of type 1 and type 2-General technical specification.
BS 1212 (3 Parts)	Specification for Float Operated Valves (excluding floats).
BS 1010	Specification for draw-off taps and stop valves for water services.
BS 1968	Specification for floats for ball valves (copper).
BS 5433	Specification for underground stop valves for water services (copper).
BS 2456	Specification for floats for ball valves (plastic) for cold and hot water.
BS 1415 (2 parts)	Mixing valves (manually operated).
BS 5163	Specification for predominantly key-operated cast iron wedge gate valve for water works.
BS 3377	Specification for boilers for use with domestic solid mineral fuel appliances.
BS 843	Specification for thermal storage electric water heaters.
BS 855	Specification for welded steel boilers for central heating and indirect hot water supply.

2.16 Miscellaneous Materials

2.16.1 Ferrocement

Details including material requirements are given in Chapter 12 Part 6.

2.16.2 Plastics

Plastics may be used in buildings or structures as light transmitting materials such as glazing, skylights, lighting lenses, luminous ceilings, roof panels, signs and similar purposes. Foam plastics are also used in buildings.

Applicants for approval of a plastic material shall furnish all necessary technical data required by the Building Official. The data shall include chemical composition; applicable physical, mechanical and thermal properties such as fire resistance, flammability and flame spread; weather resistance; electrical properties; products of combustion; and coefficient of expansion.

The requirements for light transmitting plastics, including roof panels and foam plastics are given below.

2.16.2.1 Light Transmitting Plastics

An approved light transmitting plastic shall be any thermoplastic, thermosetting or reinforced thermosetting plastic material which has a self-ignition temperature of 343ºC or greater when tested in accordance with, Test Method for Ignition Properties of Plastics; a smoke density rating not greater than 450 when tested in the manner intended for use in accordance with ASTM E84 Test Method for Surface Burning Characteristics of Building Materials; or not greater than 75 when tested in the thickness intended for use in accordance with ASTM D2843 Test Method for Density of Smoke from the Burning or Decomposition of Plastics; and which conforms to one of the following combustibility classifications:

Class C1 : Plastic materials which have a burning extent of 25 mm or less when tested at a nominal thickness of 1.5 mm, or in the thickness intended for use, in accordance with ASTM D635 Test Method for Rate of Burning and/or Extent and Time of Burning of Self-Supporting Plastics in Horizontal Position; or

Class C2: Plastic materials which have a burning rate of 63 mm/min or less when tested at a nominal thickness of 1.5 mm, or in the thickness intended for use, in accordance with ASTM D635.

2.16.2.2 Foam Plastics

All foam plastics and foam plastic cores of manufactured assemblies shall have a flame spread rating of not more than 75 and shall have a smoke developed rating of not more than 450 when tested in the maximum thickness intended for use in accordance with ASTM E84.

All foam plastics, unless otherwise indicated in this Section, shall be separated from the interior of a building by an approved thermal barrier of 13 mm gypsum wall board or equivalent thermal barrier material which will limit the average temperature rise of the unexposed surface to not more than 121ºC after 15 minutes of fire exposure complying with the standard time-temperature curve of ASTM E119 Test Methods for Fire Tests of Building Construction and Materials. The thermal barrier shall be installed in such a manner that it will stay in place for a minimum of 15 minutes under the same testing conditions. The thermal barrier is not required when the foam plastic is protected by a 25 mm minimum thickness of masonry or concrete.

2.16.2.3 Applicable Standards

A list of applicable Standards for plastics is given below:

BDS 885	Method for measuring viscosity number and K-value of PVC resins.
BDS 886	Method for direct measuring the specific gravity of plastics.
BDS 887	Method for measuring deformation under heat of flexible rigid PVC compounds.
BDS 888	Method for measuring temperature of deflection under load.
BDS 889	Method for measuring the Vicat Softening Temperature (VST) of thermoplastics.
BDS 890	Method for measuring the water absorption at room temperature and boiling water absorption of plastics.
BDS 891	Method for measuring the flexural modulus of plastics.
BDS 892	Method for measuring the resistance to tear propagation of flexible plastics, film or sheeting.
ASTM D543	Standard Practices for Evaluating the Resistance of Plastics to Chemical Reagents.
ASTM D635	Standard Test Method for Rate of Burning and/or Extent and Time of Burning of Plastics in a Horizontal Position.
ASTM D638	Standard Test Method for Tensile Properties of Plastics.
ASTM D695	Standard Test Method for Compressive Properties of Rigid Plastics.
ASTM D882	Standard Test Method for Tensile Properties of Thin Plastic Sheeting.
ASTM D1003	Standard Test Method for Haze and Luminous Transmittance of Transparent Plastics.
ASTM D1044	Standard Test Method for Resistance of Transparent Plastics to Surface Abrasion.
ASTM D1204	Standard Test Method for Linear Dimensional Changes of Non-rigid

	Thermoplastic Sheeting or Film at Elevated Temperature.
ASTM D1593	Standard Specification for Non-rigid Vinyl Chloride Plastic Film and Sheeting.
ASTM D2103	Standard Specification for Polyethylene Film and Sheeting.
ASTM D2126	Standard Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging.
ASTM D2842	Standard Test Method for Water Absorption of Rigid Cellular Plastics.
ASTM D2843	Standard Test Method for Density of Smoke from the Burning or Decomposition of Plastics.
ASTM D3294	Standard Specification for PTFE Resin Molded Sheet and Molded Basic Shapes.
ASTM D3678	Standard Specification for Rigid Poly(Vinyl Chloride) (PVC) Interior-Profile Extrusions.
ASTM D3679	Standard Specification for Rigid Poly(Vinyl Chloride) (PVC) Siding.
ASTM D3841	Standard Specification for Glass-Fiber-Reinforced Polyester Plastic Panels.
ASTM D4802	Standard Specification for Poly(Methyl Methacrylate) Acrylic Plastic Sheet.
ASTM E84	Standard Test Method for Surface Burning Characteristics of Building Materials.
ASTM E119	Standard Test Methods for Fire Tests of Building Construction and Materials.

2.16.3 Ballies and Wood Poles

Ballies of Sal/Gazari, Sundari and Garjan are used in building construction. These shall be free from rots, knots and sap, and straight and uniform in size. These should conform to the following Standards:

BDS 809	Specification for wood poles for overhead power and telecommunication lines.
ASTM D25	Standard Specification for Round Timber Piles.
IS 3337	Specification for Ballies for general purposes.
IS 1900	Method of testing wood poles.
IS 6711	Code of practice for maintenance of wood poles for overhead power and telecommunications lines.

2.16.4 Bamboos

The following standards shall be applicable for bamboos used for structural and nonstructural purposes:

IS 1902	Code of Practice for Preservation of Bamboo and Cane for Non-structural Purposes;
IS 6874	Method of Tests for Round Bamboos.
IS 8242	Methods of Tests for Split Bamboo.
IS 8295	Specification for Bamboo Chicks, Part I Fine.
IS 9096	Code of Practice for Preservation of Bamboos for Structural Purposes.

2.16.5 Fillers, Stoppers and Putties

These shall conform to the following standards:

IS 110	Specification for ready mixed paint, brushing, grey filler, for enamels, for use over primers.
IS 345	Specification for wood filler, transparent, liquid.
IS 419	Specification for putty for use on window frames.
IS 421	Specification for jointing paste, for bedding moldings on coaching stock.
IS 423	Specification for plastic wood, for joiners' filler.
IS 424	Specification for plastic asphalt.
IS 3709	Specification for mastic cement for bedding of metal windows.

IS 7164	Specification for Stopper.

2.16.6 Wire Ropes and Wire Products

These materials shall conform to the following standards:

ASTM A116	Standard Specification for Zinc-Coated (Galvanized) Steel Woven Wire Fence Fabric.
ASTM A121	Standard Specification for Metallic-Coated Carbon Steel Barbed Wire.
ASTM A368	Standard Specification for Stainless Steel Wire Strand.
ASTM A392	Standard Specification for Zinc-Coated Steel Chain-Link Fence Fabric.
ASTM A475	Standard Specification for Zinc-Coated Steel Wire Strand.
ASTM A492	Standard Specification for Stainless Steel Rope Wire.
ASTM A510	Standard Specification for General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel.
ASTM A586	Standard Specification for Zinc-Coated Parallel and Helical Steel Wire Structural Strand.
ASTM A603	Standard Specification for Zinc-Coated Steel Structural Wire Rope.
ASTM A817	Standard Specification for Metallic-Coated Steel Wire for Chain-Link Fence Fabric. and Marcelled Tension Wire.
ASTM A824	Standard Specification for Metallic-Coated Steel Marcelled Tension Wire for Use With Chain Link Fence.
ASTM F1183	Standard Specification for Aluminum Alloy Chain Link Fence Fabric.
IS 2365	Specification for Steel Wire Suspension Ropes for Lifts, Elevators and Hoists.

2.16.7 Waterproofing and Damp-proofing Materials

Waterproofing and damp-proofing materials shall conform to the following standards:

ASTM D41	Standard Specification for Asphalt Primer Used in Roofing, Damp proofing, and Waterproofing.
ASTM D43	Standard Specification for Coal Tar Primer Used in Roofing, Damp proofing, and Waterproofing.
ASTM D146	Standard Test Methods for Sampling and Testing Bitumen-Saturated Felts and Woven Fabrics for Roofing and Waterproofing.
ASTM D173	Standard Specification for Bitumen-Saturated Cotton Fabrics Used in Roofing and Waterproofing.
ASTM D6380	Standard Specification for Asphalt Roll Roofing (Organic Felt).
ASTM D226/D226M	Standard Specification for Asphalt-Saturated Organic Felt Used in Roofing and Waterproofing.
ASTM D227	Standard Specification for Coal-Tar-Saturated Organic Felt Used in Roofing and Waterproofing.
ASTM D449	Standard Specification for Asphalt Used in Damp proofing and Waterproofing.
ASTM D450	Standard Specification for Coal-Tar Pitch Used in Roofing, Damp proofing, and Waterproofing.
ASTM D1327	Standard Specification for Bitumen-Saturated Woven Burlap Fabrics Used in Roofing and Waterproofing.
ASTM D1668	Standard Specification for Glass Fabrics (Woven and Treated) for Roofing and Waterproofing.
ASTM D2178	Standard Specification for Asphalt Glass Felt Used in Roofing and Waterproofing.
ASTM D2626	Standard Specification for Asphalt-Saturated and Coated Organic Felt Base Sheet Used in Roofing.
ASTM D3468	Standard Specification for Liquid-Applied Neoprene and Chloro-sulfonated Polyethylene Used in Roofing and Waterproofing.

2.16.8 Glazed Tiles and Tile-setting Mortars

Glazed tiles shall conform to the following standards:

BDS 1301	Specification for glazed earthenware wall tiles.
ASTM C126	Standard Specification for Ceramic Glazed Structural Clay Facing Tile, Facing Brick, and Solid Masonry Units.
ANSI A137.1	Specification for Ceramic Tile.
BS 6431	Ceramic floor and wall tiles (Part 1 to 23).

2.16.8.1 Mortars for Ceramic Wall and Floor Tile

(a) Portland Cement Mortars: Portland cement mortars for installing ceramic wall and floor tile shall comply with ANSI A108.1 and be of the compositions indicated in Table 5.2.1.

(b) Dry-set Portland Cement Mortars: Premixed prepared Portland cement mortars, which require only the addition of water and which are used in the installation of ceramic tile, shall comply with ANSI A 118.1. The shear bond strength for tile set in such mortar shall be as required in accordance with that standard. Tile set in dry-set Portland cement mortar shall be installed in accordance with ANSI A 108.5.

(c) Electrically Conductive Dry-Set Mortars: Premixed prepared Portland cement mortars, which require only the addition of water and which comply with ANSI A118.2, shall be used in the installation of electrically conductive ceramic tile. Tile set in electrically conductive dry-set mortar shall be installed in accordance with ANSI A 108.7.

(d) Latex-modified Portland Cement Mortars: Latex-modified Portland cement thin set mortars in which Lalex is added to dry-set mortar as a replacement for all or part of the gauging water which are used for the installation of ceramic tile shall comply with ANSI A 118.4. Tile set in latex-modified Portland cement mortar shall be installed in accordance with ANSI A 108.5.

(e) Epoxy Mortar: Chemical-resistant epoxy for setting and grouting ceramic tile shall comply with ANSI A 118.3-2009. Tile set and grouted with epoxy shall be installed in accordance with ANSI A 108.6.

(f) Furan Mortar and Grout: Chemical resistant furan mortar and grout which are used to install ceramic tile shall comply with ANSI A 118.5. Tile set and grouted with furan shall be installed in accordance with ANSI A 108.8.

(g) Modified Epoxy-Emulsion Mortar and Grout: Modified epoxy-emulsion mortar and grout which are used to install ceramic tile shall comply with

ANSI A 118.8. Tile set and grouted with modified epoxy-emulsion mortar and grout shall be installed in accordance with ANSI A 108.9.

(h) Organic Adhesives: Water-resistant organic adhesives used for the installation of ceramic tile shall comply with ANSI A 136.1. The shear bond strength after water immersion shall not be less than 0.25 kN/mm^2 for Type I adhesive, and not less than 0.13 kN/mm^2 for Type II adhesive when tested in accordance with ANSI A 136.1. Tile set in organic adhesive shall be installed in accordance with ANSI A 108.4.

(i) Portland Cement Grouts: Portland cement grouts used for the installation of ceramic tile shall comply with ANSI A 118.6. Portland cement grouts for tile work shall be installed in accordance with ANSI A 108.10.

2.16.8.2 Applicable Standards

A list of applicable Standards for tiles, mortars and adhesives is given below:

BDS 1301	Specification for Glazed Earthenware Wall Tiles.
ASTM C126	Standard Specification for Ceramic Glazed Structural Clay Facing Tile, Facing Brick, and Solid Masonry Units.
ANSI A108.1	Specification for the Installation of Ceramic Tile with Portland Cement Mortar.
ANSI A108.4	Installation of Ceramic Tile with Organic Adhesives or Water Cleanable Tile Setting Epoxy Adhesive.
ANSI A108.5	Installation of Ceramic Tile with Dry-Set Portland Cement Mortar or Latex-Portland Cement Mortar.
ANSI A108.6	Installation of Ceramic Tile with Chemical Resistant, Water Cleanable Tile Setting and Grouting Epoxy.
ANSI A108.7	Specification for Electrically Conductive Ceramic Tile Installed with Conductive Dry-Set Portland Cement Mortar.
ANSI A108.8	Installation of Ceramic Tile with Chemical Resistant Furan Mortar and Grout.
ANSI A108.9	Installation of Ceramic Tile with Modified Epoxy Emulsion Mortar/Grout.
ANSI A108.10	Installation of Grout in Tile work.

ANSI A118.1	Specification for Dry-Set Portland Cement Mortar.
ANSI A118.2	Specifications for Conductive Dry-set Portland Cement Mortar.
ANSI A118.3	Specifications for Chemical Resistant Water Cleanable Tile Setting and Grouting Epoxy and Water Cleanable Tile Setting Epoxy Adhesive.
ANSI A118.4	Specifications Furan Latex-Portland Cement Mortar.
ANSI A118.5	Specifications for Chemical Resistant Furan.
ANSI A118.6	Specifications for Ceramic Tile Grouts.
ANSI A118.8	Specifications for Modified Epoxy Emulsion Mortar/Grout.
ANSI A136.1	Organic Adhesives for Installation of Ceramic Tile.
ANSI A137.1	Specifications for Ceramic Tile.
BS 6431	Floor and wall tiles.
BS 6431 Part 1	Specification for classification and making, including definitions and characteristics.
BS 6431 Part 2	Specification for struded ceramic tiles with low water absorption (E< 3%) Group A1.
BS 6431 Part 3	Extruded ceramic tiles with a water absorption of 3% < 6%. Group A 11a.
BS 6431 Part 3 Sec 3.1	Specification for general products.
BS 6431 Part 3 Sec 3.2	Specification for products Terre Cuite, Cotto, Baldosion Catalan.
BS 6431 Part 4	Extruded Ceramic Tiles with a Water Absorption of 6% <E< 10%. Group A11b.
BS 6431 Part 4	Specification for General Products.
BS 6431 Part 4 Sec 4.2	Specification for Specific Products (Terre Cuite, Cotto, Baldosion Catalan).

BS 6431 Part 5	Specification for extruded ceramic tiles with a water absorption of E>10%, Group A111.
BS 6431 Part 6	Specification for dust-pre-stressed ceramic tiles with a low-water absorption (E<3%) Group B1.
BS 6431 Part 7	Specification for dust-pre-stressed ceramic tiles with a water absorption of 3% <E ≤6% Group B11a;
BS 6431 Part 8	Specification for dust-pre-stressed ceramic tiles with water absorption of 6% < E ≤10%. Group B11b;
BS 6431 Part 9	Specification for dust-pre-stressed ceramic tiles with a water absorption of E >10%. Group B111.
BS 6431 Part 10	Method for determination of dimensions and surface quality.
BS 6431 Part 11	Method for determination of water absorption.
BS 6431 Part 12	Method for determination of modulus of.
BS 6431 Part 13	Method for determination of scratch hardness of surface according to Mhos.
BS 6431 Part 14	Method for determination of resistance to abrasion of unglazed tiles.
BS 6431 Part 15	Method for determination of linear thermal expansion.
BS 6431 Part 16	Method for determination of resistance to thermal shock.
BS 6431 Part 17	Method for determination of crazing resistance-glazed tiles.
BS 6431 Part 18	Method for determination of chemical resistance-unglazed tiles.
BS 6431 Part 19	Method for determination of chemical resistance-glazed tiles.
BS 6431 Part 20	Method for determination of resistance to surface abrasion-glazed tiles.
BS 6431 Part 23	Specification for sampling and basis for acceptance.

2.16.9 Refractories

Refractories shall conform to the following Standards:

BDS 1493	Glossary of terms used in refractory.
BDS 1494	Dimension of refractory bricks.
BDS 1495	High aluminum refractory bricks.
ISO 528	Refractory products-determination of pyrometric cone equivalent (refractoriness).
ISO 1109	Refractory products-classification of dense shaped refractory products.
ISO 1146	Pyrometric reference cones for laboratory use-specification.
ISO 1893	Refractory products-determination of refractoriness-under-load (differential with rising temperature).
ISO 1927	Prepared unshaped refractory materials (dense and insulating) classification.
ISO 2245	Shaped insulating refractory products-classification.
ISO 2477	Shaped insulating refractory products-determination of permanent change in dimensions on heating.
ISO 2478	Dense shaped refractory products-determination of permanent change in dimensions on heating.
ISO 3187	Refractory products-determination of creep in compression.
ISO 5013	Refractory products-determination of modulus of rupture at elevated temperatures.
ISO 5014	Refractory products-determination of modulus of rupture at ambient temperature.
ISO 5016	Shaped insulating refractory products-determination of bulk density and true porosity.
ISO 5017	Dense shaped refractory products-determination of bulk density, apparent porosity and true porosity.
ISO 5018	Refractory materials-determination of true density.

ISO 5019-1	Refractory bricks-dimensions-Part 1: Rectangular bricks.
ISO 5019-2	Refractory bricks-dimensions-Part 2: Arch bricks;
ISO 5019-3	Refractory bricks-dimensions-Part 3: Rectangular checker bricks for regenerative furnace.
ISO 5419-4	Refractory bricks-dimensions-Part 4: Dome bricks for electric arc furnace roofs.
ISO 5015-6	Refractory bricks-dimensions-Part 6: Basic bricks for oxygen steel making converters.
ISO 5022	Shaped refractory products-sampling and acceptance testing.
ISO 5417	Refractory bricks for use in rotary kilns-dimensions.
ISO 8656	Refractory products-sampling of raw materials and unshaped products- Part 1: Sampling scheme.
ISO 8840	Refractory materials-determination of bulk density of granular materials (grain density).
ISO 8890	Dense shaped refractory products-determination of resistance to sulfuric acid.
ISO 8894-1	Refractory materials-determination of thermal conductivity- Part 1: Hot-wire method (cross-array).
ISO 8894-2	Refractory materials-determination of thermal conductivity- Part 2: Hot-wire method (parallel).
ISO 8895	Shaped Insulating refractory products-determination of cold crushing strength.
ISO 9205	Refractory bricks for use in rotary kilns-hot-face identification marking.
ISO 10080	Refractory products-classification of dense, shaped acid-resisting products.
ISO 10081	Basic refractory products-classification- Part I: Products containing less than 7% residual carbon.

2.16.10 Thermal Insulating Materials

Thermal insulation may be in the following physical forms:

- Loose fill dry granules or nodules poured or below in place;
- Flexible or semi rigid blankets and bolts of wool like material;
- Rigid boards and blocks;
- Membrane reflective insulation;
- Spray applied mineral fibre or insulating concrete;
- Poured in plain-insulating concrete;
- Foamed in place-polyurethane;
- Gypsum plaster.

Thermal insulating materials shall conform to the Standards listed below:

ASTM C167	Standard Test Methods for Thickness and Density of Blanket or Batt Thermal Insulations.
ASTM C177	Standard Test Method for Steady-State Heat Flux Measurements and Thermal Transmission Properties by Means of the Guarded-Hot-Plate Apparatus.
ASTM C195	Standard Specification for Mineral Fiber Thermal Insulating Cement.
ASTM C196	Standard Specification for Expanded or Exfoliated Vermiculite Thermal Insulating Cement
ASTM C208	Standard Specification for Cellulosic Fiber Insulating Board.
ASTM C209	Standard Test Methods for Cellulosic Fiber Insulating Board.
ASTM C1363	Standard Test Method for Thermal Performance of Building Materials and Envelope Assemblies by Means of a Hot Box Apparatus.
ASTM C240	Standard Test Methods of Testing Cellular Glass Insulation Block.
ASTM C335	Standard Test Method for Steady-State Heat Transfer Properties of Pipe Insulation.

ASTM C411	Standard Test Method for Hot-Surface Performance of High-Temperature Thermal Insulation.
ASTM C449	Standard Specification for Mineral Fiber Hydraulic-Setting Thermal Insulating and Finishing Cement.
ASTM C516	Standard Specification for Vermiculite Loose Fill Thermal Insulation.
ASTM C518	Standard Test Method for Steady-State Thermal Transmission Properties by Means of the Heat Flow Meter Apparatus.
ASTM C520	Standard Test Methods for Density of Granular Loose Fill Insulations.
ASTM C533	Standard Specification for Calcium Silicate Block and Pipe Thermal Insulation.
ASTM C534/C534M	Standard Specification for Preformed Flexible Elastomeric Cellular Thermal Insulation in Sheet and Tubular Form.
ASTM C547	Standard Specification for Mineral Fiber Pipe Insulation.
ASTM C549	Standard Specification for Perlite Loose Fill Insulation.
ASTM C552	Standard Specification for Cellular Glass Thermal Insulation.
ASTM C553	Standard Specification for Mineral Fiber Blanket Thermal Insulation for Commercial and Industrial Applications.
ASTM C578	Standard Specification for Rigid, Cellular Polystyrene Thermal Insulation.
ASTM C591	Standard Specification for Un-faced Preformed Rigid Cellular Polyisocyanurate Thermal Insulation.
ASTM C592	Standard Specification for Mineral Fiber Blanket Insulation and Blanket-Type Pipe Insulation (Metal-Mesh Covered) (Industrial Type).
ASTM C610	Standard Specification for Molded Expanded Perlite Block and Pipe Thermal Insulation.
ASTM C612	Standard Specification for Mineral Fiber Block and Board Thermal Insulation.
ASTM C665	Standard Specification for Mineral-Fiber Blanket Thermal Insulation for Light Frame Construction and Manufactured Housing.

ASTM C726	Standard Specification for Mineral Fiber Roof Insulation Board.
ASTM C728	Standard Specification for Perlite Thermal Insulation Board.
ASTM C739	Standard Specification for Cellulosic Fiber Loose-Fill Thermal Insulation.
ASTM C764	Standard Specification for Mineral Fiber Loose-Fill Thermal Insulation.
ASTM C916	Standard Specification for Adhesives for Duct Thermal Insulation.
ASTM C991	Standard Specification for Flexible Fibrous Glass Insulation for Metal Buildings.
ASTM C1014	Standard Specification for Spray-Applied Mineral Fiber Thermal and Sound Absorbing Insulation.
ASTM C1029	Standard Specification for Spray-Applied Rigid Cellular Polyurethane Thermal Insulation.
ASTM C1071	Standard Specification for Fibrous Glass Duct Lining Insulation (Thermal and Sound Absorbing Material).

2.16.11 Screw Threads and Rivets

These shall conform to the following standards:

IS 554	Dimensions for pipe threads where pressure tight joints are required on the threads.
IS 1929	Specification for hot forged steel rivets for hot closing (12 to 36 mm diameter).
IS 2155	Specification for cold-forged solid steel rivets for hot closing (6 to 16 mm diameter).
IS 2643	Dimensions for pipe threads for fastening purposes.
	Part I-Basic profile and dimensions.
	Part II-Tolerances.
	Part III-Limits of sizes.
IS 2907	Specification for non-ferrous rivets (1.6 mm to 10 mm).
IS 2998	Specification for cold forged steel rivets for cold closing (1 to 16 mm diameter).
IS 10102	Technical supply conditions for rivets.

2.16.12 Sealants

Sealants shall conform to the following Standards:

ASTM C509	Standard Specification for Elastomeric Cellular Preformed Gasket and Sealing Material.
ASTM C542	Standard Specification for Lock-Strip Gaskets.
ASTM C564	Standard Specification for Rubber Gaskets for Cast Iron Soil Pipe and Fittings.
ASTM C716	Standard Specification for Installing Lock-Strip Gaskets and Infill Glazing Materials.
ASTM C719	Standard Test Method for Adhesion and Cohesion of Elastomeric Joint Sealants Under Cyclic Movement (Hockman Cycle).
ASTM C1193	Standard Guide for Use of Joint Sealants.
ASTM C794	Standard Test Method for Adhesion-in-Peel of Elastomeric Joint Sealants.
ASTM C834	Standard Specification for Latex Sealants.
ASTM C864	Standard Specification for Dense Elastomeric Compression Seal Gaskets, Setting Blocks, and Spacers.
ASTM C919	Standard Practice for Use of Sealants in Acoustical Applications.
ASTM C920	Standard Specification for Elastomeric Joint Sealants.
ASTM C1193	Standard Guide for Use of Joint Sealants.
ASTM D2628	Standard Specification for Preformed Polychloroprene Elastomeric Joint Seals for Concrete Pavements.
ASTM D6690	Standard Specification for Joint and Crack Sealants, Hot Applied, for Concrete and Asphalt Pavements.
ASTM D3406	Standard Specification for Joint Sealant, Hot-Applied, Elastomeric-Type, for Portland Cement Concrete Pavements.
ASTM D3667	Standard Specification for Rubber Seals Used in Flat-Plate Solar Collectors.
ASTM D3771	Standard Specification for Rubber Seals Used in Concentrating Solar Collectors.

ASTM D3832	Standard Specification for Rubber Seals Contacting Liquids in Solar Energy Systems.
ISO 3934	Rubber building gaskets-materials in preformed solid vulcanizates used for sealing glazing and panels-specification.
ISO 4633	Rubber seals-joint rings for water supply, drainage and sewerage pipelines-specifications for materials
ISO 4635	Rubber, vulcanized-preformed compression seals for use between concrete motorway paving sections-specifications for material.
ISO 5892	Rubber Building Gaskets-Materials for Preformed Solid Vulcanized Structural Gaskets-Specification;
ISO 6447	Rubber seals-joint rings used for gas supply pipes and fittings-specification for material.
ISO 9331	Rubber seals joint rings for hot water supply pipelines up to 110º C specifications for the material.

2.16.13 Joints and Jointing Products

Joints and jointing products shall conform to the following Standards:

ISO 2444	Joints in buildings-vocabulary.
ISO 3867	Agglomerated cork-material of expansion joints for construction and building test-methods.
ISO 3869	Agglomerated cork-filler material of expansion joints for construction and buildings -characteristics, sampling and packing.
ISO 3934	Rubber building gaskets-materials in preformed solid vulcanizates used for sealing glazing and panels-specification.
ISO 4633	Rubber seals-joint rings for water supply, drainage and sewerage pipelines-specification for materials.
ISO 4635	Rubber, vulcanized-preformed compression seals for use between concrete motor way paving sections-specification for material.
ISO 5892	Rubber building gaskets-materials for preformed solid vulcanized structural gaskets-specification.

ISO 6447	Rubber seals-joint rings used for gas supply pipes and fittings-specification for material.
ISO 6589	Joints in building-laboratory method of test for air permeability of joints.
ISO 7389	Building construction-jointing products-determination of elastic recovery.
ISO 7390	Building construction-jointing products-determination of resistance to flow.
ISO 7727	Joints in building-principles for jointing of building components-accommodation of dimensional deviations during construction.
ISO 8339	Building construction-jointing products-sealants-determination of tensile properties.
ISO 8340	Building construction-jointing products-sealants-determination of tensile properties at maintained extension.
ISO 8394	Building construction-jointing products-determination of extrudability of one-component sealants.
ISO 9046	Building construction-sealants-determination of adhesion/ cohesion properties at constant temperature.
ISO 9047	Building construction-sealants-determination of adhesion/ cohesion properties at variable temperatures.
ISO 9631	Rubber seals-joint rings for hot water supply pipelines up to 110^{o}C specifications for the material.
ISO 10563	Building construction-sealants for joints-determination of change in mass and volume.
ISO 10590	Building construction-sealants-determination of adhesion/cohesion properties at maintained extension after immersion in water.
ISO 10591	Building construction-sealants-determination of adhesion/cohesion properties after immersion in water.

2.16.14 Glass and Glazing

The applicable Standards for glass and glazing are listed below:

ASTM C1036	Standard Specification for Flat Glass.
ASTM C1048	Standard Specification for Heat-Treated Flat Glass-Kind HS, Kind FT Coated and Uncoated Glass.
ANSI Z 97.1	Safety Performance Specifications and Methods of Tests for Transport Safety Glazing Materials Used in Building.
CPSC 16 CFR	Safety Standard for Architectural Glazing Materials. Part 1201A.

2.17 Cgi Sheet Roofing and Walling

Galvanized corrugated steel sheets conforming to BDS 868, Galvanized Corrugated Sheet Roof and Wall Coverings, may be used over structural framing for construction of roofs and walls. Requirements for various roofing materials including CGI sheet have been specified in Sec 2.13 above.

Part VI
Chapter 1
Definitions and General Requirements

1.1 Introduction

1.1.1 Scope

The definitions providing meanings of different terms and general requirements for the structural design of buildings, structures, and components thereof are specified in this Chapter. These requirements shall apply to all buildings and structures or their components regulated by this Code. All anticipated loads required for structural design shall be determined in accordance with the provisions of Chapter 2. Design parameters required for the structural design of foundation elements shall conform to the provisions of Chapter 3. Design of structural members using various construction materials shall comply with the relevant provisions of Chapters 4 to 13. The FPS equivalents of the empirical expressions used throughout Part 6 are listed in Appendix A.

This Code shall govern in all matters pertaining to design, construction, and material properties wherever this Code is in conflict with requirements contained in other standards referenced in this Code. However, in special cases where the design of a structure or its components cannot be covered by the provisions of this Code, other relevant internationally accepted codes referred in this Code may be used.

1.1.2 Definitions

The following definitions shall provide the meaning of certain terms used in this Chapter.

BASE SHEAR	Total design lateral force or shear at the base of a structure.
BASIC WIND SPEED	Three-second gust speed at 10 m above the mean ground level in terrain Exposure-B defined in Sec 2.4.6 and associated with an annual probability of occurrence of 0.02.
BEARING WALL SYSTEM	A structural system without a complete vertical load carrying space frame.
BRACED FRAME	An essentially vertical truss system of the concentric or eccentric type which is provided to resist lateral forces.
BUILDING FRAME SYSTEM	An essentially complete space frame which provides support for loads.

CONCENTRIC BRACED FRAME (CBF)	A steel braced frame designed in conformance with Sec 10.20.13 or Sec 10.20.14.
COLLECTOR	A member or element used to transfer lateral forces from a portion of a structure to the vertical elements of the lateral force resisting elements.
DEAD LOAD	The load due to the weight of all permanent structural and nonstructural components of a building or a structure, such as walls, floors, roofs and fixed service equipment.
DIAPHRAGM	A horizontal or nearly horizontal system acting to transmit lateral forces to the vertical resisting elements. The term "diaphragm" includes horizontal bracing systems.
DUAL SYSTEM	A combination of Moment Resisting Frames and Shear Walls or Braced Frames to resist lateral loads designed in accordance with the criteria of Sec 1.3.2.4.
ECCENTRIC BRACED FRAME (EBF)	A steel braced frame designed in conformance with Sec 10.20.15.
HORIZONTAL BRACING SYSTEM	A horizontal truss system that serves the same function as a floor or roof diaphragm.
INTERMEDIATE MOMENT FRAME (IMF)	A concrete moment resisting frame designed in accordance with Sec 8.3.10.
LIVE LOAD	The load superimposed by the use and occupancy of a building.
MOMENT RESISTING FRAME	A frame in which members and joints are capable of resisting forces primarily by flexure.
ORDINARY MOMENT FRAME (OMF)	A moment resisting frame not meeting special detailing requirements for ductile behaviour.
PRIMARY FRAMING SYSTEM	That part of the structural system assigned to resist lateral forces.
SHEAR WALL	A wall designed to resist lateral forces parallel to the plane of the wall (sometimes referred to as a vertical diaphragm or a structural wall).

SLENDER BUILDINGS AND STRUCTURES	Buildings and structures having a height exceeding five times the least horizontal dimension, or having a fundamental natural frequency less than 1 Hz. For those cases where the horizontal dimensions vary with height, the least horizontal dimension at mid height shall be used.
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 stiffness of the three storeys above.
SPACE FRAME	A three-dimensional structural system without bearing walls composed of members interconnected so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.
SPECIAL MOMENT FRAME (SMF)	A moment resisting frame specially detailed to provide ductile behaviour complying with the requirements of Chapter 8 or 10 for concrete or steel frames respectively.
SPECIAL STRUCTURAL SYSTEM	A structural system not listed in Table 6.1.3 and specially designed to carry the lateral loads. (See Sec 1.3.2.5).
STOREY	The space between any two floor levels including the roof of a building. Storey-x is the storey below level x.
STOREY SHEAR, V_x	The summation of design lateral forces above the storey under consideration.
STRENGTH	The usable capacity of an element or a member to resist the load as prescribed in these provisions.
TERRAIN	The ground surface roughness condition when considering the size and arrangement of obstructions to the wind.
THREE-SECOND GUST SPEED	The highest average wind speed over a 3 second duration at a height of 10 m. The three-second gust speed is derived using Durst's model in terms of the mean wind speed and turbulence intensity.
TOWER	A tall, slim vertical structure.
VERTICAL LOAD-CARRYING FRAME	A space frame designed to carry all vertical gravity loads.
WEAK STOREY	Storey in which the lateral strength is less than 80 percent of that of the storey above.

1.1.3 Symbols and Notation

The following symbols and notation shall apply to the provisions of this Chapter:

D	=	Dead load on a member including self-weight and weight of components, materials and permanent equipment supported by the member
E	=	Earthquake load
F_i	=	Lateral force applied at level $-\ i$ of a building
h	=	Height of a building or a structure above ground level in metres
h_i, h_n, h_x	=	Height in metres above ground level to level $-\ i$, $-n$ or $-x$ respectively
level $-\ i$	=	i^{th} level of a structure above the base; i = 1 designates the first level above the base
level $-\ n$	=	Upper most level of a structure
level $-\ x$	=	x^{th} level of a structure above the base; x = 1 designates the first level above the base.
L	=	Live load due to intended use or occupancy
l	=	Span of a member or component.
M_x	=	Overturning moment at level $-\ x$
V	=	Total design lateral force or shear at the base
V_x	=	Storey shear at storey level $-\ x$
R	=	Response modification or reduction coefficient for structural system given in Table 6.2.19 for seismic design.
T	=	Fundamental period of vibration in seconds
W	=	Load due to wind pressure.
W'	=	Weight of an element or component
Z	=	Seismic zone coefficient given in Figure 6.2.24 or Table 6.2.14 or Table 6.2.15
Δ	=	Storey lateral drift.

1.2 Basic Considerations

1.2.1 General

All buildings and structures shall be designed and constructed in conformance with the provisions of this Section. The buildings and portions thereof shall support all loads including dead load specified in this Chapter and elsewhere in this Code. Impact, fatigue and self-straining forces shall be considered where these forces occur.

1.2.2 Buildings and Structures

A structure shall ordinarily be described as an assemblage of framing members and components arranged to support both gravity and lateral forces. Structures may be classified as building and non-building structures. Structures that enclose a space and are used for various occupancies shall be called buildings or building structures. Structures other than buildings, such as water tanks, bridges, communication towers, chimneys etc., shall be called non-building structures. When used in conjunction with the word building(s), the word structure(s) shall mean non-building structures, e.g. 'buildings and structures' or 'buildings or structures'. Otherwise the word 'structures' shall include both buildings and non-building structures.

1.2.3 Building and Structure Occupancy Categories

Buildings and other structures shall be classified, based on the nature of occupancy, according to Table 6.1.1 for the purposes of applying flood, surge, wind and earthquake provisions. The occupancy categories range from I to IV, where Occupancy Category I represents buildings and other structures with a low hazard to human life in the event of failure and Occupancy Category IV represents essential facilities. Each building or other structure shall be assigned to the highest applicable occupancy category or categories. Assignment of the same structure to multiple occupancy categories based on use and the type of load condition being evaluated (e.g., wind or seismic) shall be permissible.

When buildings or other structures have multiple uses (occupancies), the relationship between the uses of various parts of the building or other structure and the independence of the structural systems for those various parts shall be examined. The classification for each independent structural system of a multiple-use building or other structure shall be that of the highest usage group in any part of the building or other structure that is dependent on that basic structural system.

Table 6.1.1: Occupancy Category of Buildings and other Structures for Flood, Surge, Wind and Earthquake Loads.

Nature of Occupancy	Occupancy Category
Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities	I
All buildings and other structures except those listed in Occupancy Categories I, III and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to: • Buildings and other structures where more than 300 people congregate in one area • Buildings and other structures with day care facilities with a capacity greater than 150 • Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250 • Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities • Healthcare facilities with a capacity of 50 or more resident patients, but not having surgery or emergency Treatment facilities • Jails and detention facilities Buildings and other structures, not included in Occupancy Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure, including, but not limited to: • Power generating stations[a] • Water treatment facilities • Sewage treatment facilities • Telecommunication centers Buildings and other structures not included in Occupancy Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.	III

Buildings and other structures designated as essential facilities, including, but not limited to: • Hospitals and other healthcare facilities having surgery or emergency treatment facilities • Fire, rescue, ambulance, and police stations and emergency vehicle garages • Designated earthquake, hurricane, or other emergency shelters • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response • Power generating stations and other public utility facilities required in an emergency • Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, • Electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Occupancy Category IV structures during an emergency • Aviation control towers, air traffic control centers, and emergency aircraft hangars • Community water storage facilities and pump structures required to maintain water pressure for fire suppression • Buildings and other structures having critical national defense functions Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction.	IV

[a] Cogeneration power plants that do not supply power on the national grid shall be designated Occupancy Category II

1.2.4 Safety

Buildings, structures and components thereof, shall be designed and constructed to support all loads, including dead loads, without exceeding the allowable stresses or specified strengths (under applicable factored loads) for the materials of construction in the structural members and connections.

1.2.5 Serviceability

Structural framing systems and components shall be designed with adequate stiffness to have deflections, vibration, or any other deformations within the serviceability limit of building or structure. The deflections of structural members shall not exceed the more restrictive of the limitations provided in Chapters 2 through 13 or that permitted by Table 6.1.2 or the notes that follow. For wind and earthquake loading, story drift and sway shall be limited in accordance with the provisions of Sec 1.5.6. In checking the serviceability, the load combinations and provisions of Sec 2.7.5 shall be followed.

Table 6.1.2: Deflection Limits[a, b, c, h] (Except earthquake load)

Construction	L	W^f	$D^g + L^d$
Roof members:[e]			
Supporting plaster ceiling	$l/360$	$l/360$	$l/240$
Supporting non-plaster ceiling	$l/240$	$l/240$	$l/180$
Not supporting ceiling	$l/180$	$l/180$	$l/120$
Floor members	$l/360$	-	$l/240$
Exterior walls and interior partitions			
With brittle finishes	-	$l/240$	
With flexible finishes	-	$l/120$	
Farm buildings	-		$l/180$
Greenhouses	-		$l/120$

Where, l, L, W and D stands for span of the member under consideration, live load, wind load and dead load respectively.

Notes:

a. For structural roofing and siding made of formed metal sheets, the total load deflection shall not exceed $l/60$. For secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed $l/150$. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed $l/90$. For roofs, this exception only applies when the metal sheets have no roof covering.

b. Interior partitions not exceeding 2 m in height and flexible, folding and portable partitions are not governed by the provisions of this Section.

c. For cantilever members, l shall be taken as twice the length of the cantilever.

d. For wood structural members having a moisture content of less than 16% at time of installation and used under dry conditions, the deflection resulting from $L + 0.5D$ is permitted to be substituted for the deflection resulting from $L + D$.

e. The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to assure adequate drainage shall be investigated for ponding. See Sec 1.6.5 for rain and ponding requirements.

f. The wind load is permitted to be taken as 0.7 times the "component and cladding" loads for the purpose of determining deflection limits herein.

g. Deflection due to dead load shall include both instantaneous and long term effects.

h. For aluminum structural members or aluminum panels used in skylights and sloped glazing framing, roofs or walls of sunroom additions or patio covers, not supporting edge of glass or aluminum sandwich panels, the total load deflection shall not exceed $l/60$. For continuous aluminum structural members supporting edge of glass, the total load deflection shall not exceed $l/175$ for each glass lite or $l/60$ for the entire length of the member, whichever is more stringent. For aluminum sandwich panels used in roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed $l/120$.

1.2.6 Rationality

Structural systems and components thereof shall be analyzed, designed and constructed based on rational methods which shall include, but not be limited to the provisions of Sec 1.2.7.

1.2.7 Analysis

Analysis of the structural systems shall be made for determining the load effects on the resisting elements and connections, based on well-established principles of mechanics taking equilibrium, geometric compatibility and both short and long term properties of the construction materials into account and incorporating the following:

1.2.7.1 Mathematical model

A mathematical model of the physical structure shall represent the spatial distribution of stiffness and other properties of the structure which is adequate to provide a complete load path capable of transferring all loads and forces from their points of origin to the load-resisting elements for obtaining various load effects. For dynamic analysis, mathematical model shall also incorporate the appropriately distributed mass and damping properties of the structure adequate for the determination of the significant features of its dynamic response. All buildings and structures shall be thus analyzed preferably using a three dimensional computerized model incorporating these features of mathematical model. It is essential to use three dimensional computer model to represent a structure having irregular plan configuration as mentioned in Sec 1.3.4.2 and having rigid or semirigid floor and roof diaphragms. Requirements for two-dimensional model and three dimensional models for earthquake analysis are described in Sections 2.5.11 to 2.5.14.

1.2.7.2 Loads and forces

All prescribed loads and forces to be supported by the structural systems shall be determined in accordance with the applicable provisions of this Chapter and Chapter 2. Loads shall be applied on the mathematical model specified in Sec. 1.2.7.1 at appropriate spatial locations and along desired directions.

1.2.7.3 Soil-structure interaction

Soil-structure interaction effects, where required, shall be included in the analysis by appropriately including the properly substantiated properties of soil into the mathematical model specified in Sec. 1.2.7.1 above.

1.2.8 Distribution of Horizontal Shear

The total lateral force shall be distributed to the various elements of the lateral force-resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing systems or diaphragms.

1.2.9 Horizontal Torsional Moments

Structural systems and components shall be designed to sustain additional forces resulting from torsion due to eccentricity between the centre of application of the lateral forces and the centre of rigidity of the lateral force resisting system. Forces shall not be decreased due to torsional effects. For accidental torsion effects on seismic forces, requirements shall conform to Sec 2.5.7.6.

1.2.10 Stability Against Overturning and Sliding

Every building or structure shall be designed to resist the overturning and sliding effects caused by the lateral forces specified in this Chapter.

1.2.11 Anchorage

Anchorage of the roof to wall and columns, and of walls and columns to foundations, shall be provided to resist the uplift and sliding forces resulting from the application of the prescribed loads. Additional requirements for masonry or concrete walls shall be those given in Sec 1.7.3.6.

1.2.12 General Structural Integrity

Buildings and structural systems shall possess general structural integrity that is the ability to sustain local damage caused due to misuse or accidental overloading, with the structure as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage.

1.2.13 Proportioning of Structural Elements

Structural elements, components and connections shall be proportioned and detailed based on the design methods provided in the subsequent Chapters for various materials of construction, such as reinforced concrete, masonry, steel etc. to resist various load effects obtained from a rational analysis of the structural system.

1.2.14 Walls and Framing

Walls and structural framing shall be erected true to plumb in accordance with the design. Interior walls, permanent partitions and temporary partitions exceeding 1.8 m of height shall be designed to resist all loads to which they are subjected. If not otherwise specified elsewhere in this Code, walls shall be designed for a minimum load of 0.25 kN/m^2 applied perpendicular to the wall surfaces. The deflection of such walls under a load of 0.25 kN/m^2 shall not exceed $\frac{1}{240}$ of the span for walls with brittle finishes and $\frac{1}{120}$ of the span for walls with flexible finishes. However, flexible, folding or portable partitions shall not be required to meet the above load and deflection criteria, but shall be anchored to the supporting structure.

1.2.15 Additions to Existing Structures

When an existing building or structure is extended or otherwise altered, all portions thereof affected by such cause shall be strengthened, if necessary, to comply with the safety and serviceability requirements provided in Sections 1.2.4 and 1.2.5 respectively.

1.2.16 Phased Construction

When a building or structure is planned or anticipated to undergo phased construction, structural members therein shall be investigated and designed for any additional stresses arising due to such construction.

1.2.17 Load Combinations and Stress Increase

Every building, structure, foundation or components thereof shall be designed to sustain, within the allowable stress or specified strength (under factored load), the most unfavourable effects resulting from various combinations of loads specified in Sec 2.7. Except otherwise permitted or restricted by any other Sections of this Code, maximum increase in the allowable stress shall be 33% when allowable or working stress method of design is followed. For soil stresses due to foundation loads, load combinations and stress increase specified in Sec 2.7.2 for allowable stress design method shall be used.

1.3 Structural Systems

1.3.1 General

Every structure shall have one of the basic structural systems specified in Sec 1.3.2 or a combination thereof. The structural configuration shall be as specified in Sec 1.3.4 with the limitations imposed in Sec 2.5.5.4.

1.3.2 Basic Structural Systems

Structural systems for buildings and other structures shall be designated as one of the types A to G listed in Table 6.1.3. Each type is again classified as shown in the Table by the types of vertical elements used to resist lateral forces. A brief description of different structural systems are presented in following sub-sections.

1.3.2.1 Bearing wall system

A structural system having bearing walls/bracing systems without a complete vertical load carrying frame to support gravity loads. Resistance to lateral loads is provided by shear walls or braced frames.

1.3.2.2 Building frame system

A structural system with an essentially complete space frame providing support for gravity loads. Resistance to lateral loads is provided by shear walls or braced frames separately.

1.3.2.3 Moment resisting frame system

A structural system with an essentially complete space frame providing support for gravity loads. Moment resisting frames also provide resistance to lateral load primarily by flexural action of members, and may be classified as one of the following types:

(a) Special Moment Frames (SMF)

(b) Intermediate Moment Frames (IMF)

(c) Ordinary Moment Frames (OMF).

The framing system, IMF and SMF shall have special detailing to provide ductile behaviour conforming to the provisions of Sections 8.3 and 10.20 of Part 6 for concrete and steel structures respectively. OMF need not conform to these special ductility requirements of Chapter 8 or 10.

Table 6.1.3: Basic Structural Systems

A. BEARING WALL SYSTEMS (no frame)
1. Special reinforced concrete shear walls
2. Ordinary reinforced concrete shear walls
3. Ordinary reinforced masonry shear walls
4. Ordinary plain masonry shear walls
B. BUILDING FRAME SYSTEMS (with bracing or shear wall)
1. Steel eccentrically braced frames, moment resisting connections at columns away from links
2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links
3. Special steel concentrically braced frames
4. Ordinary steel concentrically braced frames
5. Special reinforced concrete shear walls
6. Ordinary reinforced concrete shear walls
7. Ordinary reinforced masonry shear walls
8. Ordinary plain masonry shear walls

C. MOMENT RESISTING FRAME SYSTEMS (no shear wall)

1. Special steel moment frames
2. Intermediate steel moment frames
3. Ordinary steel moment frames
4. Special reinforced concrete moment frames
5. Intermediate reinforced concrete moment frames
6. Ordinary reinforced concrete moment frames

D. DUAL SYSTEMS: SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)

1. Steel eccentrically braced frames
2. Special steel concentrically braced frames
3. Special reinforced concrete shear walls
4. Ordinary reinforced concrete shear walls

E. DUAL SYSTEMS: INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)

1. Special steel concentrically braced **f**rames
2. Special reinforced concrete shear walls
3. Ordinary reinforced masonry shear walls
4. Ordinary reinforced concrete shear walls

F. DUAL SHEAR WALL-FRAME SYSTEM: ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS

G. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE

1.3.2.4 Dual system

A structural system having a combination of the following framing systems:

(a) Moment resisting frames (SMF, IMF or steel OMF), and

(b) Shear walls or braced frames.

The two systems specified in (a) and (b) above shall be designed to resist the total lateral force in proportion to their relative rigidities considering the interaction of the dual system at all levels. However, the moment resisting frames shall be capable of resisting at least 25% of the applicable total seismic lateral force, even when wind or any other lateral force governs the design.

1.3.2.5 Special structural system

A structural system not defined above nor listed in Table 6.1.3 and specially designed to carry the lateral loads, such as tube-in-tube, bundled tube, etc.

1.3.2.6 Non-building structural system

A structural system used for purposes other than in buildings and conforming to Sections 1.5.4.8, 1.5.4.9, 2.4 and 2.5 of Part 6.

1.3.3 Combination of Structural Systems

When different structural systems of Sec 1.3.2 are combined for incorporation into the same structure, design of the combined seismic force resisting system shall conform to the provisions of Sec 2.5.5.5.

1.3.4 Structural Configurations

Based on the structural configuration, each structure shall be designated as a regular or irregular structure as defined below:

1.3.4.1 Regular structures

Regular structures have no significant physical discontinuities or irregularities in plan or vertical configuration or in their lateral force resisting systems. Typical features causing irregularity are described in Sec 1.3.4.2.

1.3.4.2 Irregular structures

Irregular structures have either vertical irregularity or plan irregularity or both in their structural configurations or lateral force resisting systems.

1.3.4.2.1 Vertical irregularity

Structures having one or more of the irregular features listed in Table 6.1.4 shall be designated as having a vertical irregularity.

1.3.4.2.2 Plan irregularity

Structures having one or more of the irregular features listed in Table 6.1.5 shall be designated as having a plan irregularity.

Table 6.1.4: Vertical Irregularities of Structures

Vertical Irregularity Type	Definition	Reference Section
I	**a. Stiffness Irregularity (Soft Storey):** 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 stiffness of the three storeys above. **b. Stiffness Irregularity (Extreme Soft Storey):** Extreme soft storey irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	1.7.3.8, 2.5.5 to 2.5.14 and 2.5.17
II	**Mass Irregularity:** Mass irregularity shall be considered to exist where the effective mass of any storey is more than 150 percent of the effective mass of an adjacent storey. A roof which is lighter than the floor below need not be considered.	2.5.5 to 2.5.14
III	**Vertical Geometric Irregularity:** Vertical geometric irregularity shall be considered to exist where horizontal dimension of the lateral force-resisting system in any storey is more than 130 percent of that in an adjacent storey, one-storey penthouses need not be considered.	2.5.5 to 2.5.14
IV	**In-Plane Discontinuity in Vertical Lateral Force-Resisting Element:** An in-plane offset of the lateral load-resisting elements greater than the length of those elements.	1.7.3.8, 2.5.5 to 2.5.14
Va	**Discontinuity in Capacity (Weak Storey):** A weak storey is one in which the storey strength is less than 80 percent of that in the storey above. The storey strength is the total strength of all seismic-resisting elements sharing the storey shear for the direction under consideration.	2.5.5 to 2.5.14 and 2.5.17
Vb	**Extreme Discontinuity in Capacity (Very Weak Storey):** A very weak storey is one in which the storey strength is less than 65 percent of that in the storey above.	2.5.5 to 2.5.14 and 2.5.17

Table 6.1.5: Plan (Horizontal) Irregularities of Structures

Plan Irregularity Type	Definition	Reference Section
I	**Torsional Irregularity (to be considered when diaphragms are not flexible):** a. Torsional irregularity shall be considered to exist when the maximum storey drift, computed including accidental torsion, at one end of the structure is more than 1.2 times the average of the storey drifts at the two ends of the structure. b. Extreme Torsional Irregularity is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	1.7.3.8, 2.5.5 to 2.5.14
II	**Reentrant Corners:** Plan configurations of a structure and its lateral force-resisting system contain reentrant corners, where both projections of the structure beyond a reentrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	1.7.3.8, 2.5.5 to 2.5.14
III	**Diaphragm Discontinuity:** Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next.	1.7.3.8, 2.5.5 to 2.5.14
IV	**Out-of-plane Offsets:** Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.	1.7.3.8, 2.5.5 to 2.5.14
V	**Nonparallel Systems:** The vertical lateral load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral force-resisting system.	2.5.5 to 2.5.15

1.4 Design For Gravity Loads

1.4.1 General

Design of buildings and components thereof for gravity loads shall conform to the requirements of this Section. Gravity loads, such as dead load and live loads applied at the floors or roof of a building shall be determined in accordance with the provisions of Chapter 2 of this Part.

1.4.2 Floor Design

Floor slabs and decks shall be designed for the full dead and live loads as specified in Sections 2.2 and 2.3 respectively. Floor supporting elements such as beams, joists, columns etc. shall be designed for the full dead load and the appropriately reduced live loads set forth by the provisions of Sec 2.3.13. Design of floor elements shall also conform to the following provisions:

(a) Uniformly Distributed Loads: Where uniform floor loads are involved, consideration may be limited to full dead load on all spans in combination with full live load on adjacent spans and on alternate spans to determine the most unfavourable effect of stresses in the member concerned.

(b) Concentrated Loads: Provision shall be made in designing floors for a concentrated load as set forth in Sec 2.3.5 applied at a location wherever this load acting upon an otherwise unloaded floor would produce stresses greater than those caused by the uniform load required therefore.

(c) Partition Loads: Loads due to permanent partitions shall be treated as a dead load applied over the floor as a uniform line load having intensity equal to the weight per metre run of the partitions as specified in Sec 2.2.5. Loads for light movable partitions shall be determined in accordance with the provisions of Sec 2.3.6.

(d) Design of Members: Floor members, such as slabs or decks, beams, joists etc. shall be designed to sustain the worst effect of the dead plus live loads or any other load combinations as specified in Sec 2.7. Where floors are used as diaphragms to transmit lateral loads between various resisting elements, those loads shall be determined following the provisions of Sec 1.7.3.8. Detailed design of the floor elements shall be performed using the procedures provided in Chapters 4 to 13 of Part 6 for various construction materials.

(e) Floors and associated structural members shall have adequate strength and stiffness to prevent undesirable vibration due to human activity (e.g walking, dancing, jumping, sporting activities etc.) or vibration caused by machines which causes discomfort to the occupants and which is detrimental to the safety, integrity and durability of the structure.

1.4.3 Roof Design

Roofs and their supporting elements shall be designed to sustain, within their allowable stresses or specified strength limits, all dead loads and live loads as set out by the provisions of Sections 2.2 and 2.3 respectively. Design of roof members shall also conform to the following requirements:

(a) Application of Loads: When uniformly distributed loads are considered for the design of continuous structural members, load including full dead loads on all spans in combination with full live loads on adjacent spans and on alternate span, shall be investigated to determine the worst effects of loading. Concentrated roof live loads and special roof live loads, where applicable, shall also be considered in design.

(b) Unbalanced Loading: Effects due to unbalanced loads shall be considered in the design of roof members and connections where such loading will result in more critical stresses. Trusses and arches shall be designed to resist the stresses caused by uniform live loads on one half of the span if such loading results in reverse stresses, or stresses greater in any portion than the stresses produced by this unit live load when applied upon the entire span.

(c) Rain Loads: Roofs, where ponding of rain water is anticipated due to blockage of roof drains, excessive deflection or insufficient slopes, shall be designed to support such loads. Loads on roofs due to rain shall be determined in accordance with the provisions of Sec 2.6.2. In addition to the dead load of the roof, either the roof live load or the rain load, whichever is of higher intensity, shall be considered in design.

1.4.4 Reduction of Live Loads

The design live loads specified in Sec 2.3, may be reduced to appropriate values as permitted by the provisions of Sections 2.3.13 and 2.3.14.

1.4.5 Posting of Live Loads

In every building, of which the floors or parts thereof have a design live load of 3.5 kN/m^2 or more, and which are used as library stack room, file room, parking garage, machine or plant room, or used for industrial or storage purposes, the owner of the building shall ensure that the live loads for which such space has been designed, are posted on durable metal plates as shown in Figure 6.1.1, securely affixed in a conspicuous place in each space to which they relate. If such plates are lost, removed, or defaced, owner shall be responsible to have them replaced.

1.4.6 Restrictions on Loading

The building owner shall ensure that the live load for which a floor or roof is or has been designed, will not be exceeded during its use.

Figure 6.1.1 Sample live load sign

1.4.7 Special Considerations

In the absence of actual dead and live load data, the minimum values of these loads shall be those specified in Sections 2.2 and 2.3. In addition, special consideration shall be given to the following aspects of loading and due allowances shall be made in design if occurrence of such loading is anticipated after construction of a building:

(a) Increase in Dead Load: Actual thickness of the concrete slabs or other members may become larger than the designed thickness due to movements or deflections of the formwork during construction.

(b) Future Installations: Changes in the numbers, types and positions of partitions and other installations may increase actual load on the floors of a building.

(c) Occupancy Changes: Increase in live loads due to changes of occupancy involving loads heavier than that being designed for.

1.4.8 Deflection and Camber

Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections. The deflections of structural members shall not exceed the more restrictive of the limitations of Chapters 2 to 13 of this Part or that permitted by Table 6.1.2.or provisions of Sec 1.2.5 of this Chapter. In calculating deflections due to gravity loads, long term effects (e.g. creep, shrinkage or stress relaxation) should also be considered.

1.5 Design For Lateral Loads

1.5.1 General

Every building, structure or portions thereof shall be designed to resist the lateral load effects, such as those due to wind or earthquake forces, in compliance with the requirements prescribed in this Section.

1.5.2 Selection of Lateral Force for Design

Any of the lateral loads prescribed in Chapter 2, considered either alone or in combination with other forces, whichever produces the most critical effect, shall govern the design. However, the structural detailing requirements shall comply with those prescribed in Sec 1.7 of this Chapter. When a dual structural system is used to resist lateral loads, design shall also conform to Sec 1.3.2.4 of this Chapter.

1.5.3 Design for Wind Load

Design of buildings and their components to resist wind induced forces shall comply with the following requirements:

1.5.3.1 Direction of wind

Structural design for wind forces shall be based on assumption that wind may blow from any horizontal direction.

1.5.3.2 Design considerations

Design wind load on the primary framing systems and components of a building or structure shall be determined on the basis of the procedures provided in Sec 2.4 Chapter 2 Part 6 considering the basic wind speed, shape and size of the building, and the terrain exposure condition of the site. For slender buildings and structures, dynamic response characteristics, such as fundamental natural frequency, shall be determined to estimate gust response coefficient. Load effects, such as forces, moments, and deflections etc. on various components of building due to wind shall be determined from static analysis of the structure as specified in Sec 1.2.7.1 of this Chapter.

1.5.3.3 Shielding effect

Reductions in wind pressure on buildings and structures due to apparent direct shielding effects of the up wind obstructions, such as man-made constructions or natural terrain features, shall not be permitted.

1.5.3.4 Dynamic effects

Dynamic wind forces such as that from along-wind vibrations caused by the dynamic wind-structure interaction effects, as set forth by the provisions of Sec 2.4.8 Chapter 2 Part 6, shall be considered in the design of regular shaped slender buildings. For other dynamic effects such as cross-wind or torsional responses as may be experienced by buildings or structures having unusual geometrical shapes (i.e. vertical or plan irregularities listed in Tables 6.1.4 and 6.1.5), response characteristics, or site locations, structural design shall be made based on the information obtained either from other reliable references or from wind-tunnel test specified in Sec 1.5.3.5 below, complying with the other requirements of this Section.

1.5.3.5 Wind tunnel test

Properly conducted wind-tunnel tests shall be required for those buildings or structures having unusual geometric shapes, response characteristics, or site locations for which cross-wind response such as vortex shedding, galloping etc. warrant special consideration, and for which no reliable literature for the determination of such effects is available. This test is also recommended for those buildings or structures for which more accurate wind-loading information is desired than those given in this Section and in Sec 2.4. Tests for the determination of mean and fluctuating components of forces and pressures shall be considered to be properly conducted only if the following requirements are satisfied:

(a) The natural wind has been modelled to account for the variation of wind speed with height,

(b) The intensity of the longitudinal components of turbulence has been taken into consideration in the model,

(c) The geometric scale of the structural model is not more than three times the geometric scale of the longitudinal component of turbulence,

(d) The response characteristics of the wind tunnel instrumentation are consistent with the measurements to be made, and

(e) The Reynolds number is taken into consideration when determining forces and pressures on the structural elements.

Tests for the purpose of determining the dynamic response of a structure shall be considered to be properly conducted only if requirements (a) through (e) above are fulfilled and, in addition, the structural model is scaled with due consideration to length, distribution of mass, stiffness and damping of the structure.

1.5.3.6 Wind loads during construction

Buildings, structures and portions thereof under construction, and construction structures such as formwork, staging etc. shall be provided with adequate temporary bracings or other lateral supports to resist the wind load on them during the erection and construction phase.

1.5.3.7 Masonry construction in high-wind regions

Design and construction of masonry structures in high-wind regions shall conform to the requirements of relevant Sections of Chapter 7 Part 6.

1.5.3.8 Height limits

Unless otherwise specified elsewhere in this Code, no height limits shall be imposed, in general, on the design and construction of buildings or structures to resist wind induced forces.

1.5.4 Design for Earthquake Forces

Design of structures and components thereof to resist the effects of earthquake forces shall comply with the requirements of this Section.

1.5.4.1 Basic design consideration

For the purpose of earthquake resistant design, each structure shall be placed in one of the seismic zones as given in Sec 2.5.4.2 and assigned with a structure importance category as set forth in Sec 2.5.5.1. The seismic forces on structures shall be determined considering seismic zoning, site soil characteristics, structure importance, structural systems and configurations, height and dynamic properties of the structure as provided in Sec 2.5. The structural system and configuration types for a building or a structure shall be determined in accordance with the provisions of Sec 2.5.5.4. Other seismic design requirements shall be those specified in this Section.

1.5.4.2 Requirements for directional effects

The directions of application of seismic forces used in the design shall be those which will produce the most critical load effects. Earthquake forces act in both principal directions of the building simultaneously. Design provisions for considering earthquake component in orthogonal directions have been provided in Sec 2.5.13.1.

1.5.4.3 Structural system and configuration requirements

Seismic design provisions impose the following limitations on the use of structural systems and configurations:

(a) The structural system used shall satisfy requirements of the Seismic Design Category (defined in Sec. 2.5.5.2) and height limitations given in Sec 2.5.5.4.

(b) Structures assigned to Seismic Design Category D having vertical irregularity Type Vb of Table 6.1.4 shall not be permitted. Structures with such vertical irregularity may be permitted for Seismic Design Category B or C but shall not be over two stories or 9 m in height.

(c) Structures having irregular features described in Table 1.3.2 or Table 1.3.3 shall be designed in compliance with the additional requirements of the Sections referenced in these Tables.

(d) Special Structural Systems defined in Sec 1.3.2.5 may be permitted if it can be demonstrated by analytical and test data to be equivalent, with regard to dynamic characteristics, lateral force resistance and energy absorption, to one of the structural systems listed in Table 6.2.19, for obtaining an equivalent R and C_d value for seismic design.

1.5.4.4 Methods of analysis

Earthquake forces and their effects on various structural elements shall be determined by using either a static analysis method or a dynamic analysis method whichever is applicable based on the limitations set forth in Sections 2.5.5 to 2.5.12 and conforming to Sec 1.2.7.

1.5.4.5 Minimum design seismic force

The minimum design seismic forces shall be those determined in accordance with the Sections 2.5.5 to 2.5.14 whichever is applicable.

1.5.4.6 Distribution of seismic forces

The total lateral seismic forces and moments shall be distributed among various resisting elements at any level and along the vertical direction of a building or structure in accordance with the provisions of Sections 2.5.5 to 2.5.12 as appropriate.

1.5.4.7 Vertical components of seismic forces

Design provisions for considering vertical component of earthquake ground motion is given in Sec 2.5.13.2

1.5.4.8 Height limits

Height limitations for different structural systems are given in Table 6.2.19 of Sec 2.5.3.4 Chapter 2 Part 6 of this Code as a function of seismic design category.

1.5.4.9 Non-building structures

Seismic lateral force on non-building structures shall be determined in accordance with the provisions of ASCE 7: Minimum Design Loads for Buildings and other Structures. However, provisions of ASCE 7 may be simplified, consistent with the provisions of Sec 2.5 Part 6 of this Code. Other design requirements shall be those provided in this Chapter.

1.5.5 Overturning Requirements

Every structure shall be designed to resist the overturning effects caused by wind or earthquake forces specified in Sections 2.4 and 2.5 respectively as well other lateral forces like earth pressure, tidal surge etc. The overturning moment M_x at any storey level-x of a building shall be determined as:

$$M_x = \sum_{i=1}^{n} F_i (h_i - h_x) \qquad (6.1.1)$$

Where,

h_i, h_x, h_n = Height in metres at level- i, -x or -n respectively.

F_i = Lateral force applied at level- i, $i = 1$ to n.

At any level, the increment of overturning moment shall be distributed to the various resisting elements in the same manner as the distribution of horizontal shear prescribed in Sec 2.5.7.5. Overturning effects on every element shall be carried down to the foundation level.

1.5.6 Drift and Building Separation

1.5.6.1 Storey drift limitation

Storey drift is the horizontal displacement of one level of a building or structure relative to the level above or below due to the design gravity (dead and live loads) or lateral forces (e.g. wind and earthquake loads). Calculated storey drift shall include both translational and torsional deflections and conform to the following requirements:

(a) Storey drift, Δ, for loads other than earthquake loads, shall be limited as follows:

$\Delta \leq 0.005h$ for $T < 0.7$ second

$\Delta \leq 0.004h$ for $T \geq 0.7$ second

$\Delta \leq 0.0025h$ for unreinforced masonry structures.

Where, h = height of floor. The period T used in this calculation shall be the same as that used for determining the base shear in Sec 2.5.7.2.

(b) The drift limits set out in (a) above may be exceeded where it can be demonstrated that greater drift can be tolerated by both structural and nonstructural elements without affecting life safety.

(c) For earthquake loads, the story drift, Δ shall be limited in accordance with the limits set forth in Sec 2.5.14.1

1.5.6.2 Sway limitation

The overall sway (horizontal deflection) at the top level of the building or structure due to wind loading shall not exceed $\frac{1}{500}$ times the total height of the building above ground, in accordance with Sec 2.7.5.

1.5.7 Building Separation

All components of a structure shall be designed and constructed to act as an integral unit unless they are separated structurally by a distance sufficient to avoid contact under the most unfavorable condition of deflections due to lateral loads. For seismic loads, design guidelines are given in Sec 2.5.14.3.

1.5.8 P-Delta Effects

The resulting member forces and moments and the storey drifts induced by P-Delta effects need not be considered when the stability coefficient (θ) remains within 0.10. This coefficient (described in Sec 2.5.7.9) may be evaluated for any storey as the product of the total vertical dead and live loads above the storey and the lateral drift in that storey divided by the product of the storey shear in that storey and the height of that storey.

1.5.9 Uplift Effects

Uplift effects caused due to lateral loads shall be considered in design. When allowable (working) stress method is used for design, dead loads used to reduce uplift shall be multiplied by a factor of 0.85.

1.6 Design For Miscellaneous Loads

1.6.1 General

Buildings, structures and components thereof, when subject to loads other than dead, live, wind and earthquake loads, shall be designed in accordance with the provisions of this Section. Miscellaneous loads, such as those due to temperature, rain, flood and surge etc. on buildings or structures, shall be determined in accordance with Sec

2.6. Structural members subject to miscellaneous loads, not specified in Sec 2.6 shall be designed using well established methods given in any reliable references, and complying with the other requirements of this Code.

1.6.2 Self-Straining Forces

Self-straining forces such as those arising due to assumed differential settlements of foundations and from restrained dimensional changes due to temperature, moisture, shrinkage, creep, and similar effects, shall be taken into consideration in the design of structural members.

1.6.3 Stress Reversal and Fatigue

Structural members and joints shall be investigated and designed against possible stress reversals caused due to various construction loads. Where required, allowance shall be made in the design to account for the effects of fatigue. The allowable stress may be appropriately reduced to account for such effects in the structural members.

1.6.4 Flood, Tidal/Storm Surge and Tsunami

Buildings, structures and components thereof shall be designed, constructed and anchored to resist flotation, collapse or any permanent movement due to loads including flood, tidal/Storm surge and tsunami, when applicable. Structural members shall be designed to resist both hydrostatic and significant hydrodynamic loads and effects of buoyancy resulting from flood or surge. Flood and surge loads on buildings and structures shall be determined in accordance with Sec 2.6.3. Load combination including flood and surge loads shall conform to Sec 2.7. Design of foundations to sustain these load effects shall conform to the provisions of Sec 1.8.

Stability against overturning and sliding caused due to wind and flood or surge loads simultaneously shall be investigated, and such effects shall be resisted with a minimum factor of safety of 1.5, considering dead load only.

1.6.5 Rain Loads

Roofs of the buildings and structures as well as their other components which may have the capability of retaining rainwater shall be designed for adequate gravity load induced by ponding. Roofs and such other components shall be analysed and designed for load due to ponding caused by accidental blockage of drainage system complying with Sec. 2.6.2.

1.6.6 Other Loads

Buildings and structures and their components shall be analyzed and designed for stresses caused by the following effects:

(a) Temperature Effects (Sec 2.6.4).

(b) Soil and Hydrostatic Pressure (Sec 2.6.5).

(c) Impacts and Collisions

(d) Explosions (Sec 2.6.6).

(e) Fire

(f) Vertical Forces on Air Raid Shelters (Sec 2.6.7).

(g) Loads on Helicopter Landing Areas (Sec 2.6.8).

(h) Erection and Construction Loads (Sec 2.6.9).

(i) Moving Loads for Crane Movements

(j) Creep and Shrinkage

(k) Dynamic Loads due to Vibrations

(l) Construction Loads

Design of buildings and structures shall include loading and stresses caused by the above effects in accordance with the provisions set forth in Chapter 2.

1.7 Detailed Design Requirements

1.7.1 General

All structural framing systems shall comply with the requirements of this Section. Only the elements of the designated lateral force resisting systems can be used to resist design lateral forces specified in Chapter 2. The individual components shall be designed to resist the prescribed forces acting on them. Design of components shall also comply with the specific requirements for the materials contained in Chapters 4 to 13. In addition, such framing systems and components shall comply with the design requirements provided in this Section.

1.7.2 Structural Framing Systems

The basic structural systems are defined in Sec 1.3.2 and shown in Table 6.1.3, and each type is subdivided by the types of framing elements used to resist the lateral forces. The structural system used shall satisfy requirements of seismic design category and height limitations indicated in Table 6.2.19. Special framing requirements are given in the following Sections in addition to those provided in Chapters 4 to 13.

1.7.3 Detailing Requirements for Combinations of Structural Systems

For components common to different structural systems, a more restrictive detailing shall be provided.

1.7.3.1 Connections to resist seismic forces

Connections which resist prescribed seismic forces shall be designed in accordance with the seismic design requirements provided in Chapters 4 to 13. Detailed sketches for these connections shall be given in the structural drawings.

1.7.3.2 Deformation compatibility

All framing elements not required by design to be part of the lateral force resisting system, shall be investigated and shown to be adequate for vertical load carrying capacity when subjected to lateral displacements resulting from the seismic lateral forces. For designs using working stress methods, this capacity may be determined using an allowable stress increase of 30 percent. Geometric non-linear (*P-Delta)* effects on such elements shall be accounted for.

(a) Adjoining Rigid Elements : Moment resisting frames may be enclosed or adjoined by more rigid elements which would tend to prevent a space frame from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load resisting ability of the space frame.

(b) Exterior Elements : Exterior nonbearing, non-shear wall panels or elements which are attached to or enclose the exterior of a structure, shall be designed to resist the forces according to Sec. 2.5.15 of Chapter 2, if seismic forces are present, and shall accommodate movements of the structure resulting from lateral forces or temperature changes. Such elements shall be supported by structural members or by mechanical connections and fasteners joining them to structural members in accordance with the following provisions:

  (i) Connections and panel joints shall allow for a relative movement between storeys of not less than two times the storey drift caused by wind forces or design seismic forces, or 12 mm, whichever is greater.

  (ii) Connections to permit movement in the plane of the panel for storey drift shall be either sliding connections using slotted or oversized holes, connections which permit movement by bending of steel, or other connections providing equivalent sliding and ductility capacity.

(iii) Bodies of connections shall have sufficient ductility and rotation capability to preclude any fracture of the anchoring elements or brittle failures at or near welding.

(iv) Bodies of the connection shall be designed for 1.33 times the seismic force determined by Sec. 2.5.15 of Chapter 2, or equivalent.

(v) All fasteners in the connection system, such as bolts, inserts, welds, dowels etc. shall be designed for 4 times the forces determined by Sec. 2.5.15 of Chapter 2 or equivalent.

(vi) Fasteners embedded in concrete shall be attached to, or hooked around reinforcing steel, or otherwise terminated so as to transfer forces to the reinforcing steel effectively.

1.7.3.3 Ties and continuity

All parts of a structure shall be interconnected. These connections shall be capable of transmitting the prescribed lateral force to the lateral force resisting system. Individual members, including those not part of the seismic force–resisting system, shall be provided with adequate strength to resist the shears, axial forces, and moments determined in accordance with this Code. Connections shall develop the strength of the connected members and shall be capable of transmitting the seismic force (F_p) induced by the parts being connected.

1.7.3.4 Collector elements

Collector elements shall be provided which are capable of transferring the lateral forces originating in other portions of the structure to the element providing the resistance to those forces.

1.7.3.5 Concrete frames

When concrete frames are provided by design to be part of the lateral force resisting system, they shall conform to the provisions of Chapter 8 of this Part.

1.7.3.6 Anchorage of concrete and masonry structural walls

The concrete and masonry structural walls shall be anchored to supporting construction. The anchorage shall provide a positive direct connection between the wall and floor or roof and shall be capable of resisting the horizontal forces specified in Sections 2.4.11 and 2.5.15, or a minimum force of 4.09 kN/m of wall. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 1.2 m. In masonry walls of hollow units or cavity walls, anchors shall be embedded in a reinforced grouted structural element of the wall. Deformations of the floor and roof diaphragms shall be considered in the design of the supported walls and the anchorage forces in the diaphragms shall be determined in accordance with Sec 1.7.3.9 below.

1.7.3.7 Boundary members

Specially detailed boundary members shall be considered for shear walls and shear wall elements whenever their design is governed by flexure.

1.7.3.8 Floor and roof diaphragms

Deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection which will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads. Design of diaphragms shall also comply with the following requirements.

(a) Diaphragm Forces: Diaphragms shall be designed to resist the seismic forces given in Sec 2.5 or for similar non-seismic lateral forces, whichever is greater.

(b) Diaphragm Ties: Diaphragms supporting concrete or masonry walls shall have continuous ties, or struts between the diaphragm chords to distribute the anchorage forces specified in Sec 1.7.3.6 above. Added chords may be provided to form sub-diaphragms to transmit the anchorage forces to the main cross ties.

(c) Wood Diaphragms: Where wood diaphragms are used to laterally support concrete or masonry walls, the anchorage shall conform to Sec 1.7.3.6 above. In seismic Zones 2, 3 and 4 the following requirements shall also apply:

    (i) Anchorage shall not be accomplished by use of toe nails or nails subject to withdrawal, nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension.

    (ii) The continuous ties required by paragraph (b) above, shall be in addition to the diaphragm sheathing.

(d) Structures having irregularities

    (i) For structures assigned to Seismic Design Category D and having a plan irregularity of Type I, II, III, or IV in Table 6.1.5 or a vertical structural irregularity of Type IV in Table 6.1.4, the design forces determined from Sec 2.5.7 shall be increased 25 percent for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements. Collectors and their connections also shall be designed for these increased forces unless they are designed for the load combinations with over strength factor.

(ii) For structures having a plan irregularity of Type II in Table 6.1.5, diaphragm chords and collectors shall be designed considering independent movement of any projecting wings of the structure. Each of these diaphragm elements shall be designed for the more severe of the following cases:

- Motion of the projecting wings in the same direction.
- Motion of the projecting wings in opposing directions.

Exception:

This requirement may be deemed to be satisfied if the procedures of Sec 2.5.8 when seismic forces are present, in conjunction with a three dimensional model, have been used to determine the lateral seismic forces for design.

1.7.3.9 Framing below the base

When structural framings continue below the base, the following requirements shall be satisfied.

(a) **Framing between the Base and the Foundation:** The strength and stiffness of the framing between the base and the foundation shall not be less than that of the superstructure. The special detailing requirements of Sec 8.3 or Sec 10.20, as appropriate for reinforced concrete or steel, shall apply to columns supporting discontinuous lateral force resisting elements and to SMF, IMF, and EBF system elements below the base that are required to transmit forces resulting from lateral loads to foundation.

(b) **Foundations :** The foundation shall be capable of transmitting the design base shear and the overturning forces from the superstructure into the supporting soil, but the short term dynamic nature of the loads may be taken into account in establishing the soil properties. Sec 1.8 below prescribes the additional requirements for specific types of foundation construction.

1.8 Foundation Design Requirements

1.8.1 General

The design and construction of foundation, foundation components and connection between the foundation and superstructure shall conform to the requirements of this Section and applicable provisions of Chapter 3 and other portions of this Code.

1.8.2 Soil Capacities

The bearing capacity of the soil, or the capacity of the soil-foundation system including footing, pile, pier or caisson and the soil, shall be sufficient to support the structure with all prescribed loads, considering the settlement of the structure. For piles, this refers to pile capacity as determined by pile-soil friction and bearing which may be determined in accordance with the provisions of Chapter 3. For the load combination including earthquake, the soil capacity shall be sufficient to resist loads at acceptable strains considering both the short time loading and the dynamic properties of the soil. The stress and settlement of soil under applied loads shall be determined based on established methods of Soil Mechanics.

1.8.3 Superstructure-to-Foundation Connection

The connection of superstructure elements to the foundation shall be adequate to transmit to the foundation the forces for which the elements are required to be designed.

1.8.4 Foundation-Soil Interface

For regular buildings the base overturning moments for the entire structure or for any one of its lateral force-resisting elements, shall not exceed two-thirds of the dead load resisting moment. The weight of the earth superimposed over footings may be used to calculate the dead load resisting moment.

1.8.5 Special Requirements for Footings, Piles and Caissons in Seismic Zones 2, 3 and 4

1.8.5.1 Piles and caissons

Piles and caissons shall be designed for flexure whenever the top of such members is anticipated to be laterally displaced by earthquake motions. The criteria and detailing requirements of Sec 8.3 for concrete and Sec 10.20 for steel shall apply for a length of such members equal to 120 percent of the flexural length.

1.8.5.2 Footing interconnection

(a) Footings and pile caps shall be completely interconnected by strut ties or other equivalent means to restrain their lateral movements in any orthogonal direction.

(b) The strut ties or other equivalent means as specified in (a) above, shall be capable of resisting in tension or compression a force not less than 10% of the larger footing or column load unless it can be demonstrated that equivalent restraint can be provided by frictional and passive soil resistance or by other established means.

1.8.6 Retaining wall design

Retaining walls shall be designed to resist the lateral pressure of the retained material, under drained or undrained conditions and including surcharge, in accordance with established engineering practice. For such walls, the minimum factor of safety against base overturning and sliding due to applied earth pressure shall be 1.5.

1.9 Design and Construction Review

Every building or structure designed shall have its design documents prepared in accordance with the provisions of Sec 1.9.1. The minimum requirements for design review and construction observation shall be those set forth under Sections 1.9.2 and 1.9.3 respectively.

1.9.1 Design Document

The design documents shall be prepared and signed by the Engineer responsible for the structural design of any building or structure intended for construction. The design documents shall include a design report, material specifications and a set of structural drawings, which shall be prepared in compliance with Sections 1.9.2 and 1.9.3 below for submittal to the concerned authority. For the purpose of this provision, the concerned authority shall be either persons from the government approval agency for the construction, or the owner of the building or the structure, or one of his representatives.

1.9.2 Design Report

The design report shall contain the description of the structural design with basic design information as provided below, so that any other structural design engineer will be able to independently verify the design parameters and the member sizes using these basic information. The design report shall include, but not be limited to, the following:

(a) Mention of this Code including relevant Part, Chapter and Section.

(b) Name of other referenced standards, and the specific portions, stating chapter, section etc. of these Code and standards including any specialist report used for the structural design.

(c) Methods used for the calculation of all applied loads along with basic load coefficients and other basic information including any assumption or judgment made under special circumstances.

(d) A drawing of the complete mathematical model prepared in accordance with Sec 1.2.7.1 to represent the structure and showing on it the values, locations and directions of all applied loads, and location of the lateral load resisting systems such as shear walls, braced frames etc.

(e) Methods of structural analysis, and results of the analysis such as shear, moment, axial force etc., used for proportioning various structural members and joints including foundation members.

(f) Methods of structural design including types and strength of the materials of construction used for proportioning the structural members.

(g) Reference of the soil report or any other documents used in the design of the structure, foundation or components thereof.

(h) Statement supporting the validity of the above design documents with date and signature of the engineer responsible for the structural design.

(i) When computer programs are used, to any extent, to aid in the analysis or design of the structure, the following items, in addition to items (a) to (g) above, shall be required to be included in the design report:

(i) A sketch of the mathematical model used to represent the structure in the computer generated analysis.

(ii) The computer output containing the date of processing, program identification, identification of structures being analysed, all input data, units and final results. The computer input data shall be clearly distinguished from those computed in the program.

(iii) A program description containing the information necessary to verify the input data and interpret the results to determine the nature and extent of the analysis and to check whether the computations comply with the provisions of this Code.

(iv) The first sheet of each computer run shall be signed by the engineer responsible for the structural design.

1.9.3 Structural Drawings and Material Specifications

The structural drawings shall include, but not be limited to, the following:

(a) The first sheet shall contain :

(i) Identification of the project to which the building or the structure, or portion thereof belongs,

(ii) Reference to the design report specified in Sec 1.9.2 above,

(iii) Date of completion of design, and

(iv) Identification and signature with date of the engineer responsible for the structural design.

(b) The second sheet shall contain detail material specifications showing:

(i) Specified compressive strength of concrete at stated ages or stages of construction for which each part of structure is designed.

(ii) Specified strength or grade of reinforcement

(iii) Specified strength of prestressing tendons or wires

(iv) Specified strength or grade of steel

(v) Specified strengths for bolts, welds etc.

(vi) Specified strength of masonry, timber, bamboo, ferrocement

(vii) Minimum concrete compressive strength at time of post-tensioning

(viii) Stressing sequence for post-tensioning tendons

(ix) General notes indicating clear cover, development lengths of reinforcements, or any other design parameter relevant to the member or connection details provided in drawings to be followed, as applicable, and

(x) Identification and signature with date of the Engineer responsible for the structural design.

(c) Drawing sheets, other than the first two, shall include structural details of the elements of the structure clearly showing all sizes, cross-sections and relative locations, connections, reinforcements, laps, stiffeners, welding types, lengths and locations etc. whichever is applicable for a particular construction. Floor levels, column centres and offset etc., shall be dimensioned. Camber of trusses and beams, if required, shall be shown on drawings. For bolt connected members, connection types such as slip, critical, tension or bearing type, shall be indicated on the drawing.

(d) Drawings shall be prepared to a scale large enough to show the information clearly and the scales shall be marked on the drawing sheets. If any variation from the design specifications provided in sheet two occurs, the drawing sheet shall be provided additionally with the design specifications including material types and strength, clear cover and development lengths of reinforcements, or any other design parameter relevant to the member or connection details provided in that drawing sheet. Each drawing sheet shall also contain the signature with date of the engineer responsible for the structural design.

1.9.4 Design Review

The design documents specified in Sec 1.9.1 shall be available for review when required by the concerned authority. Review shall be accomplished by an independent structural engineer qualified for this task and appointed by the concerned authority. Design review shall be performed through independent calculations, based on the information provided in the design documents prepared and signed by the original structural design engineer, to verify the design parameters including applied loads, methods of analysis and design, and final design dimensions and other details of the structural elements. The reviewing engineer shall also check the sufficiency and appropriateness of the supplied structural drawings for construction.

1.9.5 Construction Observation

Construction observation shall be performed by a responsible person who will be a competent professional appointed by the owner of the building or the structure. Construction observation shall include, but not be limited to, the following:

(a) Specification of an appropriate testing and inspection schedule prepared and signed with date by the responsible person;

(b) Review of testing and inspection reports; and

(c) Regular site visit to verify the general compliance of the construction work with the structural drawings and specifications provided in Sec 1.9.3 above.

PART VI
Chapter 2
Loads on Buildings and Structures

2.1 Introduction

2.1.1 Scope

This Chapter specifies the minimum design forces including dead load, live load, wind and earthquake loads, miscellaneous loads and their various combinations. These loads shall be applicable for the design of buildings and structures in conformance with the general design requirements provided in Chapter 1.

2.1.2 Limitations

Provisions of this Chapter shall generally be applied to majority of buildings and other structures covered in this Code subject to normally expected loading conditions. For those buildings and structures having unusual geometrical shapes, response characteristics or site locations, or for those subject to special loading including tornadoes, special dynamic or hydrodynamic loads etc., site-specific or case-specific data or analysis may be required to determine the design loads on them. In such cases, and all other cases for which loads are not specified in this Chapter, loading information may be obtained from reliable references or specialist advice may be sought. However, such loads shall be applied in compliance with the provisions of other Parts or Sections of this Code.

2.1.3 Terminology

The following definitions apply only to the provisions of this Chapter:

ALLOWABLE STRESS DESIGN METHOD (ASD)	A method for proportioning structural members such that the maximum stresses due to service loads obtained from an elastic analysis does not exceed a specified allowable value. This is also called Working Stress Design Method (WSD).
APPROVED	Acceptable to the authority having jurisdiction.
BASE	The level at which the earthquake motions are considered to be imparted to the structures or the level at which the structure as a dynamic vibrator is supported.
BASE SHEAR	Total design lateral force or shear due to earthquake at the base of a structure.

BASIC WIND SPEED, V	Three-second gust speed at 10 m above the ground in Exposure B (Sec 2.4.6.3) having a return period of 50 years.
BEARING WALL SYSTEM	A structural system without a complete vertical load carrying space frame.
BRACED FRAME	An essentially vertical truss system of the concentric or eccentric type provided to resist lateral forces.
BUILDING, ENCLOSED	A building that does not comply with the requirements for open or partially enclosed buildings.
BUILDING ENVELOPE	Cladding, roofing, exterior walls, glazing, door assemblies, window assemblies, skylight assemblies, and other components enclosing the building.
BUILDING, LOW-RISE	Enclosed or partially enclosed buildings that comply with the following conditions 1. Mean roof height h less than or equal to 18.3 m. 2. Mean roof height h does not exceed least horizontal dimension.
BUILDING, OPEN	A building having each wall at least 80 percent open. This condition is expressed for each wall by the equation $A_o \geq 0.8A_g$ where, A_o = total area of openings in a wall that receives positive external pressure (m^2). A_g = the gross area of that wall in which A_o is identified (m^2).
BUILDING, PARTIALLY ENCLOSED	A building that complies with both of the following conditions: 1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10 percent. 2. The total area of openings in a wall that receives positive external pressure exceeds 0.37 m^2 or 1 percent of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20 percent.

	These conditions are expressed by the following equations: 1. $A_o > 1.10A_{oi}$ 2. $A_o > 0.37\text{m}^2$ or $> 0.01A_g$, whichever is smaller, and $A_{oi}/A_{gi} \leq 0.20$ Where, A_o, A_g are as defined for open building A_{oi} = the sum of the areas of openings in the building envelope (walls and roof) not including A_o, in m^2. A_{gi} = the sum of the gross surface areas of the building envelope (walls and roof) not including A_g, in m^2.
BUILDING, SIMPLE DIAPHRAGM	A building in which both windward and leeward wind loads are transmitted through floor and roof diaphragms to the same vertical MWFRS (e.g., no structural separations).
BUILDING FRAME SYSTEM	An essentially complete space frame which provides support for gravity loads.
BUILDING OR OTHER STRUCTURE, FLEXIBLE	Slender buildings or other structures that have a fundamental natural frequency less than 1 Hz.
BUILDING OR OTHER STRUCTURE, REGULAR SHAPED	A building or other structure having no unusual geometrical irregularity in spatial form.
BUILDING OR OTHER STRUCTURES, RIGID	A building or other structure whose fundamental frequency is greater than or equal to 1 Hz.
CAPACITY CURVE	A plot of the total applied lateral force, V_j, versus the lateral displacement of the control point, δ_j, as determined in a nonlinear static analysis.
COMPONENTS AND CLADDING	Elements of the building envelope that do not qualify as part of the MWFRS.
CONTROL POINT	A point used to index the lateral displacement of the structure in a nonlinear static analysis.
CRITICAL DAMPING	Amount of damping beyond which the free vibration will not be oscillatory.

CYCLONE PRONE REGIONS	Areas vulnerable to cyclones; in Bangladesh these areas include the Sundarbans, southern parts of Barisal and Patuakhali, Hatia, Bhola, eastern parts of Chittagong and Cox's Bazar
DAMPING	The effect of inherent energy dissipation mechanisms in a structure (due to sliding, friction, etc.) that results in reduction of effect of vibration, expressed as a percentage of the critical damping for the structure.
DESIGN ACCELERATION RESPONSE SPECTRUM	Smoothened idealized plot of maximum acceleration of a single degree of freedom structure as a function of structure period for design earthquake ground motion.
DESIGN EARTHQUAKE	The earthquake ground motion considered (for normal design) as two-thirds of the corresponding Maximum Considered Earthquake (MCE).
DESIGN FORCE, F	Equivalent static force to be used in the determination of wind loads for open buildings and other structures.
DESIGN PRESSURE, p	Equivalent static pressure to be used in the determination of wind loads for buildings.
DESIGN STRENGTH	The product of the nominal strength and a resistance factor.
DIAPHRAGM	A horizontal or nearly horizontal system of structures acting to transmit lateral forces to the vertical resisting elements. The term "diaphragm" includes reinforced concrete floor slabs as well as horizontal bracing systems.
DUAL SYSTEM	A combination of a Special or Intermediate Moment Resisting Frame and Shear Walls or Braced Frames designed in accordance with the criteria of Sec 1.3.2.4
DUCTILITY	Capacity of a structure, or its members to undergo large inelastic deformations without significant loss of strength or stiffness.
EAVE HEIGHT, h	The distance from the ground surface adjacent to the building to the roof eave line at a particular wall. If the height of the eave varies along the wall, the average height shall be used.

ECCENTRIC BRACED FRAME (EBF)	A steel braced frame designed in conformance with Sec 10.20.15.
EFFECTIVE WIND AREA, *A*	The area used to determine GC_p. For component and cladding elements, the effective wind area as mentioned in Sec 2.4.11 is the span length multiplied by an effective width that need not be less than one-third the span length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.
EPICENTRE	The point on the surface of earth vertically above the focus (point of origin) of the earthquake.
ESCARPMENT	Also known as scarp, with respect to topographic effects in Sec 2.4.7, a cliff or steep slope generally separating two levels or gently sloping areas (see Figure 6.2.4).
ESSENTIAL FACILITIES	Buildings and structures which are necessary to remain functional during an emergency or a post disaster period.
FACTORED LOAD	The product of the nominal load and a load factor.
FLEXIBLE DIAPHRAGM	A floor or roof diaphragm shall be considered flexible, for purposes of this provision, when the maximum lateral deformation of the diaphragm is more than two times the average storey drift of the associated storey. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm under lateral load with the storey drift of adjoining vertical resisting elements under equivalent tributary lateral load.
FLEXIBLE ELEMENT OR SYSTEM	An element or system whose deformation under lateral load is significantly larger than adjoining parts of the system.
FREE ROOF	Roof (monoslope, pitched, or troughed) in an open building with no enclosing walls underneath the roof surface.
GLAZING	Glass or transparent or translucent plastic sheet used in windows, doors, skylights, or curtain walls.

GLAZING, IMPACT RESISTANT	Glazing that has been shown by testing in accordance with ASTM E1886 and ASTM E1996 or other approved test methods to withstand the impact of wind-borne missiles likely to be generated in wind-borne debris regions during design winds.
HILL	With respect to topographic effects in Sec 2.4.7, a land surface characterized by strong relief in any horizontal direction (Figure 6.2.4).
HORIZONTAL BRACING SYSTEM	A horizontal truss system that serves the same function as a floor or roof diaphragm.
IMPACT RESISTANT COVERING	A covering designed to protect glazing, which has been shown by testing in accordance with ASTM E1886 and ASTM E1996 or other approved test methods to withstand the impact of wind-borne debris missiles likely to be generated in wind-borne debris regions during design winds.
IMPORTANCE FACTOR, WIND LOAD	A factor that accounts for the degree of hazard to human life and damage to property.
IMPORTANCE FACTOR, EARTHQUAKE LOAD	It is a factor used to increase the design seismic forces for structures of importance.
INTENSITY OF EARTHQUAKE	It is a measure of the amount of ground shaking at a particular site due to an earthquake
INTERMEDIATE MOMENT FRAME (IMF)	A concrete or steel frame designed in accordance with Sec 8.3.10 or Sec 10.20.10 respectively.
LIMIT STATE	A condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).
LIQUEFACTION	State in saturated cohesionless soil wherein the effective shear strength is reduced to negligible value due to pore water pressure generated by earthquake vibrations, when the pore water pressure approaches the total confining pressure. In this condition, the soil tends to behave like a liquid.

LOAD EFFECTS	Forces, moments, deformations and other effects produced in structural members and components by the applied loads.
LOAD FACTOR	A factor that accounts for unavoidable deviations of the actual load from the nominal value and for uncertainties in the analysis that transforms the load into a load effect.
LOADS	Forces or other actions that arise on structural systems from the weight of all permanent constructions, occupants and their possessions, environmental effects, differential settlement, and restrained dimensional changes. Permanent loads are those loads in which variations in time are rare or of small magnitude. All other loads are variable loads.
MAGNITUDE OF EARTHQUAKE	The magnitude of earthquake is a number, which is a measure of energy released in an earthquake.
MAIN WIND-FORCE RESISTING SYSTEM (MWFRS)	An assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface.
MAXIMUM CONSIDERED EARTHQUAKE (MCE)	The most severe earthquake ground motion considered by this Code.
MEAN ROOF HEIGHT, *h*	The average of the roof eave height and the height to the highest point on the roof surface, except that, for roof angles of less than or equal to 10^{o}, the mean roof height shall be the roof heave height.
MODAL MASS	Part of the total seismic mass of the structure that is effective in mode k of vibration.
MODAL PARTICIPATION FACTOR	Amount by which mode k contributes to the overall vibration of the structure under horizontal and vertical earthquake ground motions.
MODAL SHAPE COEFFICIENT	When a system is vibrating in a normal mode, at any particular instant of time, the vibration amplitude of mass *i* expressed as a ratio of the vibration amplitude of one of the masses of the system, is known as modal shape coefficient

MOMENT RESISTING FRAME	A frame in which members and joints are capable of resisting lateral forces primarily by flexure. Moment resisting frames are classified as ordinary moment frames (OMF), intermediate moment frames (IMF) and special moment frames (SMF).
NOMINAL LOADS	The magnitudes of the loads such as dead, live, wind, earthquake etc. specified in Sections 2.2 to 2.6 of this Chapter.
NOMINAL STRENGTH	The capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modelling effects and differences between laboratory and field conditions.
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 ground floor deck or fitted between the building columns. But, it includes the basement storeys, when they are not so connected.
OPENINGS	Apertures or holes in the building envelope that allow air to flow through the building envelope and that are designed as "open" during design winds as defined by these provisions.
ORDINARY MOMENT FRAME (OMF)	A moment resisting frame not meeting special detailing requirements for ductile behaviour.
PERIOD OF BUILDING	Fundamental period (for 1st mode) of vibration of building for lateral motion in direction considered.
P-DELTA EFFECT	It is the secondary effect on shears and moments of frame members due to action of the vertical loads due to the lateral displacement of building resulting from seismic forces.
RATIONAL ANALYSIS	An analysis based on established methods or theories using mathematical formulae and actual or appropriately assumed data.

RECOGNIZED LITERATURE	Published research findings and technical papers that are approved.
RESISTANCE FACTOR	A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure. This is also known as strength reduction factor.
RESPONSE REDUCTION FACTOR	It is the factor by which the actual base shear force that would develop if the structure behaved truly elastic during earthquake, is reduced to obtain design base shear. This reduction is allowed to account for the beneficial effects of inelastic deformation (resulting in energy dissipation) that can occur in a structure during a major earthquake, still ensuring acceptable response of the structure.
RIDGE	With respect to topographic effects in Sec 2.4.7, an elongated crest of a hill characterized by strong relief in two directions (Figure 6.2.4).
SEISMIC DESIGN CATEGORY	A classification assigned to a structure based on its importance factor and the severity of the design earthquake ground motion at the site.
SEISMIC-FORCE-RESISTING SYSTEM	That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces.
SHEAR WALL	A wall designed to resist lateral forces acting in its plane (sometimes referred to as a vertical diaphragm or a structural wall).
SITE CLASS	Site is classified based on soil properties of upper 30 m.
SITE-SPECIFIC DATA	Data obtained either from measurements taken at a site or from substantiated field information required specifically for the structure concerned.
SOFT STOREY	Storey in which the lateral stiffness is less than 70 percent of the stiffness of the storey above or less than 80 percent of the average lateral stiffness of the three storeys above.
SPACE FRAME	A three-dimensional structural system without bearing walls composed of members interconnected so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

SPECIAL MOMENT FRAME (SMF)	A moment resisting frame specially detailed to provide ductile behaviour complying with the seismic requirements provided in Chapters 8 and 10 for concrete and steel frames respectively.
STOREY	The space between consecutive floor levels. Storey-x is the storey below level-x.
STOREY DRIFT	The horizontal deflection at the top of the story relative to bottom of the storey.
STOREY SHEAR	The total horizontal shear force at a particular storey (level).
STRENGTH	The usable capacity of an element or a member to resist the load as prescribed in these provisions.
STRENGTH DESIGN METHOD	A method of proportioning structural members using load factors and resistance factors satisfying both the applicable limit state conditions. This is also known as Load Factor Design Method (LFD) or Ultimate Strength Design Method (USD).
TARGET DISPLACEMENT	An estimate of the maximum expected displacement of the control point calculated for the design earthquake ground motion in nonlinear static analysis.
VERTICAL LOAD-CARRYING FRAME	A space frame designed to carry all vertical gravity loads.
WEAK STOREY	Storey in which the lateral strength is less than 80 percent of that of the storey above.
WIND-BORNE DEBRIS REGIONS	Areas within cyclone prone regions located: 1. Within 1.6 km of the coastal mean high water line where the basic wind speed is equal to or greater than 180 km/h or 2. In areas where the basic wind speed is equal to or greater than 200 km/h.
WORKING STRESS DESIGN METHOD (WSD)	See ALLOWABLE STRESS DESIGN METHOD.

2.1.4 Symbols and Notation

The following symbols and notation apply only to the provisions of this Chapter:

A	=	Effective wind area, in m^2
A_f	=	Area of open buildings and other structures either normal to the wind direction or projected on a plane normal to the wind direction, in m^2.
A_g	=	Gross area of that wall in which A_o is identified, in m^2.
A_{gi}	=	Sum of gross surface areas of the building envelope (walls and roof) not including A_g, in m^2
A_o	=	Total area of openings in a wall that receives positive external pressure, in m^2.
A_{oi}	=	Sum of the areas of openings in the building envelope (walls and roof) not including A_o, in m^2
A_{og}	=	Total area of openings in the building envelope in m^2
A_s	=	Gross area of the solid freestanding wall or solid sign, in m^2
A_x	=	Torsion amplification factor at level-x.
B	=	Horizontal dimension of building measured normal to wind direction, in m.
C_d	=	Deflection amplification factor.
C_f	=	Force coefficient to be used in determination of wind loads for other structures
C_N	=	Net pressure coefficient to be used in determination of wind loads for open buildings
C_p	=	External pressure coefficient to be used in determination of wind loads for buildings
C_s	=	Normalized acceleration response spectrum.
C_t	=	Numerical coefficient to determine building period
D	=	Diameter of a circular structure or member in m (as used in Sec 2.4).
D	=	Dead loads, or related internal moments and forces, Dead load consists of: a) weight of the member itself, b) weight of all materials of construction incorporated into the building to be permanently supported by the member, including built-in partitions, c) weight of permanent equipment (as used in Sec 2.7).

D'	=	Depth of protruding elements such as ribs and spoilers in m.
E	=	Total load effects of earthquake that include both horizontal and vertical, or related internal moments and forces. The horizontal seismic load effect shall include system overstrength factor, Ω_o, if applicable. For specific definition of the earthquake load effect, E, see Sec 2.5.
E_h	=	Horizontal seismic load effect when the effect of system overstrength factor, Ω_o, is not included.
E_{mh}	=	Horizontal seismic load effect when the effect of system overstrength factor, Ω_o, is included.
E_v	=	Vertical effect of seismic load.
F	=	Design wind force for other structures, in N (as used in Sec 2.4).
F	=	Loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights or related internal moments and forces (as used in Sec 2.7).
F_a	=	Loads due to flood or tidal surge or related internal moments and forces.
F_i, F_n, F_x	=	Design lateral force applied to level-i, -n, or -x respectively.
F_c	=	Lateral forces on an element or component or on equipment supports.
G	=	Gust effect factor
G_f	=	Gust effect factor for MWFRSs of flexible buildings and other structures
GC_p	=	Product of external pressure coefficient and gust effect factor to be used in determination of wind loads for buildings
GC_{pf}	=	Product of the equivalent external pressure coefficient and gust-effect factor to be used in determination of wind loads for MWFRS of low-rise buildings
GC_{pi}	=	Product of internal pressure coefficient and gust effect factor to be used in determination of wind loads for buildings
GC_{pn}	=	Combined net pressure coefficient for a parapet
H	=	Height of hill or escarpment in Figure 6.2.4 in m.

H = Loads due to weight and pressure of soil, water in soil, or other materials, or related internal moments and forces (as used in Sec 2.7)

I = Importance factor

I_z = Intensity of turbulence from Eq. 6.2.7

K_1, K_2, K_3 = Multipliers in Figure 6.2.4 to obtain K_{zt}

K_d = Wind directionality factor in Table 6.2.12

K_h = Velocity pressure exposure coefficient evaluated at height $z = h$

K_z = Velocity pressure exposure coefficient evaluated at height z

K_{zt} = Topographic factor as defined in Sec 2.4.7

L = Horizontal dimension of a building measured parallel to the wind direction, in m (as used in Sec 2.4)

L = Live loads due to intended use and occupancy, including loads due to movable objects and movable partitions and loads temporarily supported by the structure during maintenance, or related internal moments and forces, L includes any permissible reduction. If resistance to impact loads is taken into account in design, such effects shall be included with the live load L. (as used in Sec 2.7).

L_h = Distance upwind of crest of hill or escarpment in Figure 6.2.4 to where the difference in ground elevation is half the height of hill or escarpment, in m.

L_r = Roof live loads, or related internal moments and forces. (as used in Sec 2.7)

L_r = Horizontal dimension of return corner for a solid freestanding wall or solid sign from Figure 6.2.20, in m. (as used in Sec 2.4)

$L_{\check{z}}$ = Integral length scale of turbulence, in m.

Level-i = Floor level of the structure referred to by the subscript i, e.g., $i = 1$ designates the first level above the base.

Level- n = Uppermost level in the main portion of the structure.

M_x = Overturning moment at level-x

N_1 = Reduced frequency from Eq. 6.2.14

N_i = Standard Penetration Number of soil layer i

P_{net}	=	Net design wind pressure from Eq. 6.2.4, in N/m^2
P_{net30}	=	Net design wind pressure for Exposure A at h = 9.1 m and I = 1.0 from Figure 6.2.3, in N/m^2.
P_p	=	Combined net pressure on a parapet from Eq. 6.2.22, in N/m^2.
P_s	=	Net design wind pressure from Eq. 6.2.3, in N/m^2.
P_{s30}	=	Simplified design wind pressure for Exposure A at h = 9.1 m and I = 1.0 from Figure 6.2.2, in N/m^2.
P_x	=	Total vertical design load at level-x
P_w	=	Wind pressure acting on windward face in Figure 6.2.9, in N/m^2.
Q	=	Background response factor from Eq. 6.2.8
R	=	Resonant response factor from Eq. 6.2.12
R	=	Response reduction factor for structural systems. (as used in Sec 2.5)
R	=	Rain load, or related internal moments and forces. (as used in Sec 2.7)
R_B, R_h, R_L	=	Values from Eq. 6.2.15
R_i	=	Reduction factor from Eq. 6.2.18
R_n	=	Value from Eq. 6.2.13
S	=	Soil factor.
S_a	=	Design Spectral Acceleration (in units of g)
S_{ui}	=	Undrained shear strength of cohesive layer i
T	=	Fundamental period of vibration of structure, in seconds, of the structure in the direction under consideration. (as used in Sec 2.5)
T	=	Self-straining forces and cumulative effect of temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete, or combinations thereof, or related internal moments and forces. (as used in Sec 2.7)
T_e	=	Effective fundamental period of the structure in the direction under consideration, as determined for nonlinear static analysis
V	=	Basic wind speed obtained from Figure 6.2.1 or Table 6.2.8, in m/s. The basic wind speed corresponds to a 3-s gust speed at 10 m above ground in Exposure Category B having an annual probability of occurrence of 0.02.

V	=	Total design base shear calculated by equivalent static analysis. (as used in Sec 2.5)
V_i	=	Unpartitioned internal volume m^3
$\bar{V}_{\bar{z}}$	=	mean hourly wind speed at height $\bar{z}$, m/s.
V_1	=	Total applied lateral force at the first increment of lateral load in nonlinear static analysis.
V_y	=	Effective yield strength determined from a bilinear curve fitted to the capacity curve
V_{rs}	=	Total design base shear calculated by response spectrum analysis
V_{th}	=	Total design base shear calculated by time history analysis
V_{si}	=	Shear wave velocity of soil layer i
V_x	=	Design storey shear in storey x
W	=	Width of building in Figures 6.2.12, 6.2.14(a) and 6.2.14(b), and width of span in Figures 6.2.13 and 6.2.15 in m.
W	=	Total seismic weight of building. (as used in Sec 2.5)
W	=	Wind load, or related internal moments and forces. (as used in Sec 2.7)
X	=	Distance to center of pressure from windward edge in Figure 6.2.18, in m.
Z	=	Seismic zone coefficient.
a	=	Width of pressure coefficient zone, in m.
b	=	Mean hourly wind speed factor in Eq. 6.2.16 from Table 6.2.10
$\hat{b}$	=	3-s gust speed factor from Table 6.2.10
c	=	Turbulence intensity factor in Eq. 6.2.7 from Table 6.2.10
e_{ai}	=	Accidental eccentricity of floor mass at level-i
g	=	Acceleration due to gravity.
g_Q	=	Peak factor for background response in Equations 6.2.6 and 6.2.10
g_R	=	Peak factor for resonant response in Eq. 6.2.10
g_V	=	Peak factor for wind response in Equations 6.2.6 and 6.2.10

h	=	Mean roof height of a building or height of other structure, except that eave height shall be used for roof angle θ of less than or equal to 10°, in m.
h_e	=	Roof eave height at a particular wall, or the average height if the eave varies along the wall
h_i, h_n, h_x	=	Height in metres above the base to level i,-n or -x respectively
h_{sx}	=	Storey Height of storey x (below level- x)
l	=	Integral length scale factor from Table 6.2.10 in m.
n_1	=	Building natural frequency, Hz
p	=	Design pressure to be used in determination of wind loads for buildings, in N/m^2
p_L	=	Wind pressure acting on leeward face in Figure 6.2.9, in N/m^2
q	=	Velocity pressure, in N/m^2.
q_h	=	Velocity pressure evaluated at height $z = h$, in N/m^2
q_i	=	Velocity pressure for internal pressure determination, in N/m^2.
q_p	=	Velocity pressure at top of parapet, in N/m^2.
q_z	=	Velocity pressure evaluated at height z above ground, in N/m^2.
r	=	Rise-to-span ratio for arched roofs.
s	=	Vertical dimension of the solid freestanding wall or solid sign from Figure 6.2.20, in m.
w_i, w_x	=	Portion of W which is assigned to level i and x respectively
x	=	Distance upwind or downwind of crest in Figure 6.2.4, in m.
z	=	Height above ground level, in m.
$\bar{z}$	=	Equivalent height of structure, in m.
z_g	=	Nominal height of the atmospheric boundary layer used in this standard. Values appear in Table 6.2.10
z_{min}	=	Exposure constant from Table 6.2.10
Δ_a	=	Maximum allowable storey drift
Δ_x	=	Design storey drift of storey x

$\in$	=	Ratio of solid area to gross area for solid freestanding wall, solid sign, open sign, face of a trussed tower, or lattice structure
$\bar{\in}$	=	Integral length scale power law exponent in Eq. 6.2.9 from Table 6.2.10
Ω_o	=	Horizontal seismic overstrength factor from Table 6.2.19
α	=	3-s gust-speed power law exponent from Table 6.2.10
$\hat{\alpha}$	=	Reciprocal of α from Table 6.2.10
$\bar{\alpha}$	=	Mean hourly wind-speed power law exponent in Eq. 6.2.16 from Table 6.2.10
β	=	Damping ratio, percent critical for buildings or other structures
δ_i	=	Horizontal displacement at level-i relative to the base due to applied lateral forces.
δ_j	=	The displacement of the control point at load increment j.
δ_T	=	The target displacement of the control point.
δ_1	=	The displacement of the control point at the first increment of lateral load.
δ_y	=	The effective yield displacement of the control point determined from a bilinear curve fitted to the capacity curve
η	=	Value used in Eq. 6.2.15 (see Sec 2.4.8.2)
η	=	Damping correction factor
θ	=	Angle of plane of roof from horizontal, in degrees. (as used in Sec 2.4)
θ	=	Stability coefficient to assess P-delta effects. (as used in Sec 2.5)
λ	=	Adjustment factor for building height and exposure from Figures 6.2.2 and 6.2.3
ν	=	Height-to-width ratio for solid sign
ξ	=	Viscous damping ratio of the structure
ϕ_{ik}	=	Modal shape coefficient at level i for mode k

2.2 Dead Loads

2.2.1 General

The minimum design dead load for buildings and portions thereof shall be determined in accordance with the provisions of this Section. In addition, design of the overall structure and its primary load-resisting systems shall conform to the general design provisions given in Chapter 1.

2.2.2 Definition

Dead Load is the vertical load due to the weight of permanent structural and non-structural components and attachments of a building such as walls, floors, ceilings, permanent partitions and fixed service equipment etc.

2.2.3 Assessment of Dead Load

Dead load for a structural member shall be assessed based on the forces due to:

- weight of the member itself,
- weight of all materials of construction incorporated into the building to be supported permanently by the member,
- weight of permanent partitions,
- weight of fixed service equipment, and
- net effect of prestressing.

2.2.4 Weight of Materials and Constructions

In estimating dead loads, the actual weights of materials and constructions shall be used, provided that in the absence of definite information, the weights given in Tables 6.2.1 and 6.2.2 shall be assumed for the purposes of design.

Table 6.2.1: Unit Weight of Basic Materials

Material	Unit Weight (kN/m^3)	Material	Unit Weight (kN/m^3)
Aluminium	27.0	Granite, Basalt	26.4
Asphalt	21.2	Iron - cast	70.7
Brass	83.6	- wrought	75.4
Bronze	87.7	Lead	111.0
Brick	18.9	Limestone	24.5
Cement	14.7	Marble	26.4
Coal, loose	8.8	Sand, dry	15.7

Material	Unit Weight (kN/m^3)	Material	Unit Weight (kN/m^3)
Concrete -stone aggregate (unreinforced)	22.8*	Sandstone	22.6
-brick aggregate (unreinforced)	20.4*	Slate	28.3
Copper	86.4	Steel	77.0
Cork, normal	1.7	Stainless Steel	78.75
Cork, compressed	3.7	Timber	5.9-11.0
Glass, window (soda-lime)	25.5	Zinc	70.0

* for reinforced concrete, add 0.63 kN/m^3 for each 1% by volume of main reinforcement

Table 6.2.2: Weight of Construction Materials.

Material/Component/Member	Veight per Unit Area (kN/m^2)	Material/Component/Member	Weight per Unit Area (kN/m^2)
Floor		**Walls and Partitions**	
Asphalt, 25 mm thick	0.526	Acrylic resin sheet, flat, per mm thickness	0.012
Clay tiling, 13 mm thick	0.268		
Concrete slab (stone aggregate)*:		Asbestos cement sheeting:	
solid, 100 mm thick	2.360	4.5 mm thick	0.072
solid, 150 mm thick	3.540	6.0 mm thick	0.106
Galvanized steel floor deck (excl. topping)	0.147-0.383	Brick masonry work, excl. plaster:	
Magnesium oxychloride:		burnt clay, per 100 mm thickness	1.910
normal (sawdust filler), 25 mm thick	0.345	sand-lime, per 100 mm thickness	1.980
heavy duty (mineral filler), 25 mm thick	0.527	Concrete (stone aggregate)*:	
Terrazzo paving 16 mm thick	0.431	100 mm thick	2.360

Material/Component/Member	Weight per Unit Area (kN/m^2)
Roof	
Acrylic resin sheet, corrugated:	
3 mm thick, standard corrugations	0.043
3 mm thick, deep corrugations	0.062
Aluminium, corrugated sheeting:	
(incl. lap and fastenings)	
1.2 mm thick	0.048
0.8 mm thick	0.028
0.6 mm thick	0.024
Aluminium sheet(plain):	
1.2 mm thick	0.033
1.0 mm thick	0.024
0.8 mm thick	0.019
Bituminous felt (5 ply) and gravel	0.431
Slates:	
4.7 mm thick	0.335
9.5 mm thick	0.671
Steel sheet, flat galvanized:	
1.00 mm thick	0.082
0.80 mm thick	0.067
0.60 mm thick	0.053
Steel, galvanized std. corrugated sheeting:	
(incl. lap and fastenings)	
1.0 mm thick	0.120
0.8 mm thick	0.096
0.6 mm thick	0.077
Tiles :	
terra-cotta tiles (French pattern)	0.575
concrete , 25 mm thick	0.527
clay tiles	0.6-0.9
150 mm thick	3.540
250 mm thick	5.900
Fibre insulation board, per 10 mm thickness	0.034
Fibrous plaster board, per 10 mm thickness	0.092
Glass, per 10 mm thickness	0.269
Hardboard, per 10 mm thickness	0.961
Particle or flake board, per 10 mm thickness	0.075
Plaster board, per 10 mm thickness	0.092
Plywood, per 10 mm thickness	0.061
Ceiling	
Fibrous plaster, 10 mm thick	0.081
Cement plaster, 13 mm thick	0.287
Suspended metal lath and plaster	0.480
(two faced incl. studding)	
Miscellaneous	
Felt (insulating), per 10 mm thickness	0.019
Plaster:	
Cement plaster, per 10 mm thickness	0.230
Lime plaster, per 10 mm thickness	0.191
PVC sheet, per 10 mm thickness	0.153
Rubber paving, per 10 mm thickness	0.151
Terra-cotta Hollow Block Masonry:	
75 mm thick	0.671
100 mm thick	0.995
150 mm thick	1.388

* For brick aggregate, 90% of the listed values may be used.

2.2.5 Weight of Permanent Partitions

When partition walls are indicated on the plans, their weight shall be considered as dead load acting as concentrated line loads in their actual positions on the floor. The loads due to anticipated partition walls, which are not indicated on the plans, shall be treated as live loads and determined in accordance with Sec 2.3.6.

2.2.6 Weight of Fixed Service Equipment

Weights of fixed service equipment and other permanent machinery, such as electrical feeders and other machinery, heating, ventilating and air-conditioning systems, lifts and escalators, plumbing stacks and risers etc. shall be included as dead load whenever such equipment are supported by structural members.

2.2.7 Additional Loads

In evaluating the final dead loads on a structural member for design purposes, allowances shall be made for additional loads resulting from the (i) difference between the prescribed and the actual weights of the members and construction materials; (ii) inclusion of future installations; (iii) changes in occupancy or use of buildings; and (iv) inclusion of structural and non-structural members not covered in Sections 2.2.2 and 2.2.3.

2.3 Live Loads

2.3.1 General

The live loads used for the structural design of floors, roof and the supporting members shall be the greatest applied loads arising from the intended use or occupancy of the building, or from the stacking of materials and the use of equipment and propping during construction, but shall not be less than the minimum design live loads set out by the provisions of this Section. For the design of structural members for forces including live loads, requirements of the relevant Sections of Chapter 1 shall also be fulfilled.

2.3.2 Definition

Live load is the load superimposed by the use or occupancy of the building not including the environmental loads such as wind load, rain load, earthquake load or dead load.

2.3.3 Minimum Floor Live Loads

The minimum floor live loads shall be the greatest actual imposed loads resulting from the intended use or occupancy of the floor, and shall not be less than the uniformly distributed load patterns specified in Sec 2.3.4 or the concentrated loads specified in Sec 2.3.5 whichever produces the most critical effect. The live loads shall be assumed to act vertically upon the area projected on a horizontal plane.

2.3.4 Uniformly Distributed Loads

The uniformly distributed live load shall not be less than the values listed in Table 6.2.3, reduced as may be specified in Sec 2.3.13, applied uniformly over the entire area of the floor, or any portion thereof to produce the most adverse effects in the member concerned.

2.3.5 Concentrated Loads

The concentrated load to be applied non-concurrently with the uniformly distributed load given in Sec 2.3.4, shall not be less than that listed in Table 6.2.3. Unless otherwise specified in Table 6.2.3 or in the following paragraph, the concentrated load shall be applied over an area of 300 mm × 300 mm and shall be located so as to produce the maximum stress conditions in the structural members.

In areas where vehicles are used or stored, such as car parking garages, ramps, repair shops etc., provision shall be made for concentrated loads consisting of two or more loads spaced nominally 1.5 m on centres in absence of the uniform live loads. Each load shall be 40 percent of the gross weight of the maximum size vehicle to be accommodated and applied over an area of 750 mm × 750 mm. For the storage of private or pleasure-type vehicles without repair or fuelling, floors shall be investigated in the absence of the uniform live load, for a minimum concentrated wheel load of 9 kN spaced 1.5 m on centres, applied over an area of 750 mm × 750 mm. The uniform live loads for these cases are provided in Table 6.2.3. The condition of concentrated or uniform live load producing the greater stresses shall govern.

Table 6.2.3: Minimum Uniformly Distributed and Concentrated Live Loads[a]

Occupancy or Use	Uniform kN/m^2	Concentrated kN
Apartments (see Residential)		
Access floor systems		
Office use	2.40	9.0
Computer use	4.80	9.0
Armories and drill rooms	7.20	--

Occupancy or Use	**Uniform kN/m²**	**Concentrated kN**
Assembly areas and theaters		
Fixed seats (fastened to floor)	2.90	--
Lobbies	4.80	--
Movable seats	4.80	--
Platforms (assembly)	4.80	--
Stage floors	7.20	--
Balconies (exterior)	4.80	--
On one- and two-family residences only, and not exceeding 19.3 m^2	2.90	--
Bowling alleys, poolrooms, and similar recreational areas	3.60	--
Catwalks for maintenance access	2.00	1.33
Corridors		
First floor	4.80	--
Other floors, same as occupancy served except as indicated		
Dance halls and ballrooms	4.80	--
Decks (patio and roof)	Same as area served, or for the type of occupancy accommodated	
Dining rooms and restaurants	4.80	--
Dwellings (see *Residential*)	--	
Elevator machine room grating (on area of 2,580 mm^2)	--	1.33
Finish light floor plate construction (on area of 645 mm^2)	--	0.90
Fire escapes	4.80	--
On single-family dwellings only	2.00	--
Fixed ladders	See Sec 2.3.11	
Garages (passenger vehicles only), Trucks and buses	$2.0^{b,c}$	
Grandstands	See *Stadiums and arenas, Bleachers*	
Gymnasiums—main floors and balconies	4.80	--
Handrails, guardrails, and grab bars	See Sec 2.3.11	
Hospitals		
Operating rooms, laboratories	2.90	4.50
Patient rooms	2.00	4.50
Corridors above first floor	3.80	4.50

Occupancy or Use	**Uniform kN/m²**	**Concentrated kN**
Hotels	See *Residential*	
Libraries		
Reading rooms	2.90	4.50
Stack rooms	7.20 [d]	4.50
Corridors above first floor	3.80	4.50
Manufacturing*		
Light	4.00	6.00
Medium	6.00	9.00
Heavy	12.00	13.40
Garments manufacturing floor except stacking or storage area	4.00 [e]	--
Stacking or storage area of garments manufacturing industry	6.00 [f]	10.00 [f]
Marquees	3.60	--
Office Buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first-floor corridors	4.80	9.00
Offices	2.40	9.00
Corridors above first floor	3.80	9.00
Penal Institutions		
Cell blocks	2.00	--
Corridors	4.80	--
Residential		
Dwellings (one- and two-family)		
Uninhabitable attics without storage	0.50	--
Uninhabitable attics with storage	1.00	--
Habitable attics and sleeping areas	1.50	--
All other areas except stairs and balconies	2.00	--
Hotels and multifamily houses		
Private rooms and corridors serving them	2.00	--
Public rooms and corridors serving them	4.80	--
Reviewing stands, grandstands, and bleachers	4.80 [g]	--

Occupancy or Use	**Uniform kN/m²**	**Concentrated kN**
Roofs		
Ordinary flat roof	1.00 [h]	--
Pitched and curved roofs	See Table 6.2.4	
Roofs used for promenade purposes	2.90	--
Roofs used for roof gardens or assembly purposes	4.80	--
Roofs used for other special purposes	See Note [i] below	
Awnings and canopies		
Fabric construction supported by a lightweight rigid skeleton structure	0.24 (nonreduceable)	--
All other construction	1.00	--
Primary roof members exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages	--	9.00
All other occupancies	--	1.33
All roof surfaces subject to maintenance workers	--	1.33
Schools		
Classrooms	2.00	4.50
Corridors above first floor	3.80	4.50
First-floor corridors	4.80	4.50
Scuttles, skylight ribs, and accessible ceilings		0.90
Sidewalks, vehicular driveways, and yards subject to trucking	12.00 [j]	35.60 [k]
Stadiums and arenas		
Bleachers	4.80 [g]	--
Fixed seats (fastened to floor)	2.90 [g]	--
Stairs and exit ways	4.80	See Note [l] below
One- and two-family residences only	2.00	--

Occupancy or Use	**Uniform kN/m²**	**Concentrated kN**
Storage areas above ceilings	1.00	--
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	6.00	--
Heavy	12.00	--
Stores		
Retail		
First floor	4.80	4.50
Upper floors	3.60	4.50
Wholesale, all floors	6.00	4.50
Vehicle barriers	See Sec 2.3.11	
Walkways and elevated platforms (other than exit ways)	2.90	--
Yards and terraces, pedestrian	4.80	--

Notes:

a It must be ensured that the average weight of equipment, machinery, raw materials and products that may occupy the floor is less than the specified value in the Table. In case the weight exceeds the specified values in the Table, actual maximum probable weight acting in the actual manner shall be used in the analysis and design.

b Floors in garages or portions of a building used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 6.2.3 or the following concentrated load: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 13.35 kN acting on an area of 114 mm by 114 mm footprint of a jack; and (2) for mechanical parking structures without slab or deck that are used for storing passenger car only, 10 kN per wheel.

c Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provisions for truck and bus loadings.

d The loading applies to stack room floors that support non-mobile, double-faced library book stacks subject to the following limitations: (1) The nominal book stack unit height shall not exceed 2290 mm; (2) the nominal shelf depth shall not exceed 300 mm for each face; (3) parallel rows of double-faced book stacks shall be separated by aisles not less than 900 mm wide.

[e] Subject to the provisions of reduction of live load as per Sec 2.3.13

[f] Uniformly distributed and concentrated load provisions are applicable for a maximum floor height of 3.5 m. In case of higher floor height, the load(s) must be proportionally increased.

[g] In addition to the vertical live loads, the design shall include horizontal swaying forces applied to each row of the seats as follows: 0.350 kN per linear meter of seat applied in a direction parallel to each row of seats and 0.15 kN per linear meter of seat applied in a direction perpendicular to each row of seats. The parallel and perpendicular horizontal swaying forces need not be applied simultaneously.

[h] Where uniform roof live loads are reduced to less than 1.0 kN/m^2 in accordance with Sec 2.3.14.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the greatest unfavorable effect.

[i] Roofs used for other special purposes shall be designed for appropriate loads as approved by the authority having jurisdiction.

[j] Other uniform loads in accordance with an approved method, which contains provisions for truck loadings, shall also be considered where appropriate.

[k] The concentrated wheel load shall be applied on an area of 114 mm by 114 mm footprint of a jack.

[l] Minimum concentrated load on stair treads (on area of 2,580 mm^2) is 1.33 kN.

* The loading in industrial buildings varies considerably and so the loadings under the terms ‘light,’ ‘medium’ and ‘heavy’ are introduced in order to allow for which the relevant floor is designed. It is however important to assess the actual loads to ensure that they are not in excess of the stipulated load, in case where they are in excess, the design shall be based on the actual loadings.

2.3.6 Provision for Partition Walls

When partitions, not indicated on the plans, are anticipated to be placed on the floors, their weight shall be included as an additional live load acting as concentrated line loads in an arrangement producing the most severe effect on the floor, unless it can be shown that a more favourable arrangement of the partitions shall prevail during the future use of the floor.

In the case of light partitions, wherein the total weight per metre run is not greater than 5.5 kN, a uniformly distributed live load may be applied on the floor in lieu of the concentrated line loads specified above. Such uniform live load per square metre shall be at least 33% of the weight per metre run of the partitions, subject to a minimum of 1.2 kN/m^2.

2.3.7 More than One Occupancy

Where an area of a floor is intended for two or more occupancies at different times, the value to be used from Table 6.2.3 shall be the greatest value for any of the occupancies concerned.

2.3.8 Minimum Roof Live Loads

Roof live loads shall be assumed to act vertically over the area projected by the roof or any portion of it upon a horizontal plane, and shall be determined as specified in Table 6.2.4.

Table 6.2.4: Minimum Roof Live Loads([1])

Type and Slope of Roof		Distributed Load, kN/m^2	Concentrated Load, kN
I	Flat roof (slope = 0)	See Table 6.2.3	
II	(A) Pitched or sloped roof ($0 < \text{slope} < 1/3$)	1.0	0.9
	(B) Arched roof or dome (rise < 1/8 span)		
III	(A) Pitched or sloped roof ($1/3 \le \text{slope} < 1.0$)	0.8	0.9
	(B) Arched roof or dome ($1/8 \le$ rise < 3/8 span)		
IV	(A) Pitched or sloped roof (slope $\ge$ 1.0)	0.6	0.9
	(B) Arched roof or dome (rise $\ge$ 3/8 span)		
V	Greenhouse, and agriculture buildings	0.5	0.9
VI	Canopies and awnings, except those with cloth covers	Same as given in I to IV above based on the type and slope.	

Note: [(1)] Greater of this load and rain load as specified in Sec 2.6.2 shall be taken as the design live load for roof. The distributed load shall be applied over the area of the roof projected upon a horizontal plane and shall not be applied simultaneously with the concentrated load. The concentrated load shall be assumed to act upon a 300 mm × 300 mm area and need not be considered for roofs capable of laterally distributing the load, e.g. reinforced concrete slabs.

2.3.9 Loads not Specified

Live loads, not specified for uses or occupancies in Sections 2.3.3, 2.3.4 and 2.3.5, shall be determined from loads resulting from:

(a) weight of the probable assembly of persons;

(b) weight of the probable accumulation of equipment and furniture, and

(c) weight of the probable storage of materials.

2.3.10 Partial Loading and Other Loading Arrangements

The full intensity of the appropriately reduced live load applied only to a portion of the length or area of a structure or member shall be considered, if it produces a more unfavourable effect than the same intensity applied over the full length or area of the structure or member.

Where uniformly distributed live loads are used in the design of continuous members and their supports, consideration shall be given to full dead load on all spans in combination with full live loads on adjacent spans and on alternate spans whichever produces a more unfavourable effect.

2.3.11 Other Live Loads

Live loads on miscellaneous structures and components, such as handrails and supporting members, parapets and balustrades, ceilings, skylights and supports, and the like, shall be determined from the analysis of the actual loads on them, but shall not be less than those given in Table 6.2.5.

2.3.12 Impact and Dynamic Loads

The live loads specified in Sec 2.3.3 shall be assumed to include allowances for impacts arising from normal uses only. However, forces imposed by unusual vibrations and impacts resulting from the operation of installed machinery and equipment shall be determined separately and treated as additional live loads. Live loads due to vibration or impact shall be determined by dynamic analysis of the supporting member or structure including foundations, or from the recommended values supplied by the manufacture of the particular equipment or machinery. In absence of definite information, values listed in Table 6.2.6 for some common equipment, shall be used for design purposes.

Table 6.2.5: Miscellaneous Live Loads

Structural Member or Component	Live Load[1] (kN/m)
A. Handrails, parapets and supports:	
(a) Light access stairs, gangways etc.	
(i) width $\leq$ 0.6 m	0.25
(ii) width > 0.6 m	0.35
(b) Staircases other than in (a) above, ramps, balconies:	
(i) Single dwelling and private	0.35
(ii) Staircases in residential buildings	0.35
(iii) Balconies or portion thereof, stands etc. having fixed seats within 0.55 m of the barrier	1.5
(iv) Public assembly buildings including theatres, cinemas, assembly halls, stadiums, mosques, churches, schools etc.	3.0
(v) Buildings and occupancies other than (i) to (iv) above	0.75

B. Vehicle barriers for car parks and ramps:	
(a) For vehicles having gross mass ≤ 2500 kg	100[2]
(b) For vehicles having gross mass > 2500 kg	165[2]
(c) For ramps of car parks etc.	see note[3]

Notes: (1) These loads shall be applied non-concurrently along horizontal and vertical directions, except as specified in note (2) below.

(2) These loads shall be applied only in the horizontal direction, uniformly distributed over any length of 1.5 m of a barrier and shall be considered to act at bumper height. For case 2(a) bumper height may be taken as 375 mm above floor level.

(3) Barriers to access ramps of car parks shall be designed for horizontal forces equal to 50% of those given in 2(a) and 2(b) applied at a level of 610 mm above the ramp. Barriers to straight exit ramps exceeding 20 m in length shall be designed for horizontal forces equal to twice the values given in 2(a) and 2(b).

Table 6.2.6: Minimum Live Loads on Supports and Connections of Equipment due to Impact[1]

Equipment or Machinery	Additional load due to impact as percentage of static load including self-weight	
	Vertical	**Horizontal**
1. Lifts, hoists and related operating machinery	100%	Not applicable
2. Light machinery (shaft or motor driven)	20%	Not applicable
3. Reciprocating machinery, or power driven units.	50%	Not applicable
4. Hangers supporting floors and balconies	33%	Not applicable
5. Cranes :		(i) Transverse to the rail :
(a) Electric overhead cranes	25% of maximum wheel load	20% of the weight of trolley and lifted load only, applied one-half at the top of each rail
		(ii) Along the rail :
		10% of maximum wheel load applied at the top of each rail

Equipment or Machinery	Additional load due to impact as percentage of static load including self-weight	
	Vertical	Horizontal
(b) Manually operated cranes	50% of the values in (a) above	50% of the values in (a) above
(c) Cab-operated travelling cranes	25%	Not applicable

[1] All these loads shall be increased if so recommended by the manufacturer. For machinery and equipment not listed, impact loads shall be those recommended by the manufacturers, or determined by dynamic analysis.

2.3.13 Reduction of Live Loads

Except for roof uniform live loads, all other minimum uniformly distributed live loads, L_o in Table 6.2.3, may be reduced according to the following provisions.

2.3.13.1 General

Subject to the limitations of Sections 2.3.13.2 to 2.3.13.5, members for which a value of $K_{LL}A_T$ is 37.16 m^2 or more are permitted to be designed for a reduced live load in accordance with the following formula:

$$L = L_o \left(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}\right) \qquad (6.2.1)$$

Where, L = reduced design live load per m^2 of area supported by the member; L_0= unreduced design live load per m^2 of area supported by the member (Table 6.2.3); K_{LL}= live load element factor (Table 6.2.7); A_T= tributary area in m^2, L shall not be less than $0.50L_0$ for members supporting one floor and L shall not be less than $0.40L_0$ for members supporting two or more floors.

Table 6.2.7: Live Load Element Factor, K_{LL}

Element	K_{LL}*
Interior columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2

All other members not identified including: • Edge beams with cantilever slabs • Cantilever beams • One-way slabs • Two-way slabs • Members without provisions for continuous shear transfer normal to their span	1

*In lieu of the preceding values, K_{LL} is permitted to be calculated.

2.3.13.2 Heavy live loads

Live loads that exceed 4.80 kN/m^2 shall not be reduced.

Exception: Live loads for members supporting two or more floors may be reduced by 20 percent.

2.3.13.3 Passenger car garages

The live loads shall not be reduced in passenger car garages.

Exception: Live loads for members supporting two or more floors may be reduced by 20 percent.

2.3.13.4 Special occupancies

(a) Live loads of 4.80 kN/m^2 or less shall not be reduced in public assembly occupancies.

(b) There shall be no reduction of live loads for cyclone shelters.

2.3.13.5 Limitations on one-way slabs

The tributary area, A_T, for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span.

2.3.14 Reduction in Roof Live Loads

The minimum uniformly distributed roof live loads, L_o in Table 6.2.3, are permitted to be reduced according to the following provisions.

2.3.14.1 Flat, pitched, and curved roofs.

Ordinary flat, pitched, and curved roofs are permitted to be designed for a reduced roof live load, as specified in Eq. 6.2.2 or other controlling combinations of loads, as discussed later in this Chapter, whichever produces the greater load. In structures such as greenhouses, where special scaffolding is used as a work surface for workmen and materials during maintenance and repair operations, a lower roof load than specified in Eq. 6.2.2 shall not be used unless approved by the authority having jurisdiction. On such structures, the minimum roof live load shall be 0.60 kN/m^2.

$$L_r = L_o R_1 R_2 \qquad (0.60 \le L_r \le 1.00) \tag{6.2.2}$$

Where,

L_r = reduced roof live load per m^2 of horizontal projection in kN/m^2

The reduction factors R_1 and R_2 shall be determined as follows:

$R_1 = 1$ for $A_t \le 18.58$ m^2

$= 1.2 - 0.011A_t$ for 18.58 m^2 $< A_t <$ 55.74 m^2

$= 0.6$ for $A_t \ge$ 55.74 m^2

A_t = tributary area in m^2 supported by any structural member and

$R_2 = 1$ for $F \le 4$

$= 1.2 - 0.05F$ for $4 < F < 12$

$= 0.6$ for $F \ge 12$

For a pitched roof, $F = 0.12 \times$ slope, with slope expressed in percentage points and, for an arch or dome, F = rise-to-span ratio multiplied by 32.

2.3.14.2 Special purpose roofs.

Roofs that have an occupancy function, such as roof gardens, assembly purposes, or other special purposes are permitted to have their uniformly distributed live load reduced in accordance with the requirements of Sec 2.3.13.

2.4 Wind Loads

2.4.1 General

Scope: Buildings and other structures, including the Main Wind-Force Resisting System (MWFRS) and all components and cladding thereof, shall be designed and constructed to resist wind loads as specified herein.

Allowed Procedures: The design wind loads for buildings and other structures, including the MWFRS and component and cladding elements thereof, shall be determined using one of the following procedures:

Method 1: Simplified Procedure as specified in Sec 2.4.2 for buildings and structures meeting the requirements specified therein;

Method 2: Analytical Procedure as specified in Sec 2.4.3 for buildings and structures meeting the requirements specified therein;

Method 3: Wind Tunnel Procedure as specified in Sec 2.4.16.

Wind Pressures Acting on opposite faces of each building surface. In the calculation of design wind loads for the MWFRS and for components and cladding for buildings, the algebraic sum of the pressures acting on opposite faces of each building surface shall be taken into account.

Minimum Design Wind Loading

The design wind load, determined by any one of the procedures specified in Sec 2.4.1, shall be not less than specified in this Section.

Main Wind-Force Resisting System: The wind load to be used in the design of the MWFRS for an enclosed or partially enclosed building or other structure shall not be less than 0.5 kN/m^2 multiplied by the area of the building or structure projected onto a vertical plane normal to the assumed wind direction. The design wind force for open buildings and other structures shall be not less than 0.5 kN/m^2 multiplied by the area A_f.

Components and Cladding**:** The design wind pressure for components and cladding of buildings shall not be less than a net pressure of 0.5 kN/m^2 acting in either direction normal to the surface.

2.4.2 Method 1: Simplified Procedure

2.4.2.1 Scope

A building whose design wind loads are determined in accordance with this Section shall meet all the conditions of Sec 2.4.2.2 or Sec 2.4.2.3. If a building qualifies only under Sec 2.4.2.3 for design of its components and cladding, then its MWFRS shall be designed by Method 2 or Method 3.

Limitations on Wind Speeds: Variation of basic wind speeds with direction shall not be permitted unless substantiated by any established analytical method or wind tunnel testing.

2.4.2.2 Main wind-force resisting systems

For the design of MWFRSs the building must meet all of the following conditions:

(1) The building is a simple diaphragm building as defined in Sec 2.1.3.

(2) The building is a low-rise building as defined in Sec 2.1.3.

(3) The building is enclosed as defined in Sec 2.1.3 and conforms to the wind-borne debris provisions of Sec 2.4.9.3.

(4) The building is a regular-shaped building or structure as defined in Sec 2.1.3.

(5) The building is not classified as a flexible building as defined in Sec 2.1.3.

(6) The building does not have response characteristics making it subject to a cross wind loading, vortex shedding, instability due to galloping or flutter; and does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

(7) The building has an approximately symmetrical cross-section in each direction with either a flat roof or a gable or hip roof with $\theta \leq 45^o$.

(8) The building is exempted from torsional load cases as indicated in Note 5 of Figure 6.2.10, or the torsional load cases defined in Note 5 do not control the design of any of the MWFRSs of the building.

2.4.2.3 Components and cladding

For the design of components and cladding the building must meet all the following conditions:

(1) The mean roof height h must be less than or equal to 18.3 m ($h \leq 18.3$ m).

(2) The building is enclosed as defined in Sec 2.1.3 and conforms to wind-borne debris provisions of Sec 2.4.9.3.

(3) The building is a regular-shaped building or structure as defined in Sec 2.1.3.

(4) The building does not have response characteristics making it subject to across-wind loading, vortex shedding, instability due to galloping or flutter; and does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

(5) The building has either a flat roof, a gable roof with $\theta \leq 45^o$, or a hip roof with $\theta \leq 27^o$.

2.4.2.4 Design procedure

(1) The basic wind speed V shall be determined in accordance with Sec 2.4.4. The wind shall be assumed to come from any horizontal direction.

(2) An importance factor I shall be determined in accordance with Sec 2.4.5.

(3) An exposure category shall be determined in accordance with Sec 2.4.6.3.

(4) A height and exposure adjustment coefficient, λ shall be determined from Figure 6.2.2.

2.4.2.4.1 Main wind-force resisting system: Simplified design wind pressures, p_s, for the MWFRSs of low-rise simple diaphragm buildings represent the net pressures (sum of internal and external) to be applied to the horizontal and vertical projections of building surfaces as shown in Figure 6.2.2. For the horizontal pressures (zones A, B, C, D), p_s is the combination of the windward and leeward net pressures. p_s shall be determined by the following equation:

$$p_s = \lambda K_{zt} I p_{s30} \qquad (6.2.3)$$

Where,

λ = adjustment factor for building height and exposure from Figure 6.2.2

K_{zt} = topographic factor as defined in Sec 2.4.7 evaluated at mean roof height, h

I = importance factor as defined in Sec 2.4.5

p_{s30} = simplified design wind pressure for Exposure A, at $h = 9.1$ m, and for $I = 1.0$, refer to Figure 6-2 of ASCE 7-05.

Minimum Pressures: The load effects of the design wind pressures from this Section shall not be less than the minimum load case from Sec 2.4.2.1 assuming the pressures, p_s, for zones A, B, C, and D all equal to + 0.5 kN/m^2, while assuming zones E, F, G, and H all equal to zero kN/m^2.

2.4.2.4.2 Components and cladding: Net design wind pressures, p_{net}, for the components and cladding of buildings designed using Method 1 represent the net pressures (sum of internal and external) to be applied normal to each building surface as shown in Figure 6.2.3. p_{net} shall be determined by the following equation:

$$p_{net} = \lambda K_{zt} I p_{net30} \qquad (6.2.4)$$

Where,

λ = adjustment factor for building height and exposure from Figure 6.2.3

K_{zt} = topographic factor as defined in Sec 2.4.7 evaluated at mean roof height, h

I = importance factor as defined in Sec 2.4.5

p_{net30} = net design wind pressure for Exposure A, at $h = 9.1$ m, and for $I = 1.0$, refer to Figure 6-3 of ASCE 7-05.

Minimum Pressures: The positive design wind pressures, p_{net}, from this Section shall not be less than $+0.5$ kN/m², and the negative design wind pressures, p_{net}, from this Section shall not be less than -0.5 kN/m².

2.4.2.4.3 Air permeable cladding

Design wind loads determined from Figure 6.2.3 shall be used for all air permeable cladding unless approved test data or the recognized literature demonstrate lower loads for the type of air permeable cladding being considered.

2.4.3 Method 2: Analytical Procedure

2.4.3.1 Scopes and limitations

A building or other structure whose design wind loads are determined in accordance with this Section shall meet all of the following conditions:

(1) The building or other structure is a regular-shaped building or structure as defined in Sec 2.1.3.

(2) The building or other structure does not have response characteristics making it subject to across-wind loading, vortex shedding, instability due to galloping or flutter; or does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

The provisions of this Section take into consideration of the load magnification effect caused by gusts in resonance with along-wind vibrations of flexible buildings or other structures. Buildings or other structures not meeting the requirements of Sec 2.4.2, or having unusual shapes or response characteristics shall be designed using recognized literature documenting such wind load effects or shall use the wind tunnel procedure specified in Sec 2.4.16.

2.4.3.2 Shielding

There shall be no reductions in velocity pressure due to apparent shielding afforded by buildings and other structures or terrain features.

2.4.3.3 Air permeable cladding

Design wind loads determined from Sec 2.4.3 shall be used for air permeable cladding unless approved test data or recognized literature demonstrate lower loads for the type of air permeable cladding being considered.

2.4.3.4 Design procedure

(1) The basic wind speed V and wind directionality factor K_d shall be determined in accordance with Sec 2.4.4.

(2) An importance factor I shall be determined in accordance with Sec 2.4.5.

(3) An exposure category or exposure categories and velocity pressure exposure coefficient K_z or K_h, as applicable, shall be determined for each wind direction in accordance with Sec 2.4.6.

(4) A topographic factor K_{zt} shall be determined in accordance with Sec 2.4.7.

(5) A gust effect factor G or G_f, as applicable, shall be determined in accordance with Sec 2.4.8.

(6) An enclosure classification shall be determined in accordance with Sec 2.4.9.

(7) Internal pressure coefficient GC_{pi} shall be determined in accordance with Sec 2.4.10.1.

(8) External pressure coefficients C_p or GC_{pf}, or force coefficients C_f, as applicable, shall be determined in accordance with Sections 2.4.10.2 or 2.4.10.3, respectively.

(9) Velocity pressure q_z or q_h, as applicable, shall be determined in accordance with Sec 2.4.9.5.

(10) Design wind load P or F shall be determined in accordance with Sec 2.4.11.

2.4.4 Basic Wind Speed

The basic wind speed, V used in the determination of design wind loads on buildings and other structures shall be as given in Figure 6.2.1 except as provided in Sec 2.4.4.1. The wind shall be assumed to come from any horizontal direction.

2.4.4.1 Special wind regions

The basic wind speed shall be increased where records or experience indicate that the wind speeds are higher than those reflected in Figure 6.2.1. Mountainous terrain, gorges, and special regions shall be examined for unusual wind conditions. The authority having jurisdiction shall, if necessary, adjust the values given in Figure 6.2.1 to account for higher local wind speeds. Such adjustment shall be based on adequate meteorological information and other necessary data.

2.4.4.2 Limitation

Tornadoes have not been considered in developing the basic wind-speed distributions.

2.4.4.3 Wind directionality factor

The wind directionality factor, K_d shall be determined from Table 6.2.12. This factor shall only be applied when used in conjunction with load combinations specified in this Chapter.

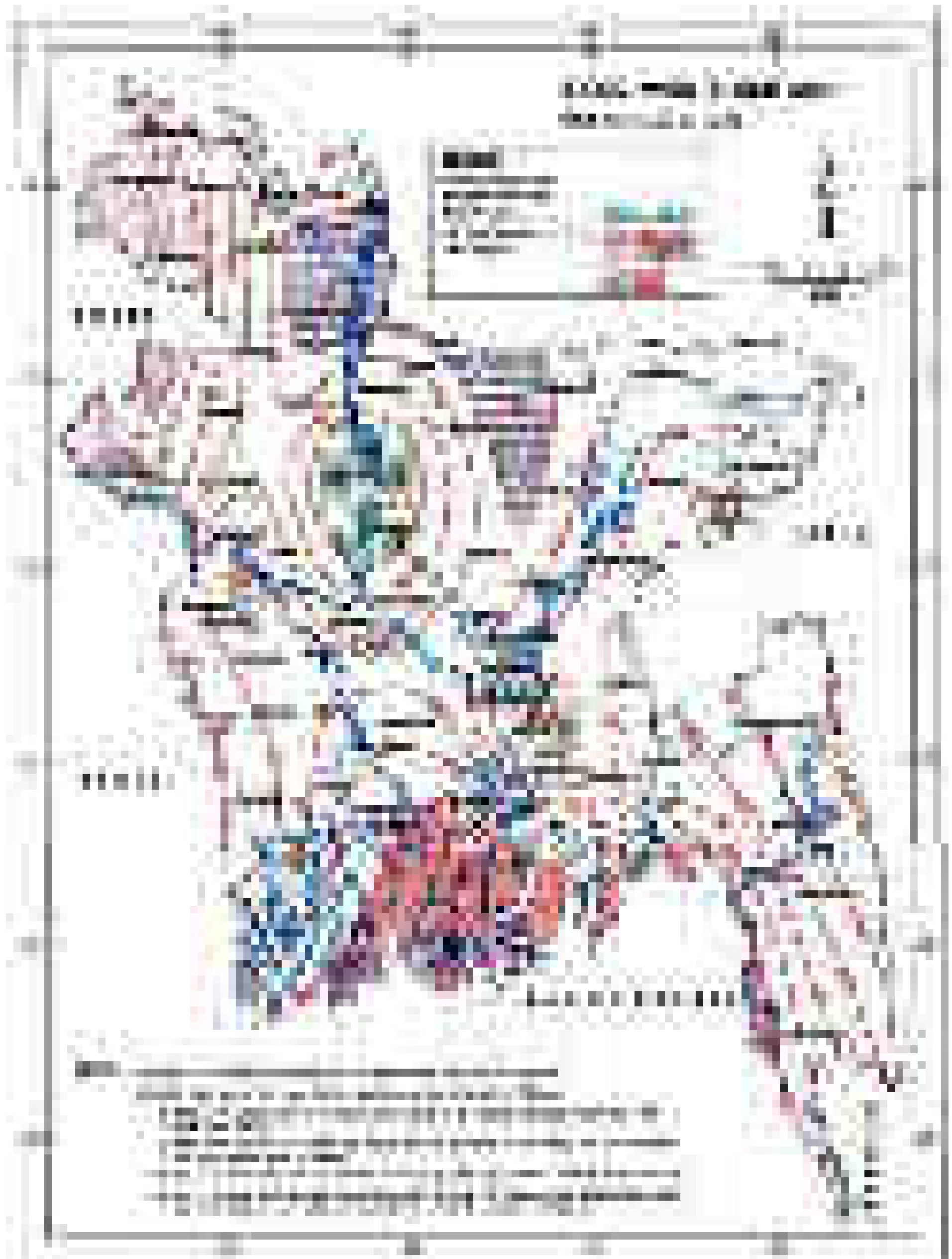


Figure 6.2.1 Basic wind speed (V, m/s) map of Bangladesh

2.4.5 Importance Factor

An importance factor, *I* for the building or other structure shall be determined from Table 6.2.9 based on building and structure categories listed in Sec 1.2.4.

2.4.6 Exposure

For each wind direction considered, the upwind exposure category shall be based on ground surface roughness that is determined from natural topography, vegetation, and constructed facilities.

2.4.6.1 Wind directions and sectors

For each selected wind direction at which the wind loads are to be evaluated, the exposure of the building or structure shall be determined for the two upwind sectors extending 45^o either side of the selected wind direction.

The exposures in these two sectors shall be determined in accordance with Sections 2.4.6.2 and 2.4.6.3 and the exposure resulting in the highest wind loads shall be used to represent the winds from that direction.

2.4.6.2 Surface roughness categories

A ground surface roughness within each 45^o sector shall be determined for a distance upwind of the site as defined in Sec 2.4.6.3 from the categories defined in the following text, for the purpose of assigning an exposure category as defined in Sec 2.4.6.3.

Surface Roughness A: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness B: Open terrain with scattered obstructions having heights generally less than 9.1 m. This category includes flat open country, grasslands, and all water surfaces in cyclone prone regions.

Surface Roughness C: Flat, unobstructed areas and water surfaces outside cyclone prone regions. This category includes smooth mud flats and salt flats.

2.4.6.3 Exposure categories

Exposure A: Exposure A shall apply where the ground surface roughness condition, as defined by Surface Roughness A, prevails in the upwind direction for a distance of at least 792 m or 20 times the height of the building, whichever is greater.

Exception: For buildings whose mean roof height is less than or equal to 9.1 m, the upwind distance may be reduced to 457 m.

Exposure B: Exposure B shall apply for all cases where Exposures A or C do not apply.

Exposure C: Exposure C shall apply where the ground surface roughness, as defined by Surface Roughness C, prevails in the upwind direction for a distance greater than 1,524 m or 20 times the building height, whichever is greater. Exposure C shall extend into downwind areas of Surface Roughness A or B for a distance of 200 m or 20 times the height of the building, whichever is greater.

For a site located in the transition zone between exposure categories, the category resulting in the largest wind forces shall be used.

Exception: An intermediate exposure between the preceding categories is permitted in a transition zone provided that it is determined by a rational analysis method defined in the recognized literature.

2.4.6.4 Exposure category for main wind-force resisting system

Buildings and Other Structures: For each wind direction considered, wind loads for the design of the MWFRS determined from Figure 6.2.6 shall be based on the exposure categories defined in Sec 2.4.6.3.

Low-Rise Buildings: Wind loads for the design of the MWFRSs for low-rise buildings shall be determined using a velocity pressure q_h based on the exposure resulting in the highest wind loads for any wind direction at the site where external pressure coefficients GC_{pf} given in Figure 6.2.10 are used.

2.4.6.5 Exposure category for components and cladding

Components and cladding design pressures for all buildings and other structures shall be based on the exposure resulting in the highest wind loads for any direction at the site.

2.4.6.6 Velocity pressure exposure coefficient

Based on the exposure category determined in Sec 2.4.6.3, a velocity pressure exposure coefficient K_z or K_h, as applicable, shall be determined from Table 6.2.11. For a site located in a transition zone between exposure categories that is near to a change in ground surface roughness, intermediate values of K_z or K_h between those shown in Table 6.2.11, are permitted, provided that they are determined by a rational analysis method defined in the recognized literature.

2.4.7 Topographic Effects

2.4.7.1 Wind speed-up over hills, ridges and escarpments

Wind speed-up effects at isolated hills, ridges, and escarpments constituting abrupt changes in the general topography located in any exposure category shall be included in the design when buildings and other site conditions and locations of structures meet all of the following conditions:

(i) The hill, ridge, or escarpment is isolated and unobstructed upwind by other similar topographic features of comparable height for 100 times the height of the topographic feature (100 H) or 3.22 km, whichever is less. This distance shall be measured horizontally from the point at which the height H of the hill, ridge, or escarpment is determined.

(ii) The hill, ridge, or escarpment protrudes above the height of upwind terrain features within a 3.22 km radius in any quadrant by a factor of two or more.

(iii) The structure is located as shown in Figure 6.2.4 in the upper one-half of a hill or ridge or near the crest of an escarpment.

(iv) $H/L_h \geq 0.2$

(v) H is greater than or equal to 4.5 m for Exposures B and C and 18.3 m for Exposure A.

2.4.7.2 Topographic factor

The wind speed-up effect shall be included in the calculation of design wind loads by using the factor K_{zt}:

$$K_{zt} = (1 + K_1 K_2 K_3)^2 \qquad (6.2.5)$$

Where, K_1, K_2, and K_3 are given in Figure 6.2.4. If site conditions and locations of structures do not meet all the conditions specified in Sec 2.4.7.1 then K_{zt}= 1.0.

2.4.8 Gust Effect Factor

2.4.8.1 Rigid structures

For rigid structures as defined in Sec 2.1.3, the gust-effect factor shall be taken as 0.85 or calculated by the formula:

$$G = 0.925 \frac{1+1.7 g_Q I_{\bar{z}} Q}{1+1.7 g_v I_{\bar{z}}} \qquad (6.2.6)$$

$$I_{\bar{z}} = c \left(\frac{10}{\bar{z}}\right)^{1/6} \qquad (6.2.7)$$

Where, $I_{\bar{z}}$= the intensity of turbulence at height $\bar{z}$ where $\bar{z}$ = the equivalent height of the structure defined as $0.6h$, but not less than z_{min} for all building heights h. z_{min} and c are listed for each exposure in Table 6.2.10; g_Q and the value of g_v shall be taken as 3.4. The background response Q is given by

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \qquad (6.2.8)$$

Where, B, h are defined in Sec 2.1.4; and $L_{\bar{z}}$ = the integral length scale of turbulence at the equivalent height given by

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{10}\right)^{\bar{\epsilon}} \qquad (6.2.9)$$

In which l and $\bar{\in}$ are constants listed in Table 6.2.10.

2.4.8.2 Flexible or dynamically sensitive structures

For flexible or dynamically sensitive structures as defined in Sec 2.1.3 (natural period greater than 1.0 second), the gust-effect factor shall be calculated by

$$G_f = 0.925\left(\frac{1+1.7I_{\bar{z}}\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_vI_{\bar{z}}}\right) \qquad (6.2.10)$$

The value of both g_Q and g_V shall be taken as 3.4 and g_R is given by

$$g_R = \sqrt{2\ln(3600n_1)} + \frac{0.577}{\sqrt{2\ln(3600n_1)}} \qquad (6.2.11)$$

R, the resonant response factor, is given by

$$R = \sqrt{\frac{1}{\beta}R_nR_hR_B(0.53+0.47R_L)} \qquad (6.2.12)$$

$$R_n = \frac{7.47N_1}{(1+10.3N_1)^{5/3}} \qquad (6.2.13)$$

$$N_1 = \frac{n_1L_{\bar{z}}}{\bar{V}_{\bar{z}}} \qquad (6.2.14)$$

$$R_l = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) \text{ for } \eta > 0 \qquad (6.2.15a)$$

$$R_l = 1 \text{ for } \eta = 0 \qquad (6.2.15b)$$

Where, the subscript l in Eq. 6.2.15 shall be taken as h, B, and L, respectively, where h, B, and L are defined in
Sec 2.1.4.

n_1 = building natural frequency

$R_l = R_h$ setting $\eta = 4.6n_1h/\bar{V}_{\bar{z}}$

$R_l = R_B$ setting $\eta = 4.6n_1B/\bar{V}_{\bar{z}}$

$R_l = R_L$ setting $\eta = 15.4n_1L/\bar{V}_{\bar{z}}$

β = damping ratio, percent of critical

$\bar{V}_{\bar{z}}$ = mean hourly wind speed at height $\bar{z}$ determined from Eq. 6.2.16.

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{10}\right)^{\bar{\alpha}} V \qquad (6.2.16)$$

Where, $\bar{b}$ and $\bar{\propto}$ are constants listed in Table 6.2.10.

2.4.8.3 Rational analysis

In lieu of the procedure defined in Sections 2.4.8.1 and 2.4.8.2, determination of the gust-effect factor by any rational analysis defined in the recognized literature is permitted.

2.4.8.4 Limitations

Where combined gust-effect factors and pressure coefficients $\left(GC_p, GC_{pi}, GC_{pf}\right)$ are given in figures and tables, the gust-effect factor shall not be determined separately.

2.4.9 Enclosure Classifications

2.4.9.1 General

For the purpose of determining internal pressure coefficients, all buildings shall be classified as enclosed, partially enclosed, or open as defined in Sec 2.1.3.

2.4.9.2 Openings

A determination shall be made of the amount of openings in the building envelope to determine the enclosure classification as defined in Sec 2.4.9.3.

2.4.9.3 Wind-borne debris

Glazing in buildings located in wind-borne debris regions shall be protected with an impact-resistant covering or be impact-resistant glazing according to the requirements specified in ASTM E1886 and ASTM E1996 or other approved test methods and performance criteria. The levels of impact resistance shall be a function of Missile Levels and Wind Zones specified in ASTM E1886 and ASTM E1996.

Exceptions:

(i) Glazing in Category II, III, or IV buildings located over 18.3 m above the ground and over 9.2 m above aggregate surface roofs located within 458 m of the building shall be permitted to be unprotected.

(ii) Glazing in Category I buildings shall be permitted to be unprotected.

2.4.9.4 Multiple classifications

If a building by definition complies with both the “open” and “partially enclosed” definitions, it shall be classified as an “open” building. A building that does not comply with either the “open” or “partially enclosed” definitions shall be classified as an “enclosed” building.

2.4.9.5 Velocity pressure

Velocity pressure, q_z evaluated at height z shall be calculated by the following equation:

$$q_z = 0.000613 K_z K_{zt} K_d V^2 I\ ;\ \ (\text{kN/m}^2),\ V \text{in m/s} \qquad (6.2.17)$$

Where K_d is the wind directionality factor, K_z is the velocity pressure exposure coefficient defined in Sec 2.4.6.6, K_{zt} is the topographic factor defined in Sec 2.4.7.2, and q_z is the velocity pressure calculated using Eq. 6.2.17 at mean roof height h. The numerical coefficient 0.000613 shall be used except where sufficient climatic data are available to justify the selection of a different value of this factor for a design application.

2.4.10 Pressure And Force Coefficients

2.4.10.1 Internal pressure coefficients

Internal Pressure Coefficient. Internal pressure coefficients, GC_{pi} shall be determined from Figure 6.2.5 based on building enclosure classifications determined from Sec 2.4.9.

Reduction Factor for Large Volume Buildings, R_i: For a partially enclosed building containing a single, unpartitioned large volume, the internal pressure coefficient, GC_{pi} shall be multiplied by the following reduction factor, R_i:

$$R_i = 1.0 \quad \text{or,} \quad R_i = 0.5\left(1 + \frac{1}{\sqrt{1 + \frac{V_i}{6951 A_{og}}}}\right) \leq 1.0 \qquad (6.2.18)$$

Where, A_{og} = total area of openings in the building envelope (walls and roof, in m^2)

V_i= unpartitioned internal volume, in m^3

2.4.10.2 External pressure coefficients

Main Wind-Force Resisting Systems: External pressure coefficients for MWFRSs C_p are given in Figures 6.2.6 to 6.2.8. Combined gust effect factor and external pressure coefficients, GC_{pf} are given in Figure 6.2.10 for low-rise buildings. The pressure coefficient values and gust effect factor in Figure 6.2.10 shall not be separated.

Components and Cladding: Combined gust effect factor and external pressure coefficients for components and cladding GC_p are given in Figures 6.2.11 to 6.2.17. The pressure coefficient values and gust-effect factor shall not be separated.

2.4.10.3 Force coefficients

Force coefficients C_f are given in Figures 6.2.20 to 6.2.23.

2.4.10.4 Roof overhangs

Main Wind-Force Resisting System: Roof overhangs shall be designed for a positive pressure on the bottom surface of windward roof overhangs corresponding to $C_p = 0.8$ in combination with the pressures determined from using Figures 6.2.6 and 6.2.10.

Components and Cladding: For all buildings, roof overhangs shall be designed for pressures determined from pressure coefficients given in Figure 6.2.11.

2.4.10.5 Parapets

Main Wind-Force Resisting System: The pressure coefficients for the effect of parapets on the MWFRS loads are given in Sec 2.4.12.2.

Components and Cladding: The pressure coefficients for the design of parapet component and cladding elements are taken from the wall and roof pressure coefficients as specified in Sec 2.4.12.3.

2.4.11 Design Wind Loads on Enclosed and Partially Enclosed Buildings

2.4.11.1 General

Sign Convention: Positive pressure acts toward the surface and negative pressure acts away from the surface.

Critical Load Condition: Values of external and internal pressures shall be combined algebraically to determine the most critical load.

Tributary Areas Greater than 65 m^2: Component and cladding elements with tributary areas greater than 65 m^2 shall be permitted to be designed using the provisions for MWFRSs.

2.4.11.2 Main wind-force resisting systems

Rigid Buildings of All Heights: Design wind pressures for the MWFRS of buildings of all heights shall be determined by the following equation:

$$p = qGC_p - q_i(GC_{pi}) \quad (\mathrm{kN/m^2}) \qquad (6.2.19)$$

Where,

$q = q_z$ for windward walls evaluated at height z above the ground

$q = q_h$ for leeward walls, side walls, and roofs, evaluated at height h

$q_i = q_h$ for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings.

$q_i = q_z$ for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For buildings sited in wind-borne debris regions, glazing that is not impact resistant or protected with an impact resistant covering, shall be treated as an opening in accordance with Sec 2.4.9.3. For positive internal pressure evaluation, q_i may conservatively be evaluated at height $h =$ $(q_i = q_h)$

G = gust effect factor from Sec 2.4.8

C_p = external pressure coefficient from Figures 6.2.6 or 6.2.8

GC_{pi} = internal pressure coefficient from Figure 6.2.5

q and q_i shall be evaluated using exposure defined in Sec 2.4.6.3. Pressure shall be applied simultaneously on windward and leeward walls and on roof surfaces as defined in Figures 6.2.6 and 6.2.8.

Low-Rise Building: Alternatively, design wind pressures for the MWFRS of low-rise buildings shall be determined by the following equation:

$$p = q_h\left[\left(GC_{pf}\right) - \left(GC_{pi}\right)\right] \text{ (kN/m}^2\text{)} \qquad (6.2.20)$$

Where,

q_h = velocity pressure evaluated at mean roof height h using exposure defined in Sec 2.4.6.3

GC_{pf} = external pressure coefficient from Figure 6.2.10

GC_{pi} = internal pressure coefficient from Figure 6.2.5

Flexible Buildings: Design wind pressures for the MWFRS of flexible buildings shall be determined from the following equation:

$$p = qG_f C_p - q_i\left(GC_{pi}\right) \text{ (kN/m}^2\text{)} \qquad (6.2.21)$$

Where, q, q_i, C_p, and GC_{pi} are as defined in Sec 2.4.11.2 and G_f= gust effect factor is defined as in Sec 2.4.8.

Parapets: The design wind pressure for the effect of parapets on MWFRSs of rigid, low-rise, or flexible buildings with flat, gable, or hip roofs shall be determined by the following equation:

$$p_p = q_p GC_{pn} \text{ (kN/m}^2\text{)} \qquad (6.2.22)$$

Where,

$p_p =$ Combined net pressure on the parapet due to the combination of the net pressures from the front and back parapet surfaces. Plus (and minus) signs signify net pressure acting toward (and away from) the front (exterior) side of the parapet

$q_p =$ Velocity pressure evaluated at the top of the parapet

$GC_{pn} =$ Combined net pressure coefficient

$= +1.5$ for windward parapet

$= -1.0$ for leeward parapet

2.4.11.3 Design wind load cases

The MWFRS of buildings of all heights, whose wind loads have been determined under the provisions of Sec 2.4.11.2, shall be designed for the wind load cases as defined in Figure 6.2.9. The eccentricity e for rigid structures shall be measured from the geometric center of the building face and shall be considered for each principal axis (e_x, e_y). The eccentricity e for flexible structures shall be determined from the following equation and shall be considered for each principal axis (e_x, e_y):

$$e = \frac{e_Q + 1.7 I_{\bar{z}} \sqrt{\left(g_Q Q e_Q\right)^2 + (g_R R e_R)^2}}{1 + 1.7 I_{\bar{z}} \sqrt{\left(g_Q Q\right)^2 + (g_R R)^2}} \qquad (6.2.23)$$

Where,

$e_Q =$ Eccentricity e as determined for rigid structures in Figure 6.2.9

$e_R =$ Distance between the elastic shear center and center of mass of each floor

$I_{\bar{z}}, g_Q, Q, g_R, R$ shall be as defined in Sec 2.1.4

The sign of the eccentricity e shall be plus or minus, whichever causes the more severe load effect.

Exception: One-story buildings with h less than or equal to 9.1 m, buildings two stories or less framed with light-frame construction, and buildings two stories or less designed with flexible diaphragms need only be designed for Load Case 1 and Load Case 3 in Figure 6.2.9.

2.4.11.4 Components and cladding.

Low-Rise Buildings and Buildings with $h \leq 18.3$ m: Design wind pressures on component and cladding elements of low-rise buildings and buildings with $h \leq$ 18.3 m shall be determined from the following equation:

$$p = q_h\left[\left(GC_p\right) - \left(GC_{pi}\right)\right] \ (\mathrm{kN/m^2}) \qquad (6.2.24)$$

Where,

q_h = Velocity pressure evaluated at mean roof height h using exposure defined in Sec 2.4.6.5

GC_p = External pressure coefficients given in Figures 6.2.11 to 6.2.16

GC_{pi} = Internal pressure coefficient given in Figure 6.2.5

Buildings with $h > 18.3$ m: Design wind pressures on components and cladding for all buildings with $h > 18.3$ m shall be determined from the following equation:

$$p = q\left(GC_p\right) - q_i\left(GC_{pi}\right) \ (\mathrm{kN/m^2}) \qquad (6.2.25)$$

Where,

$q = q_z$ for windward walls calculated at height z above the ground

$q = q_h$ for leeward walls, side walls, and roofs, evaluated at height h

$q_i = q_h$ for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings

$q_i = q_z$ for positive internal pressure evaluation in partially enclosed buildings where height z is defined as the level of the highest opening in the building that could affect the positive internal pressure. For buildings sited in wind-borne debris regions, glazing that is not impact resistant or protected with an impact-resistant covering, shall be treated as an opening in accordance with Sec 2.4.9.3. For positive internal pressure evaluation, q_i may conservatively be evaluated at height h ($q_i = q_h$)

$\left(GC_p\right) =$ external pressure coefficient from Figure 6.2.17.

$\left(GC_{pi}\right) =$ internal pressure coefficient given in Figure 6.2.5. q and q_i shall be evaluated using exposure defined in Sec 2.4.6.3.

2.4.11.5 Alternative design wind pressures for components and cladding in buildings with 18.3 m $< h <$ 27.4 m.

Alternative to the requirements of Sec 2.4.11.2, the design of components and cladding for buildings with a mean roof height greater than 18.3 m and less than 27.4 m values from Figures 6.2.11 to 6.2.17 shall be used only if the height to width ratio is one or less (except as permitted by Notes of Figure 6.2.17) and Eq. 6.2.24 is used.

Parapets: The design wind pressure on the components and cladding elements of parapets shall be designed by the following equation:

$$p = q_p(GC_p - GC_{pi}) \qquad (6.2.26)$$

Where,

q_p = Velocity pressure evaluated at the top of the parapet

GC_p = External pressure coefficient from Figures 6.2.11 to 6.2.17

GC_{pi} = Internal pressure coefficient from Figure 6.2.5, based on the porosity of the parapet envelope.

Two load cases shall be considered. Load Case A shall consist of applying the applicable positive wall pressure from Figures 6.2.11 or 6.2.17 to the front surface of the parapet while applying the applicable negative edge or corner zone roof pressure from Figures 6.2.11 to 6.2.17 to the back surface. Load Case B shall consist of applying the applicable positive wall pressure from Figures 6.2.11 or 6.2.17 to the back of the parapet surface, and applying the applicable negative wall pressure from Figures 6.2.11 or 6.2.17 to the front surface. Edge and corner zones shall be arranged as shown in Figures 6.2.11 to 6.2.17. GC_p shall be determined for appropriate roof angle and effective wind area from Figures 6.2.11 to 6.2.17. If internal pressure is present, both load cases should be evaluated under positive and negative internal pressure.

2.4.12 Design Wind Loads on Open Buildings with Monoslope, Pitched, or Troughed Roofs

2.4.12.1 General

Sign Convention: Plus and minus signs signify pressure acting toward and away from the top surface of the roof, respectively.

Critical Load Condition: Net pressure coefficients C_N include contributions from top and bottom surfaces. All load cases shown for each roof angle shall be investigated.

2.4.12.2 Main wind-force resisting systems

The net design pressure for the MWFRSs of monoslope, pitched, or troughed roofs shall be determined by the following equation:

$$p = q_h G C_N \tag{6.2.27}$$

Where,

q_h = Velocity pressure evaluated at mean roof height h using the exposure as defined in Sec 2.4.6.3 that results in the highest wind loads for any wind direction at the site

G = Gust effect factor from Sec 2.4.8

C_N = Net pressure coefficient determined from Figures 6.2.18(a) to 6.2.18(d).

For free roofs with an angle of plane of roof from horizontal θ less than or equal to 5^o and containing fascia panels, the fascia panel shall be considered an inverted parapet. The contribution of loads on the fascia to the MWFRS loads shall be determined using Sec 2.4.11.5 with q_p equal to q_h.

2.4.12.3 Component and cladding elements

The net design wind pressure for component and cladding elements of monoslope, pitched, and troughed roofs shall be determined by the following equation:

$$p = q_h G C_N \tag{6.2.28}$$

Where,

q_h = Velocity pressure evaluated at mean roof height h using the exposure as defined in Sec 2.4.6.3 that results in the highest wind loads for any wind direction at the site

G = Gust-effect factor from Sec 2.4.8

C_N = Net pressure coefficient determined from Figures 6.2.19(a) to 6.2.19(c).

2.4.13 Design Wind Loads on Solid Free Standing Walls and Solid Signs

The design wind force for solid freestanding walls and solid signs shall be determined by the following formula:

$$F = q_h G C_f A_s \text{ (kN)} \tag{6.2.29}$$

Where,

q_h = Velocity pressure evaluated at height h (Figure 6.2.20) using exposure defined in Sec2.4.6.3

G = Gust-effect factor from Sec 2.4.8

C_f = Net force coefficient from Figure 6.2.20

A_s = Gross area of the solid freestanding wall or solid sign, in m^2

2.4.14 Design Wind Loads on Other Structures

The design wind force for other structures shall be determined by the following equation:

$$F = q_z G C_f A_f \text{ (kN)} \quad (6.2.30)$$

Where,

q_z = Velocity pressure evaluated at height z of the centroid of area A_f using exposure as in Sec 2.4.6.3

G = Gust-effect factor from Sec 2.4.8

C_f = Force coefficients from Figures 6.2.21 to 6.2.23.

A_f = Projected area normal to the wind except where C_f is specified for the actual surface area, m^2

2.4.15 Rooftop Structures and Equipment for Buildings with $h \leq 18.3$ m

The force on rooftop structures and equipment with A_f less than $(0.1Bh)$ located on buildings with $h \leq 18.3$ m shall be determined from Eq. 6.2.30, increased by a factor of 1.9. The factor shall be permitted to be reduced linearly from 1.9 to 1.0 as the value of A_f is increased from $(0.1Bh)$ to (Bh).

2.4.16 Method 3 - Wind Tunnel Procedure

2.4.16.1 Scope

Wind tunnel tests shall be used where required by Sec 2.4.3.1. Wind tunnel testing shall be permitted in lieu of Methods 1 and 2 for any building or structure.

2.4.16.2 Test conditions

Wind tunnel tests, or similar tests employing fluids other than air, used for the determination of design wind loads for any building or other structure, shall be conducted in accordance with this Section. Tests for the determination of mean and fluctuating forces and pressures shall meet all of the following conditions:

(i) Natural atmospheric boundary layer has been modeled to account for the variation of wind speed with height.

(ii) The relevant macro- (integral) length and micro-length scales of the longitudinal component of atmospheric turbulence are modeled to approximately the same scale as that used to model the building or structure.

(iii) The modeled building or other structure and surrounding structures and topography are geometrically similar to their full-scale counterparts, except that, for low-rise buildings meeting the requirements of Sec 2.4.3.1, tests shall be permitted for the modeled building in a single exposure site as in Sec 2.4.6.

(iv) The projected area of the modeled building or other structure and surroundings is less than 8 percent of the test section cross-sectional area unless correction is made for blockage.

(v) The longitudinal pressure gradient in the wind tunnel test section is accounted for.

(vi) Reynolds number effects on pressures and forces are minimized.

(vii) Response characteristics of the wind tunnel instrumentation are consistent with the required measurements.

2.4.17 Dynamic Response

Tests for the purpose of determining the dynamic response of a building or other structure shall be in accordance with Sec 2.4.16.2. The structural model and associated analysis shall account for mass distribution, stiffness, and damping.

Enclosed Buildings: Walls & Roofs

Notes:

1. Pressures shown are applied to the horizontal and vertical projections, for exposure A, at h=9.1m, I=1.0, and K_{zt} = 1.0. Adjust to other conditions using Equation 6.2.3.
2. The load patterns shown shall be applied to each corner of the building in turn as the reference corner. (See Figure 6.2.10)
3. For the design of the longitudinal MWFRS use θ = 0°, and locate the zone E/F, G/H boundary at the mid-length of the building.
4. Load cases 1 and 2 must be checked for 25° < θ ≤ 45°. Load case 2 at 25° is provided only for interpolation between 25° to 30°.
5. Plus and minus signs signify pressures acting toward and away from the projected surfaces, respectively.
6. For roof slopes other than those shown, linear interpolation is permitted.
7. The total horizontal load shall not be less than that determined by assuming p_s = 0 in zones B & D.
8. The zone pressures represent the following:

   Horizontal pressure zones – Sum of the windward and leeward net (sum of internal and external) pressures on vertical projection of:

   A - End zone of wall   *C* - Interior zone of wall

   B - End zone of roof   *D* - Interior zone of roof

   Vertical pressure zones – Net (sum of internal and external) pressures on horizontal projection of:

   E - End zone of windward roof   *G* - Interior zone of windward roof

   F - End zone of leeward roof   *H* - Interior zone of leeward roof
9. Where zone E or G falls on a roof overhang on the windward side of the building, use EOH and GOH for the pressure on the horizontal projection of the overhang. Overhangs on the leeward and side edges shall have the basic zone pressure applied.
10. Notation:

    a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

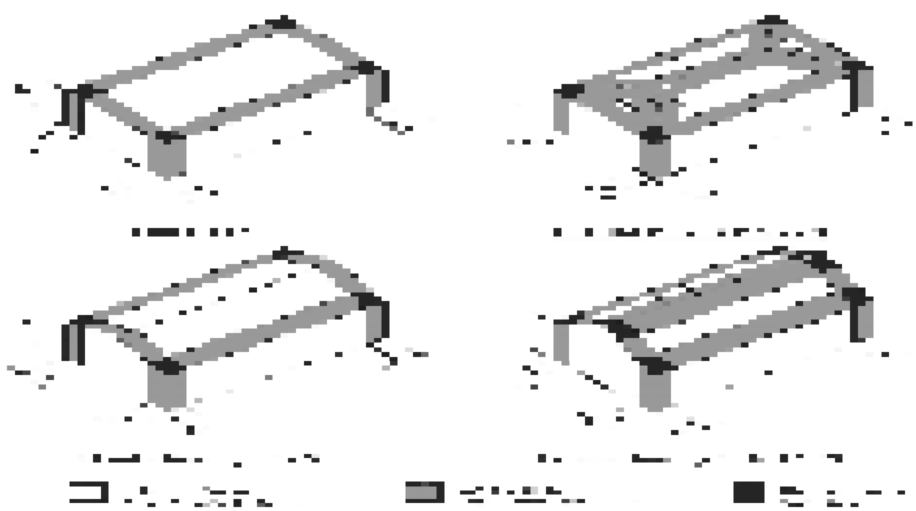
    h: Mean roof height, in feet (meters), except that eave height shall be used for roof angles <10°.

θ: Angle of plane of roof from horizontal, in degrees.

Adjustment Factor for Building Height and Exposure, λ			
Mean roof height (m)	**Exposure**		
	A	**B**	**C**
4.6	1.00	1.21	1.47
6.0	1.00	1.29	1.55
7.6	1.00	1.35	1.61
9.1	1.00	1.40	1.66
10.7	1.05	1.45	1.70
12.2	1.09	1.49	1.74
13.7	1.12	1.53	1.78
15.2	1.16	1.56	1.81
16.8	1.19	1.59	1.84
18.3	1.22	1.62	1.87

Figure 6.2.2 Design wind pressure for main wind force resisting system- Method 1 ($h \leq 18.3$ m)

Enclosed Buildings: Walls & Roofs



Notes:

1. Pressures shown are applied normal to the surface, for exposure A, at h = 9.1m, I = 1.0, and K_{zt} = 1.0. Adjust to other conditions using Equation 6.2.4.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. For hip roofs with $\theta \leq 25°$, Zone 3 shall be treated as Zone 2.
4. For effective wind areas between those given, value may be interpolated, otherwise use the value associated with the lower effective wind area.
5. Notation:

a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

h: Mean roof height, in feet (meters), except that eave height shall be used for roof angles <10°.

θ: Angle of plane of roof from horizontal, in degrees.

Roof Overhang Net Design Wind Pressure, P_{net30} (kN/m²)
(Exposure A at h = 9.1 m with I= 1.0)

Roof Pitch	Zone	Effective Wind area (m²)	Basic Wind Speed V (m/s)							
			40.23	44.7	49.17	53.64	58.11	62.58	67.05	75.99
Roof 0 to 7 degrees	2	0.930	-1.005	-1.239	-1.502	-1.785	-2.096	-2.431	-2.790	-3.584
	2	1.860	-0.986	-1.220	-1.473	-1.756	-2.058	-2.388	-2.742	-3.522
	2	4.648	-0.962	-1.191	-1.440	-1.713	-2.010	-2.330	-2.675	-3.436
	2	9.296	-0.947	-1.168	-1.412	-1.680	-1.971	-2.287	-2.627	-3.373
	3	0.930	-1.656	-2.043	-2.470	-2.943	-3.450	-4.005	-4.594	-5.905
	3	1.860	-1.297	-1.603	-1.938	-2.311	-2.708	-3.144	-3.609	-4.632
	3	4.648	-0.828	-1.024	-1.240	-1.474	-1.727	-2.005	-2.302	-2.957
	3	9.296	-0.479	-0.584	-0.708	-0.842	-0.986	-1.144	-1.311	-1.684

Figure 6.2.3 Design wind pressure for components and cladding - Method 1 ($h \leq$ 18.3 m)

Roof Overhang Net Design Wind Pressure, P_{net30} (kN/m²)
(Exposure A at h = 9.1 m with I= 1.0)

Roof Pitch	Zone	Effective Wind area (m²)	Basic Wind Speed V (m/s)							
			40.23	44.7	49.17	53.64	58.11	62.58	67.05	75.99
Roof > 7 to 27 degrees	2	0.930	-1.302	-1.603	-1.943	-2.311	-2.713	-3.144	-3.613	-4.637
	2	1.860	-1.302	-1.603	-1.943	-2.311	-2.713	-3.144	-3.613	-4.637
	2	4.648	-1.302	-1.603	-1.943	-2.311	-2.713	-3.144	-3.613	-4.637
	2	9.296	-1.302	-1.603	-1.943	-2.311	-2.713	-3.144	-3.613	-4.637
	3	0.930	-2.187	-2.699	-3.268	-3.885	-4.560	-5.292	-6.072	-7.800
	3	1.860	-1.971	-2.436	-2.948	-3.507	-4.115	-4.775	-5.479	-7.039
	3	4.648	-1.689	-2.086	-2.526	-3.005	-3.526	-4.091	-4.694	-6.034
	3	9.296	-1.479	-1.823	-2.206	-2.627	-3.082	-3.574	-4.106	-5.268

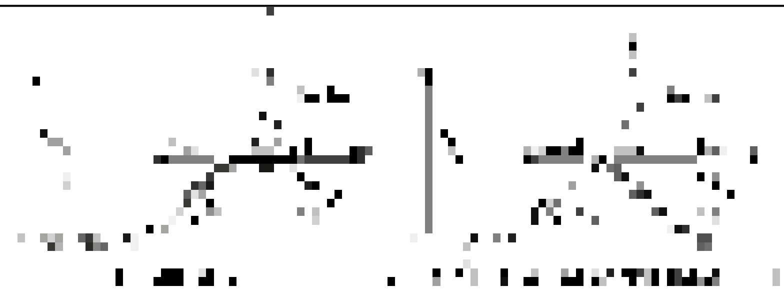
Roof >27 to 45 degrees	**2**	0.930	-1.182	-1.460	-1.766	-2.101	-2.464	-2.861	-3.282	-4.216
	2	1.860	-1.148	-1.416	-1.713	-2.038	-2.393	-2.775	-3.182	-4.091
	2	4.648	-1.101	-1.359	-1.641	-1.952	-2.292	-2.660	-3.052	-3.924
	2	9.296	-1.062	-1.311	-1.587	-1.890	-2.220	-2.574	-2.952	-3.795
	3	0.930	-1.182	-1.460	-1.766	-2.101	-2.464	-2.861	-3.283	-4.216
	3	1.860	-1.148	-1.416	-1.713	-2.038	-2.393	-2.775	-3.182	-4.091
	3	4.648	-1.101	-1.359	-1.641	-1.952	-2.292	-2.660	-3.053	-3.923
	3	9.296	-1.062	-1.311	-1.589	-1.890	-2.220	-2.574	-2.952	-3.795

Adjustment Factor for Building Height and Exposure, λ

Mean roof height (m)	**Exposure**		
	A	**B**	**C**
4.6	1.00	1.21	1.47
6.1	1.00	1.29	1.55
7.6	1.00	1.35	1.61
9.15	1.00	1.40	1.66
10.7	1.05	1.45	1.70
12.2	1.09	1.49	1.74
13.7	1.12	1.53	1.78
15.2	1.16	1.56	1.81
16.8	1.19	1.59	1.84
18.3	1.22	1.62	1.87

Unit Conversion – 1.0 ft =0.3048 m; 1.0 ft^2 = 0.0929 m^2; 1.0 psf = 0.0479 kN/m^2

Figure 6.2.3 (Contd.) Design wind pressure for components and cladding-Method 1 (*h* ≤ 18.3 m)



Topographic Multipliers for Exposure B										
H/L_h	***K_1* Multiplier**			***x/L_h***	***K_2* Multiplier**		***z/L_h***	***K_3* Multiplier**		
	2-D Ridge	**2-D Escarp.**	**3-D Axisym. Hill**		**2-D Escarp.**	**All Other Cases**		**2-D Ridge**	**2-D Escarp.**	**3-D Axisym. Hill**
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

1. For values of H/L_h, x/L_h and z/L_h other than those shown, linear interpolation is permitted.
2. For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_1 and substitute $2H$ for L_h for evaluating K_2 and K_3.
3. Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
4. Notation:

   H: Height of hill or escarpment relative to the upwind terrain, in meters.

   L_h: Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in meters.

   K_1: Factor to account for shape of topographic feature and maximum speed-up effect.

   K_2: Factor to account for reduction in speed-up with distance upwind or downwind of crest.

   K_3: Factor to account for reduction in speed-up with height above local terrain.

   x: Distance (upwind or downwind) from the crest to the building site, in meters.

   z: Height above local ground level, in meters.

   W: Horizontal attenuation factor.

   γ: Height attenuation factor

   Equation:

   $K_{zt} = (1 + K_1\ K_2\ K_3)^2$; K_1 determined from Table below; $K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$; $K_3 = e^{-\gamma z/L_h}$

Parameters for Speed-Up Over Hills and Escarpments						
Hill Shape	**$K_1/(H/L_h)$**			**γ**	**μ**	
	Exposure A	**Exposure B**	**Exposure C**		**Upwind of crest**	**Downwind of Crest**
2-dimensional ridges (or valleys with negative H in $K_1/(H/L_h)$)	1.30	1.45	1.55	3	1.5	1.5
2-dimensional escarpments	0.75	0.85	0.95	2.5	1.5	4
3-dimensional axisym. Hill	0.95	1.05	1.15	4	1.5	1.5

Figure 6.2.4 Topographic factor, K_{zt} - Method 2

Enclosed, Partially Enclosed, and Open Buildings: Walls & Roofs

Enclosure Classification	**GC_{pi}**
Open Building	0.00
Partially Enclosed Building	+0.55 -0.55
Enclosed Building	+0.18 -0.18

Notes:

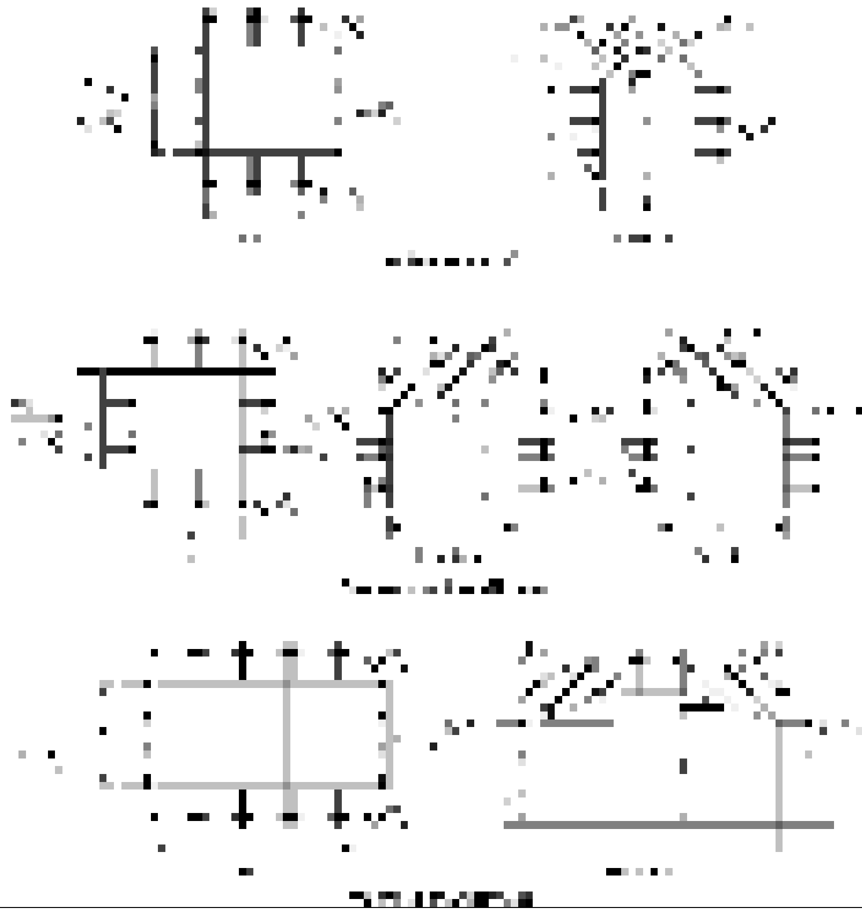
1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of GC_{pi} shall be used with q_z or q_h as specified in Sec 2.4.11.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:

   (i) a positive value of GC_{pi} applied to all internal surfaces

   (ii) a negative value of GC_{pi} applied to all internal surfaces.

Figure 6.2.5 Internal pressure coefficient, GC_{pi} main wind force resisting system component and cladding - Method 2 (All Heights)

Enclosed, Partially Enclosed Buildings: Walls & Roofs



Wall Pressure Coefficients, C_p			
Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥ 4	-0.2	
Side Wall	All values	-0.7	q_h

Figure 6.2.6 External Pressure Coefficients, C_p main wind force resisting system - Method 2 (All Heights)

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction	Windward									Leeward		
	Angle, θ (degrees)									Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	>60[#]	10	15	>20
Normal To ridge for $\theta \geq 10^0$	≤0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01θ	-0.7	-0.6	-0.6

Wind Direction	*h/L*	Horizontal distance from Windward edge	C_p
Normal To ridge for $\theta < 10^o$ and Parallel to ridge for all θ	≤ 0.5	0 to *h/2*	-0.9, -0.18
		h/2 to *h*	-0.9, -0.18
		h to *2 h*	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to *h/2*	-1.3**,-0.18
		> *h/2*	-0.7, -0.18

* Value is provided for interpolation purposes

** Value can be reduced linearly with area over which it is applicable as follows

Area (m^2)	Reduction Factor
≤ 9.3	1.0
23.2	0.9
≥ 92.9	0.8

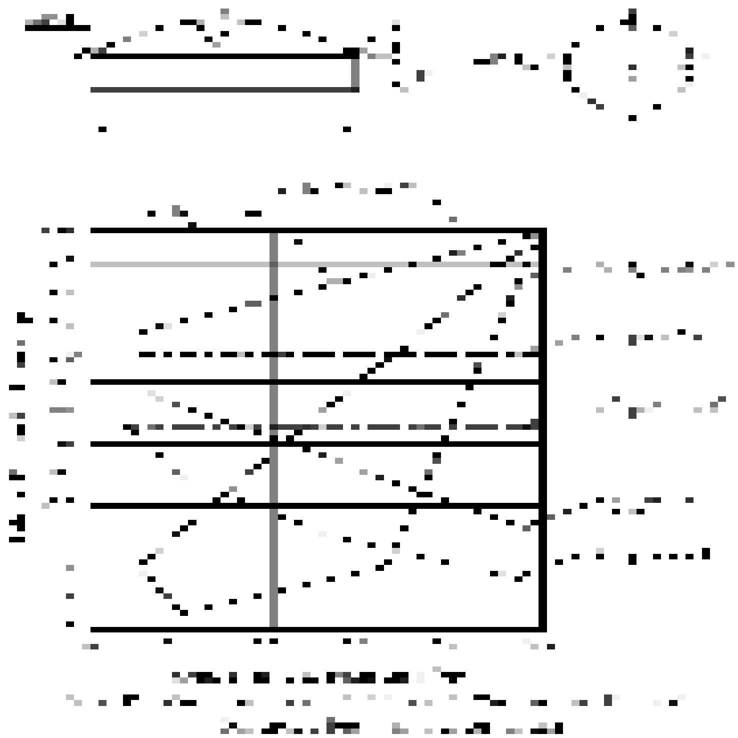
Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of *L/B, h/L* and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of *h/L* in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined by Sec 2.4.8.
6. Refer to Figure 6.2.7 for domes and Figure 6.2.8 for arched roofs.
7. Notation:
   - *B*: Horizontal dimension of building, in meter, measured normal to wind direction.
   - *L*: Horizontal dimension of building, in meter, measured parallel to wind direction.
   - *h*: Mean roof height in meters, except that eave height shall be used for e 10 degrees.
   - *z*: Height above ground, in meters.
   - *G*: Gust effect factor.
   - q_z, q_h: Velocity pressure, in N/m^2, evaluated at respective height.
   - θ: Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

[#]For roof slopes greater than 80°, use C_p = 0.8

Figure 6.2.6 (Contd.) External pressure coefficients, C_p main wind force resisting system - Method 2 (All Heights)

Enclosed, Partially Enclosed Buildings and Structures: Domed Roofs



Notes:

1. Two load cases shall be considered:

   Case A. C_p values between A and B and between B and C shall be determined by linear interpolation along arcs on the dome parallel to the wind direction;

   Case B. C_p shall be the constant value of A for θ ≤ 25 degrees, and shall be determined by linear interpolation from 25 degrees to B and from B to C.
2. Values denote C_p to be used with q_{h_D+f} where (h_D+ f) is the height at the top of the dome.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. C_p is constant on the dome surface for arcs of circles perpendicular to the wind direction; for example, the arc passing through B-B-B and all arcs parallel to B-B-B.
5. For values of h_D/D between those listed on the graph curves, linear interpolation shall be permitted.
6. θ=0 degrees on dome springline, θ=90 degrees at dome center top point. f is measured from springline to top.
7. The total horizontal shear shall not be less than that determined by neglecting wind forces roof surfaces.
8. For f/D values less than 0.05, use Figure 6.2.6.

Figure 6.2.7 External pressure coefficients, C_p main wind force resisting system - Method 2 (All Heights)

Enclosed, Partially Enclosed Buildings and Structures: Arched Roofs

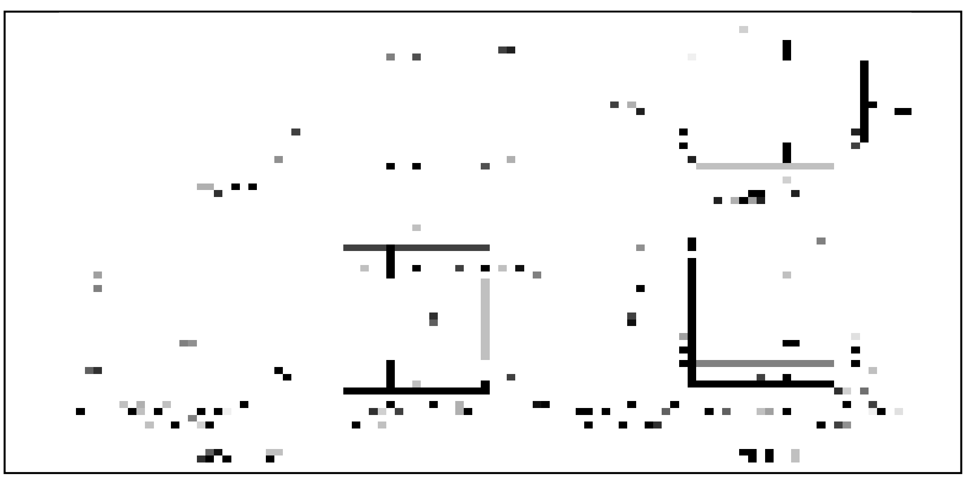
Condition	Rise-to-span ratio, r	C_p		
		Windward quarter	Center half	Leeward quarter
Roof on elevated structure	$0 < r < 0.2$	-0.9	-0.7 - r	-0.5
	$0.2 \leq r < 0.3$*	1.5 r - 0.3	-0.7 - r	-0.5
	$0.3 \leq r \leq 0.6$	2.75 r - 0.7	-0.7 - r	-0.5
Roof springing from ground level	$0 < r \leq 0.6$	1.4 r	-0.7 - r	-0.5

Notes:

* When the rise-to-span ratio is $0.2 \leq r \leq 0.3$, alternate coefficients given by ($6r$- 2.1) shall also be used for the windward quarter.

1. Values listed are for the determination of average load on main wind force resisting systems.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. For wind directed parallel to the axis of the arch, use pressure coefficients from Figure 6.2.6 with wind directed parallel to ridge.
4. For components and cladding: (1) At roof perimeter, use the external pressure coefficients in Figure 6.2.11 with e based on spring-line slope and (2) for remaining roof areas, use external pressure coefficients of this Table multiplied by 0.87.

Figure 6.2.8 External pressure coefficients, C_p main wind force resisting system component and cladding - Method 2 (All Heights)



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of Sec 2.4.11 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

   P_{wx}, P_{wY}: Windward face design pressure acting in the *x, y* principal axis, respectively.

   P_{LX}, P_{LY}: Leeward face design pressure acting in the *x, y* principal axis, respectively.

   $e(e_x, e_y)$: Eccentricity for the *x, y* principal axis of the structure, respectively.

   M_T: Torsional moment per unit height acting about a vertical axis of the building.

Figure 6.2.9 Design wind load cases for main wind force resisting system-Method 2 (All Heights)

Enclosed, Partially Enclosed Buildings: Low-rise Walls & Roofs

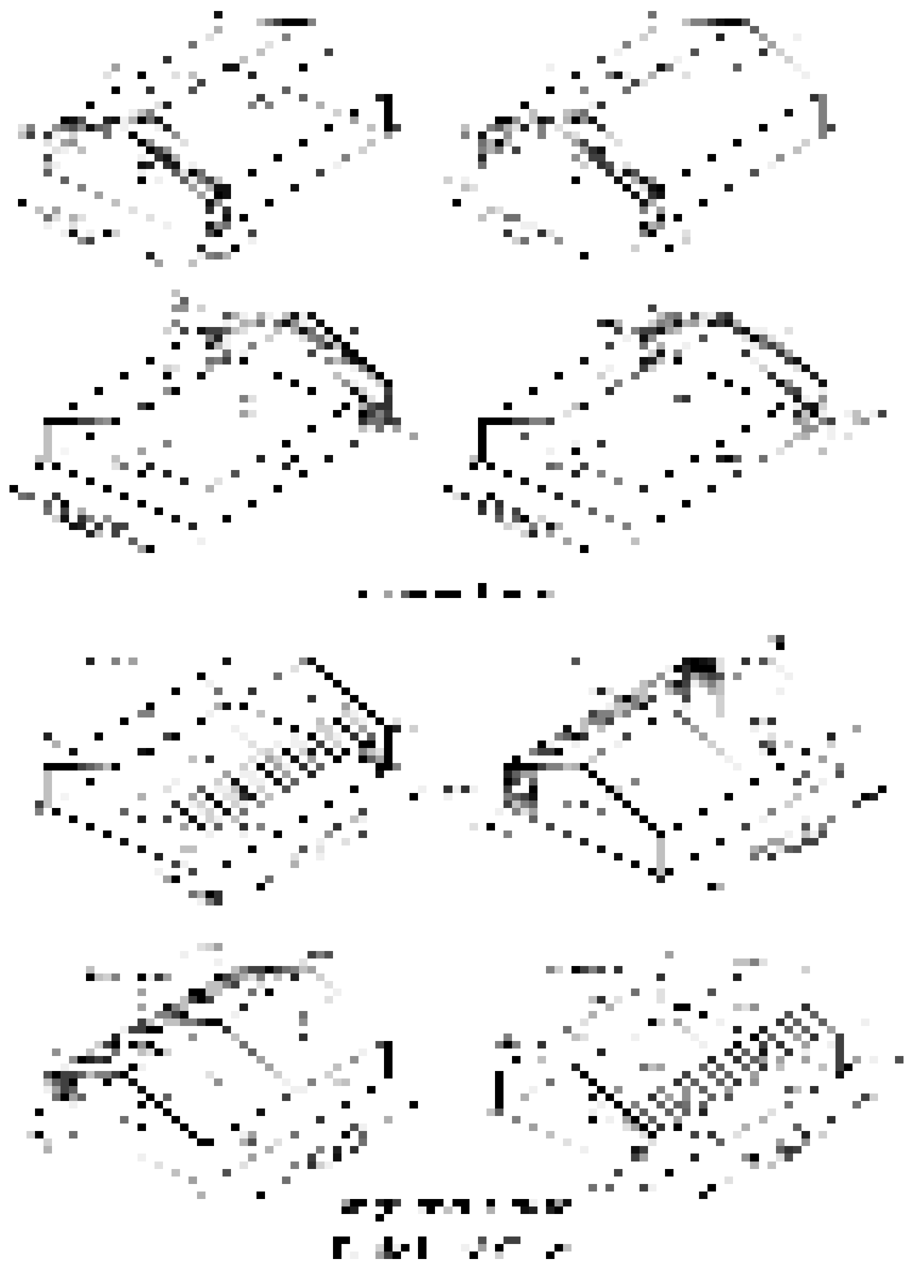


Figure 6.2.10 External pressure coefficients, GC_{pf} for main wind force resisting system- Method 2 ($h \le 18.3$ m)

Enclosed, Partially Enclosed Buildings: Low-rise Walls & Roofs

Roof Angle θ (degrees)	Building Surface									
	1	2	3	4	5	6	1E	2E	3E	4E
0-5	0.40	-0.69	-0.37	-0.29	-0.45	-0.45	0.61	-1.07	-0.53	-0.43
20	0.53	-0.69	-0.48	-0.43	-0.45	-0.45	0.80	-1.07	-0.69	-0.64
30-45	0.56	0.21	-0.43	-0.37	-0.45	-0.45	0.69	0.27	-0.53	-0.48
90	0.56	0.56	-0.37	-0.37	-0.45	-0.45	0.69	0.69	-0.48	-0.48

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. For values of θ other than those shown, linear interpolation is permitted.
3. The building must be designed for all wind directions using the 8 loading patterns shown. The load patterns are applied to each building corner in turn as the Reference Corner.
4. Combinations of external and internal pressures (see Figure 6.2.5) shall be evaluated as required to obtain the most severe loadings.
5. For the torsional load cases shown below, the pressures in zones designated with a "T" (1T, 2T, 3T, 4T) shall be 25% of the full design wind pressures (zones 1, 2, 3, 4).

   Exception: One story buildings with h less than or equal to 9.1m, buildings two stories or less framed with light frame construction, and buildings two stories or less designed with flexible diaphragms need not be designed for the torsional load cases.

   Torsional loading shall apply to all eight basic load patterns using the figures below applied at each reference corner.
6. Except for moment-resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.
7. For the design of the MWFRS providing lateral resistance in a direction parallel to a ridge line or for flat roofs, use θ = 0° and locate the zone 2/3 boundary at the mid-length of the building.
8. The roof pressure coefficient GC_{pf}, when negative in Zone 2 or *2E*, shall be applied in Zone *2/2E* for a distance from the edge of roof equal to 0.5 times the horizontal dimension of the building parallel to the direction of the MWFRS being designed or 2.5 times the eave height, he, at the windward wall, whichever is less; the remainder of Zone *2/2E* extending to the ridge line shall use the pressure coefficient GC_{pf} for Zone *3/3E.*
9. Notation:

   a: 10 percent of least horizontal dimension or *0.4h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

   h: Mean roof height, in meters, except that eave height shall be used for $\theta \leq 10°$.

   θ: Angle of plane of roof from horizontal, in degrees.

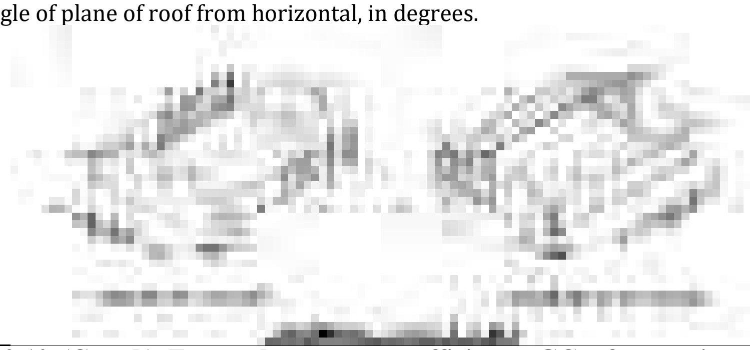
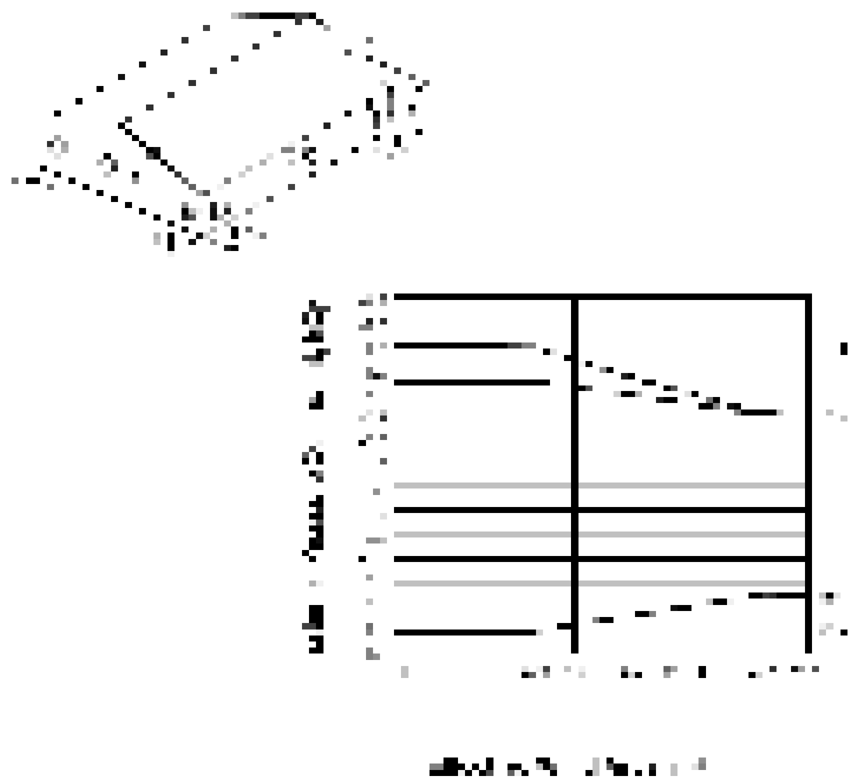


Figure 6.2.10 (Contd.) External pressure coefficients, GC_{pf} for main wind force resisting system - Method 2 (h ≤ 18.3 m)

Enclosed, Partially Enclosed Buildings: Walls



Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area, in square meters.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_P for walls shall be reduced by 10% when $\theta \leq 10^0$.
6. Notation:

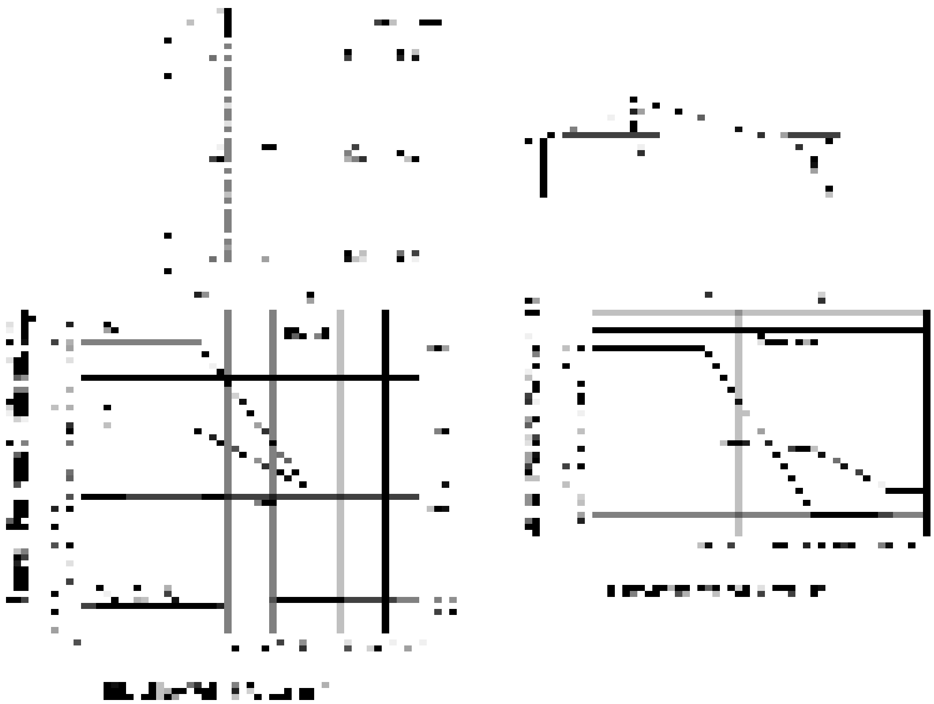
   a: 10 percent of least horizontal dimension or *0.4h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9m.

   h: Mean roof height, in meters, except that eave height shall be used for $\theta \leq 10^0$.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 6.2.11(a) External pressure coefficients, GC_p for components and cladding–Method 2 ($h \leq 18.3$ m)

Enclosed, Partially Enclosed Buildings: Gable Roofs θ ≤ 70



Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area, in square meters.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. If a parapet equal to or higher than 0.9 m is provided around the perimeter of the roof with $\theta \le 7^0$, the negative values of GC_p in Zone 3 shall be equal to those for Zone 2 and positive values of GC_P in Zones 2 and 4 shall be set equal to those for wall Zones 4 and 5 respectively in Figure 6.2.11(a).
6. Values of GC_P for roof overhangs include pressure contributions from both upper and lower surfaces.
7. Notation:

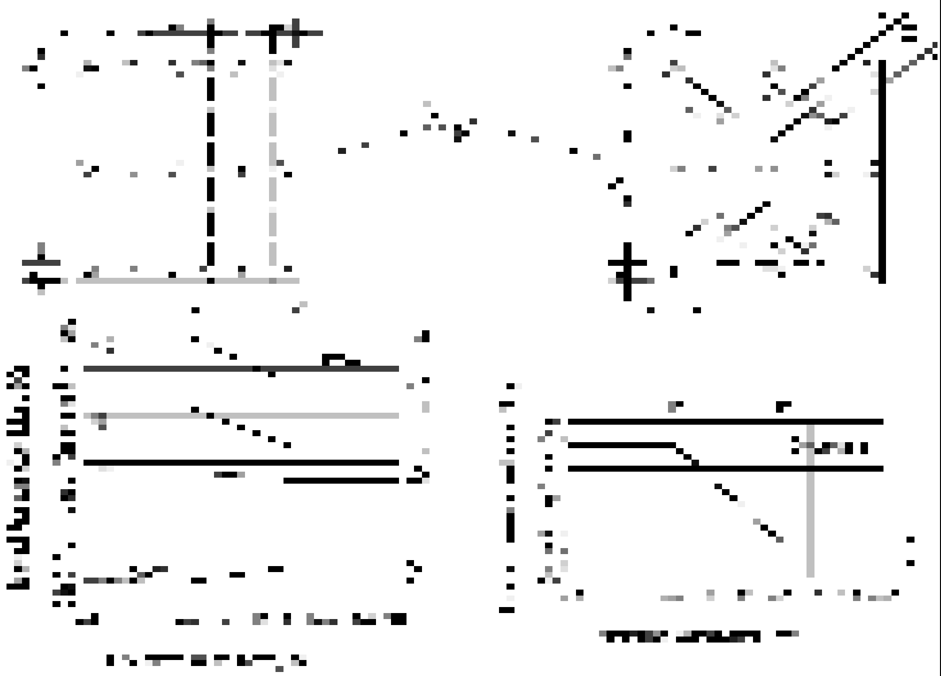
   a: 10 percent of least horizontal dimension or *0.4h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

   h: Eave height shall be used for $\theta \le 10^0$.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 6.2.11(b) External pressure coefficients, GC_p for components and cladding–Method 2 ($h \le 18.3$ m)

Enclosed, Partially Enclosed Buildings: Gable/Hip Roofs $7^0 < \theta \le 27^0$



Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_P for roof overhangs include pressure contributions from both upper and lower surfaces.
6. For hip roofs with $7^0 < \theta \le 27^0$, edge/ridge strips and pressure coefficients for ridges of gabled roofs shall apply on each hip.
7. For hip roofs with $7^0 < \theta \le 25^0$, Zone 3 shall be treated as Zone 2.
8. Notation:

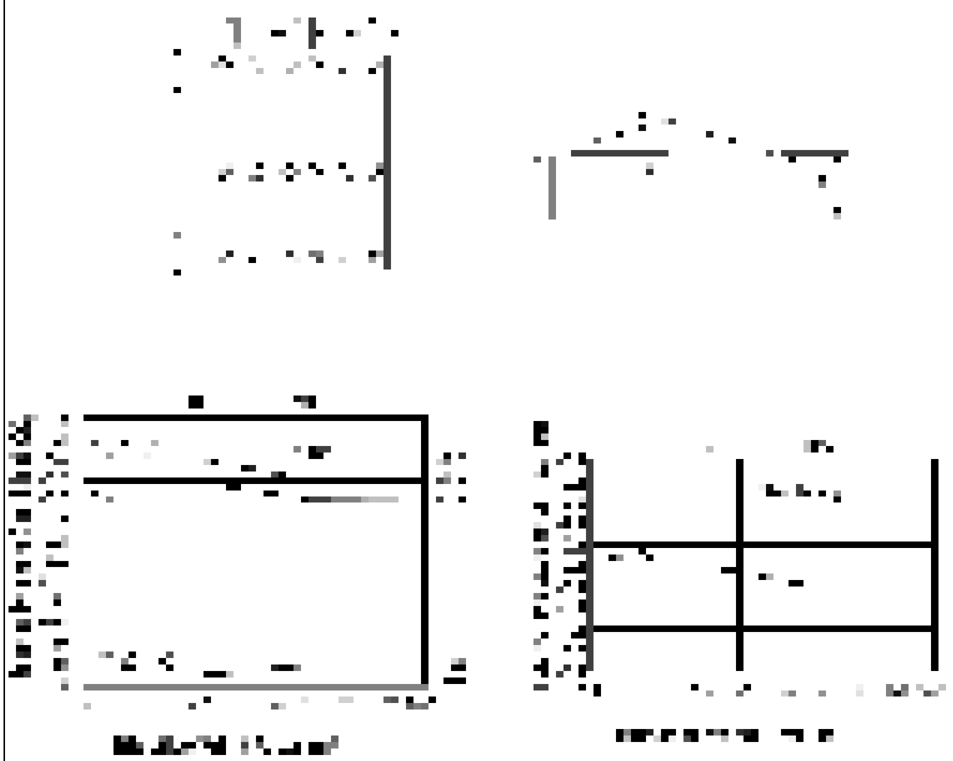
   a: 10 percent of least horizontal dimension or *0.4h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

   h: Mean roof height, in meters, except that eave height shall be used for $\theta \le 10^0$.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 6.2.11(c) External pressure coefficients, GC_p for components and cladding–Method 2 ($h \le 18.3$ m)

Enclosed, Partially Enclosed Buildings: Gable Roofs $27^0 < \theta \le 45^0$



Notes:

1. Vertical scale denotes *GC_P* to be used with *q_h.*
2. Horizontal scale denotes effective wind area, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of *GC_P* for roof overhangs include pressure contributions from both upper and lower surfaces.
6. Notation:

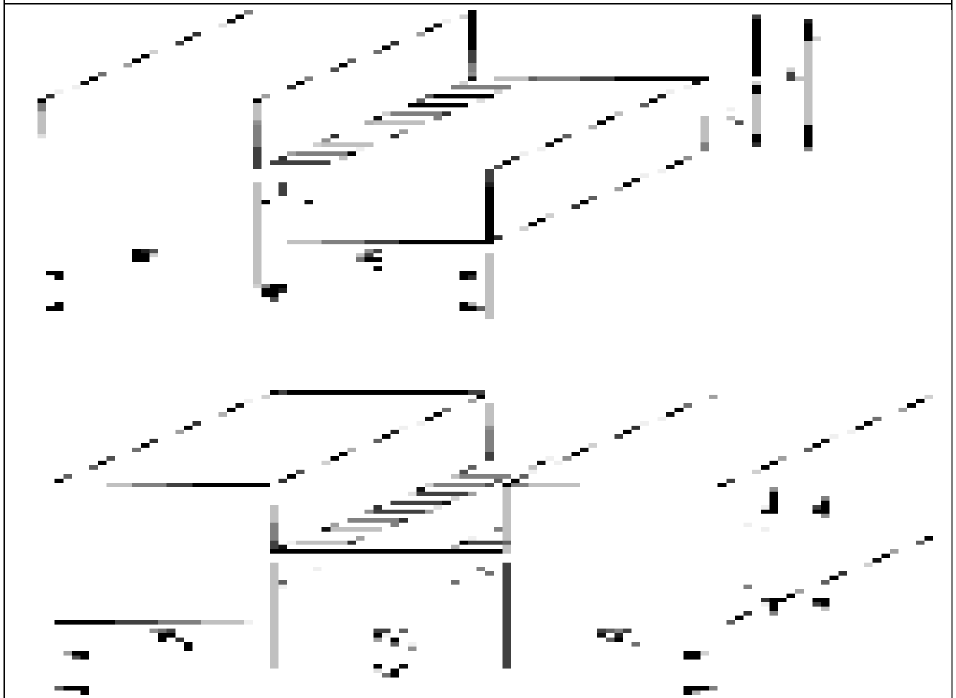
   a: 10 percent of least horizontal dimension or *0.4h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9m.

   h: Mean roof height, in meters.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 6.2.11(d) External pressure coefficients, GC_p for components and cladding–Method 2 ($h \le 18.3$ m)

Enclosed, Partially Enclosed Buildings: Stepped Roofs



Notes:

On the lower level of flat, stepped roofs shown in Figure 6.2.12, the zone designations and pressure coefficients shown in Figure 6.2.11(b) shall apply, except that at the roof-upper wall intersection(s), Zone 3 shall be treated as Zone 2 and Zone 2 shall be treated as Zone 1. Positive values of GC_p equal to those for walls in Figure 6.2.11(a) shall apply on the cross-hatched areas shown in Figure 6.2.12.

Notation:

b: 1.5h_1 in Figure 6.2.12, but not greater than 30.5 m.

h: Mean roof height, in meters.

h_i: h_1 or h_2 in Figure 6.2.12; $h = h_1 + h_2$; $h_1 \geq$ 3.1 m; h_i/h = 0.3 to 0.7.

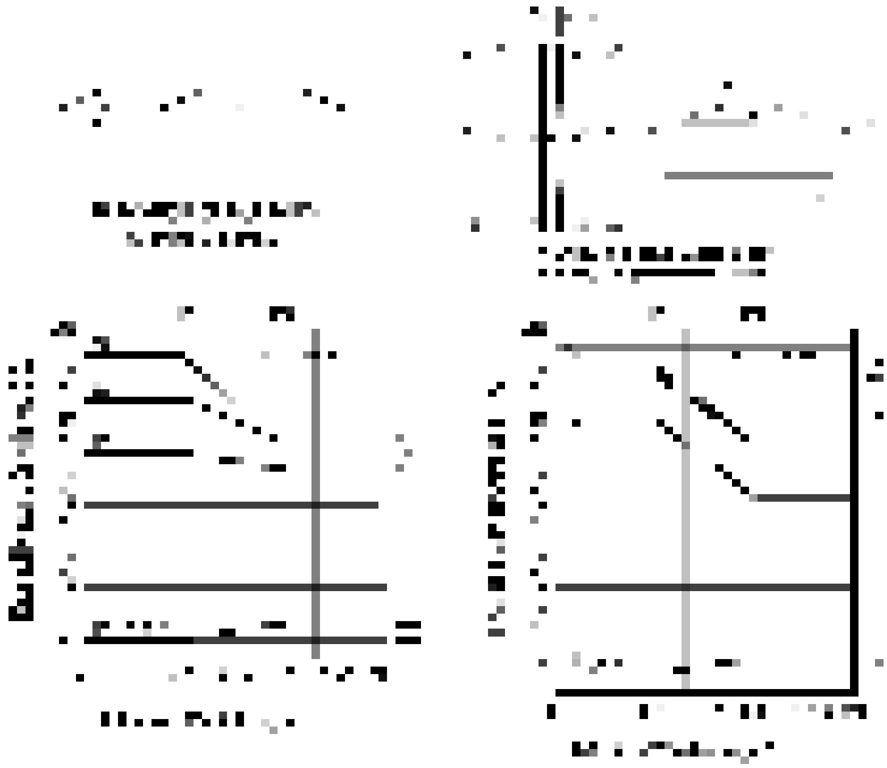
W: Building width in Figure 6.2.12.

W_i: W_1 or W_2 or W_3 in Figure 6.2.12. $W = W_1 + W_2$ or $W_1 + W_2 + W_3$; W_i/W = 0.25 to 0.75.

e: Angle of plane of roof from horizontal, in degrees.

Figure 6.2.12 External pressure coefficients, GC_p for components and cladding–Method 2 ($h \leq 18.3$ m)

Enclosed, Partially Enclosed Buildings: Multispan Gable Roofs



Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area, in square meters.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For $\theta \leq 10^0$ Values of GC_P from Figure 6.2.11 shall be used.
6. Notation:

   a: 10 percent of least horizontal dimension or 0.4*h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

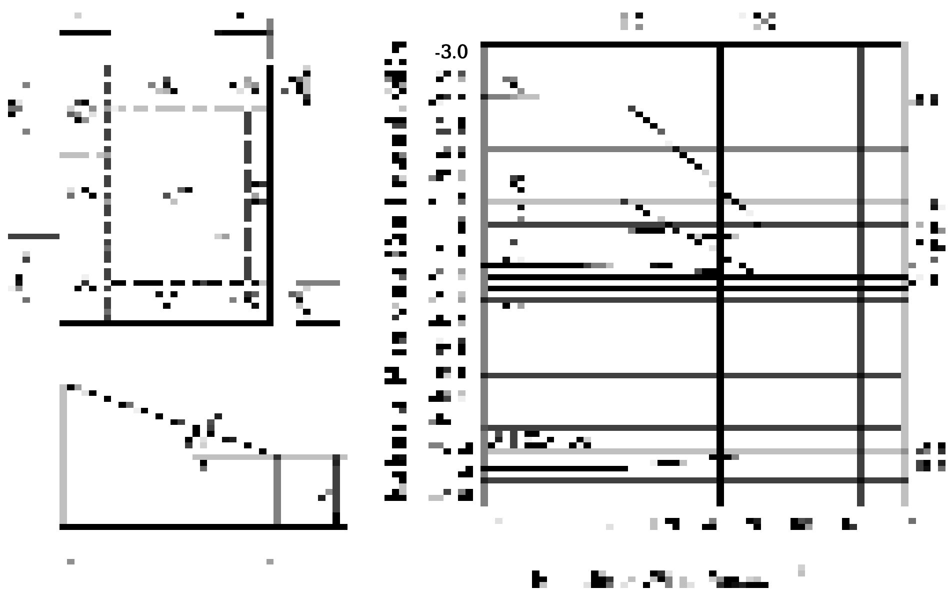
   h: Mean roof height, in feet (meters), except that eave height shall be used for $\theta \leq 10^0$.

   W: Building module width, in meters.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 6.2.13 External pressure coefficients, GC_p for components and cladding–Method 2 ($h \leq 18.3$ m)

Enclosed, Partially Enclosed Buildings: Monoslope Roofs $3^0 < \theta \leq 10^{0z}$



Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area *A,* in square meters.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For θ ≤ 30° Values of GC_P from Figure 6.2.11(b) shall be used.
6. Notation:

   a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

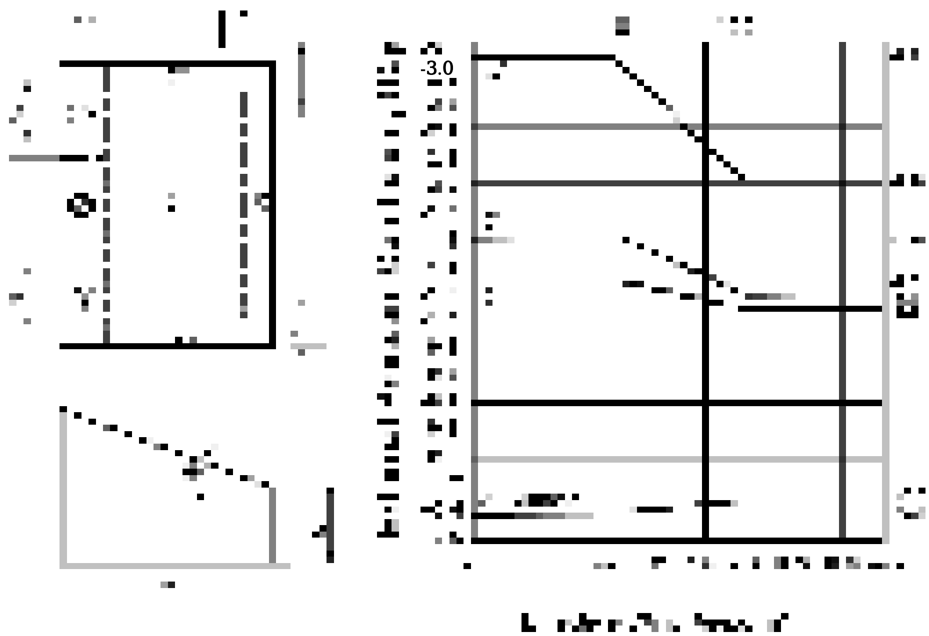
   h: Eave height shall be used for $\theta \leq 10^0$.

   W: Building width, in meters.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 6.2.14(a) External pressure coefficients, GC_p for components and cladding–Method 2 ($h \leq 18.3$ m)

Enclosed, Partially Enclosed Buildings: Monoslope Roofs $10^0 < \theta \le 30^0$



Notes:

1. Vertical scale denotes GC_P to be used with q_h
2. Horizontal scale denotes effective wind area *A*, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Notation:

   a: 10 percent of least horizontal dimension or 0.4*h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

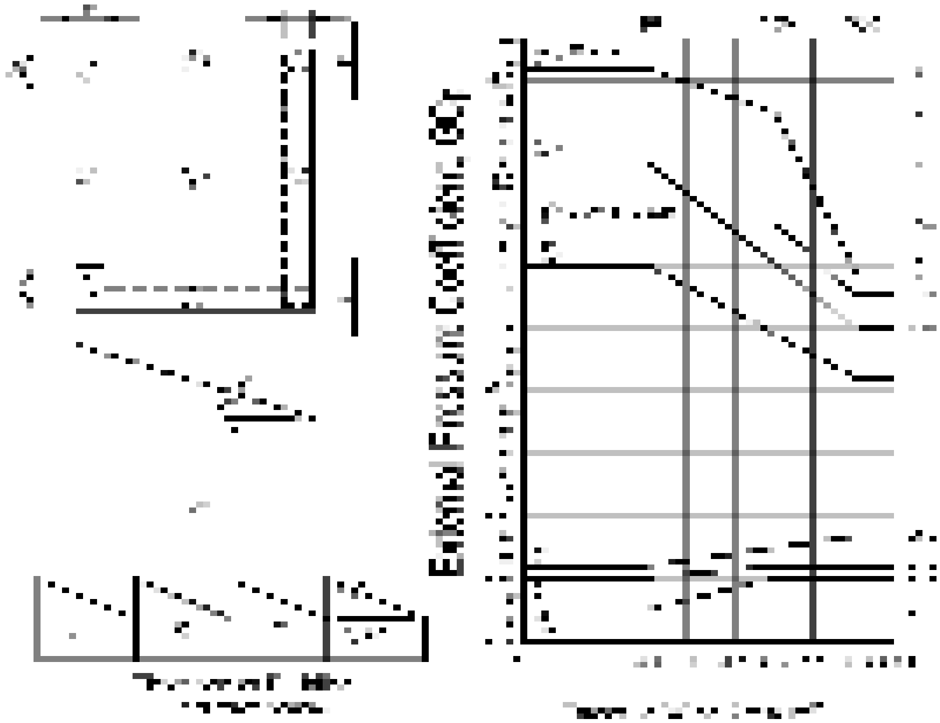
   h: Mean roof height in meters.

   W: Building width, in meters.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 6.2.14(b) External pressure coefficients, GC_p for components and cladding– Method 2 (h ≤ 18.3 m)

Enclosed, Partially Enclosed Buildings: Sawtooth Roofs



Notes:

1. Vertical scale denotes GC_P to be used with q_h.
2. Horizontal scale denotes effective wind area A, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. For $\theta \leq 10^0$ Values of GC_P from Figure 6.2.11 shall be used.
6. Notation:

   a: 10 percent of least horizontal dimension or 0.4*h*, whichever is smaller, but not less than either 4% of least horizontal dimension or 0.9 m.

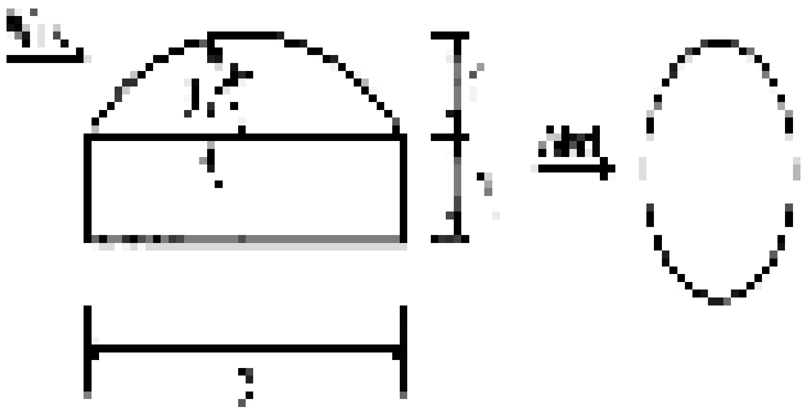
   h: Mean roof height in meters except that eave height shall be used for $\theta \leq 10^0$.

   W: Building width, in meters.

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 6.2.15 External pressure coefficients, GC_p for components and cladding–Method 2 ($h \leq 18.3$ m)

Enclosed, Partially Enclosed Buildings: Domed Roofs



External Pressure Coefficients for Domes with a circular Base			
θ, degrees	**Negative Pressures**	**Positive Pressures**	**Positive Pressures**
	0 – 90	0 – 60	61 – 90
GC_p	-0.9	+0.9	+0.5

Notes:

1. Values denote C_p to be used with $q_{(hD+f)}$ where h_D+f is the height at the top of the dome.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. Each component shall be designed for maximum positive and negative pressures.
4. Values apply to $\theta \leq h_D D \leq 0.5$, $0.2 \leq f/D \leq 0.5$.
5. $\theta = 0^{o}$ on dome springline, $\theta = 90^{o}$ at dome center top point. f is measured from springline to top.

Figure 6.2.16 External pressure coefficients, GC_p for components and cladding – Method 2 (All heights)

Enclosed, Partially Enclosed Buildings: Walls & Roofs

Notes:

1. Vertical scale denotes GC_p to be used with appropriate q_z or q_h.
2. Horizontal scale denotes effective wind area A, in square feet (square meters).
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Use q_z with positive values of GC_p and q_h with negative values of GC_p
5. Each component shall be designed for maximum positive and negative pressures.
6. Coefficients are for roofs with angle θ≤10°. For other roof angles and geometry, use GC_p values from Figure 6.2.11 and attendant q_h based on exposure defined in Sec 2.4.6.
7. If a parapet equal to or higher than 0.9 m is provided around the perimeter of the roof with $\theta \leq 10°$ Zone 3 shall be treated as Zone 2.
8. Notation:

   a: 10 percent of least horizontal dimension, but not less than 0.9 m.

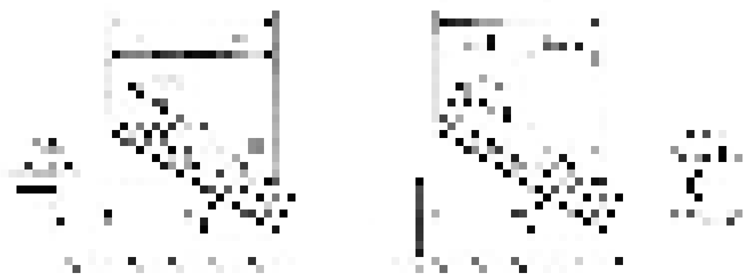
   h: Mean roof height, in meters, except that eave height shall be used for θ≤10°.

   z: height above ground, in (meters).

   θ: Angle of plane of roof from horizontal, in degrees.

Figure 6.2.17 External pressure coefficients, GC_p for components and cladding – Method 2 (h ≤ 18.3 m)

Open Buildings: Monoslope free roofs ($q \leq 45°$, $\gamma = 0°, 180°$)



Roof Angle θ	Load Case	Wind Direction, γ = 0°				Wind Direction, γ = 180°			
		Clear Wind Flow		Obstructed Wind Flow		Clear Wind Flow		Obstructed Wind Flow	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}	C_{NW}	C_{NL}	C_{NW}	C_{NL}
0°	A	1.2	0.3	-0.5	-1.2	1.2	0.3	-0.5	-1.2
	B	-1.1	-0.1	-1.1	-0.6	-1.1	-0.1	-1.1	-0.6
7.5°	A	-0.6	-1	-1	-1.5	0.9	1.5	-0.2	-1.2
	B	-1.4	0	-1.7	-0.8	1.6	0.3	0.8	-0.3
15°	A	-0.3	-1.3	-1.1	-1.5	1.3	1.6	0.4	-1.1
	B	-1.9	0	-2.1	-0.6	1.8	0.6	1.2	-0.3
22.5°	A	-1.5	-1.6	-1.5	-1.7	1.7	1.8	0.5	-1
	B	-2.4	-0.3	-2.3	-0.9	2.2	0.7	1.3	0
30°	A	-1.8	-1.8	-1.5	-1.8	2.1	2.1	0.6	-1
	B	-2.5	-0.6	-2.3	-1.1	2.6	1	1.6	0.1
37.5°	A	-1.8	-1.8	-1.5	-1.8	2.1	2.2	0.7	-0.9
	B	-2.4	-0.6	-2.2	-1.1	2.7	1.1	1.9	0.3
45°	A	-1.6	-1.8	-1.3	-1.8	2.2	2.5	0.8	-0.9
	B	-2.3	-0.7	-1.9	-1.2	2.6	1.4	2.1	0.4

Notes:

1. C_{NW} and C_{NL} denote net pressures (contributions from top and bottom surfaces) for windward and leeward half of roof surfaces, respectively.
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of e between 7.5° and 45°, linear interpolation is permitted. For values of e less than 7.5°, use Monoslope roof load coefficients.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. All load cases shown for each roof angle shall be investigated.
6. Notation:

   L : horizontal dimension of roof, measured in the along wind direction, m

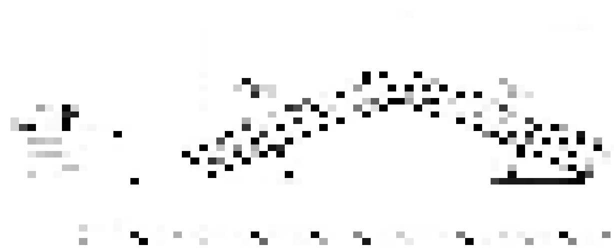
   h : mean roof height, m

   γ : direction of wind, degrees

   θ : angle of plane of roof from horizontal, degrees

Figure 6.2.18(a) Net pressure coefficient, C_N for main wind force resisting system ($0.25 \leq h/L \leq 1.0$)

Open Buildings: Pitched Free Roofs ($\theta \leq 45^o$, $\gamma = 0^o$, 180^o)



Roof Angle, θ	Load Case	Wind Direction, $\gamma = 0^o, 180^o$			
		Clear Wind Flow		Obstructed Wind Flow	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}
7.5^o	A	1.1	-0.3	-1.6	-1
	B	0.2	-1.2	-0.9	-1.7
15^o	A	1.1	-0.4	-1.2	-1
	B	0.1	-1.1	-0.6	-1.6
22.5^o	A	1.1	0.1	-1.2	-1.2
	B	-0.1	-0.8	-0.8	-1.7
30^o	A	1.3	0.3	-0.7	-0.7
	B	-0.1	-0.9	-0.2	-1.1
37.5^o	A	1.3	0.6	-0.6	-0.6
	B	-0.2	-0.6	-0.3	-0.9
45^o	A	1.1	0.9	-0.5	-0.5
	B	-0.3	-0.5	-0.3	-0.7

Notes:

1. C_{NW} and C_{NL} denote net pressures (contributions from top and bottom surfaces) for windward and leeward half of roof surfaces, respectively.
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of θ between 7.5° and 45°, linear interpolation is permitted. For values of θ less than 7.5°, use monoslope roof load coefficients.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. All load cases shown for each roof angle shall be investigated.
6. Notation:
   L : horizontal dimension of roof, measured in the along wind direction, m
   h : mean roof height, m
   γ : direction of wind, degrees
   θ : angle of plane of roof from horizontal, degrees

Figure 6.2.18(b) Net pressure coefficient, C_N for main wind force resisting system ($0.25 \leq h/L \leq 1.0$)

Open Buildings: Troughed Free Roofs ($\theta \leq 45^o$, $\gamma = 0^o, 180^o$)

Roof Angle, θ	Load Case	Wind Direction, γ=0°, 180°			
		Clear Wind Flow		Obstructed Wind Flow	
		C_{NW}	C_{NL}	C_{NW}	C_{NL}
7.5°	A	-1.1	0.3	-1.6	-0.5
	B	-0.2	1.2	-0.9	-0.8
15°	A	-1.1	0.4	-1.2	-0.5
	B	0.1	1.1	-0.6	-0.8
22.5°	A	-1.1	-0.1	-1.2	-0.6
	B	-0.1	0.8	-0.8	-0.8
30°	A	-1.3	-0.3	-1.4	-0.4
	B	-0.1	0.9	-0.2	-0.5
37.5°	A	-1.3	-0.6	-1.4	-0.3
	B	0.2	0.6	-0.3	-0.4
45°	A	-1.1	-0.9	-1.2	-0.3
	B	0.3	0.5	-0.3	-0.4

Notes:

1. C_{NW} and C_{NL} denote net pressures (contributions from top and bottom surfaces) for windward and leeward half of roof surfaces, respectively.
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of θ between 7.5° and 45°, linear interpolation is permitted. For values of θ less than 7.5°, use monoslope roof load coefficients.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. All load cases shown for each roof angle shall be investigated.
6. Notation:

   L : horizontal dimension of roof, measured in the along wind direction, m

   h : mean roof height, m

   γ : direction of wind, degrees

   θ : angle of plane of roof from horizontal, degrees

Figure 6.2.18(c) Net pressure coefficient, C_N for main wind force resisting system ($0.25 \leq h/L \leq 1.0$)

Open Buildings: Troughed Free Roofs ($\theta \leq 45^{o}$, $\lambda = 0^{o}$, 180^{o})

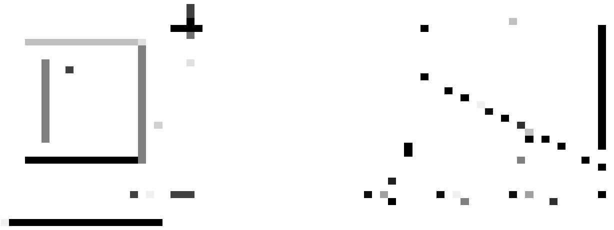
Horizontal Distance from Windward Edge	Roof Angle θ	Load Case	Clear Wind Flow	Obstructed Wind Flow
			C_N	C_N
$\leq h$	All Shapes	A	-0.8	-1.2
	$\theta \leq 45^{o}$	B	0.8	0.5
$> h, \leq 2h$	All Shapes	A	-0.6	-0.9
	$\theta \leq 45^{o}$	B	0.5	0.5
$> 2h$	All Shapes	A	-0.3	-0.6
	$\theta \leq 45^{o}$	B	0.3	0.3

Notes:

1. C_N denotes net pressures (contributions from top and bottom surfaces).
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
4. All load cases shown for each roof angle shall be investigated.
5. For monoslope roofs with theta less than 5 degrees, C_N values shown apply also for cases where gamma = 0 degrees and 0.05 less than or equal to h/L less than or equal to 0.25. See Figure 6.2.18(a) for other h/L values.
6. Notation:
   - L : horizontal dimension of roof, measured in the along wind direction, m
   - h : mean roof height, m
   - y : direction of wind, degrees
   - θ : angle of plane of roof from horizontal, degrees

Figure 6.2.18(d) Net pressure coefficient, C_N for main wind force resisting system ($0.25 \leq h/L \leq 1.0$)

Open Buildings: Monoslope Free Roofs ($\theta \leq 45°$)



Roof Angle θ	Effective Wind Area	C_N											
		Clear Wind Flow						Obstructed Wind Flow					
		Zone 3		Zone 2		Zone 1		Zone 3		Zone 2		Zone 1	
0°	$\leq a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-3.6	0.8	-1.8	0.5	-1.2
	$>a^2, \leq 4.0a^2$	1.8	-1.7	1.8	-1.7	1.2	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$>4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
7.5°	$\leq a^2$	3.2	-4.2	2.4	-2.1	1.6	-1.4	1.6	-5.1	0.5	-2.6	0.8	-1.7
	$>a^2, \leq 4.0a^2$	2.4	-2.1	2.4	-2.1	1.6	-1.4	1.2	-2.6	1.2	-2.6	0.8	-1.7
	$>4.0a^2$	1.6	-1.4	1.6	-1.4	1.6	-1.4	0.8	-1.7	0.8	-1.7	0.8	-1.7
15°	$\leq a^2$	3.6	-3.8	2.7	-2.9	1.8	-1.9	2.4	-4.2	1.8	-3.2	1.2	-2.1
	$>a^2, \leq 4.0a^2$	2.7	-2.9	2.7	-2.9	1.8	-1.9	1.8	-3.2	1.8	-3.2	1.2	-2.1
	$>4.0a^2$	1.8	-1.9	1.8	-1.9	1.8	-1.9	1.2	-2.1	1.2	-2.1	1.2	-2.3
30°	$\leq a^2$	5.2	-5	3.9	-3.8	2.6	-2.5	3.2	-4.6	2.4	-3.5	1.6	-2.3
	$>a^2, \leq 4.0a^2$	3.9	-3.8	3.9	-3.8	2.6	-2.5	2.4	-3.5	2.4	-3.5	1.6	-2.3
	$>4.0a^2$	2.6	-2.5	2.6	-2.5	2.6	-2.5	1.6	-2.3	1.6	-2.3	1.6	-2.3
45°	$\leq a^2$	5.2	-4.6	3.9	-3.5	2.6	-2.3	4.2	-3.8	3.2	-2.9	2.1	-1.9
	$>a^2, \leq 4.0a^2$	3.9	-3.5	3.9	-3.5	2.6	-2.3	3.2	-2.9	3.2	-2.9	2.1	-1.9
	$>4.0a^2$	2.6	-2.3	2.6	-2.3	2.6	-2.3	2.1	-1.9	2.1	-1.9	2.1	-1.9

Notes:

1. C_N denotes net pressures (contributions from top and bottom surfaces).
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50% wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of e other than those shown, linear interpolation is permitted.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. Components and cladding elements shall be designed for positive and negative pressure coefficients shown.
6. Notation:

   α : 10% of least horizontal dimension or $0.4h$, whichever is smaller but not less than 4% of least horizontal dimension or 0.9 m

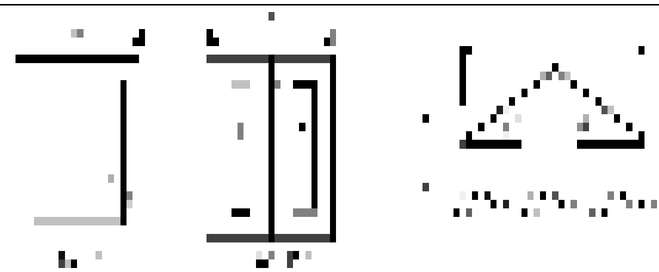
   h : mean roof height, m

   L : horizontal dimension of building, measured in along wind direction, m

   θ: angle of plane of roof from horizontal, degrees

Figure 6.2.19(a) Net pressure coefficient, C_N for components and cladding ($0.25 \leq h/L \leq 1.0$)

Open Buildings: Monoslope Free Roofs ($\theta \leq 45^o$)



Roof Angle θ	Effective Wind Area	C_N											
		Clear Wind Flow						Obstructed Wind Flow					
		Zone 3		Zone 2		Zone 1		Zone 3		Zone 2		Zone 1	
0°	$\leq a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-3.6	0.8	-1.8	0.5	-1.2
	$>a^2, \leq 4.0a^2$	1.8	-1.7	1.8	-1.7	1.2	-1.1	08	-1.8	0.8	-1.8	0.5	-1.2
	$>4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
7.5°	$\leq a^2$	2.2	-3.6	1.7	-1.8	1.1	-1.2	1	-5.1	0.8	-26	0.5	-1.7
	$>a^2, \leq 4.0a^2$	1.7	-1.8	1.7	-1.8	1.1	-1.2	0.8	-2.6	0.8	·26	0.5	-1.7
	$>4.0a^2$	1.1	-1.2	1.1	-1.2	1.1	-1.2	0.5	-1.7	0.5	-1.7	as	-1.7
15°	$\leq a^2$	2.2	-2.2	1.7	-1.7	1.1	-1.1	1	-3.2	0.8	-2.4	0.5	-1.6
	$>a^2, \leq 4.0a^2$	1.7	-1.7	1.7	-1.7	1.1	-1.1	0.8	-2.4	0.8	-2.4	0.5	-1.6
	$>4.0a^2$	1.1	-1.1	1.1	-1.1	1.1	-1.1	0.5	-1.6	0.5	-1.6	0.5	-1.6
30°	$\leq a^2$	2.6	-1.8	2	-1.4	1.3	-0.9	1	-2.4	0.8	-1.8	0.5	-1.2
	$>a^2, \leq 4.0a^2$	2	-1.4	2	-1.4	1.3	-0.9	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$>4.0a^2$	1.3	-0.9	1.3	-0.9	1.3	-0.9	0.5	-1.2	0.5	.1.2	0.5	-1.2
45°	$\leq a^2$	2.2	-1.6	1.7	-1.2	1.1	-0.8	1	-2.4	0.8	-1.8	0.5	-1.2
	$>a^2, \leq 4.0a^2$	1.7	-1.2	1.7	-1.2	1.1	-0.8	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$>4.0a^2$	1.1	-0.8	1.1	-0.8	1.1	-0.8	0.5	-1.2	0.5	-1.2	0.5	-1.2

Notes:

1. C_N denotes net pressures (contributions from top and bottom surfaces).
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of θ other than those shown, linear interpolation is permitted.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. Components and cladding elements shall be designed for positive and negative pressure coefficients shown.
6. Notation:

   α : 10% of least horizontal dimension or 0.411, whichever is smaller but not less than 4% of least horizontal dimension or 0.9 m

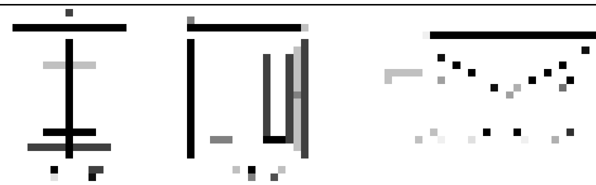
   h : mean roof height, m

   L : horizontal dimension of building, measured in along wind direction, m

   θ : angle of plane of roof from horizontal, degrees

Figure 6.2.19(b) Net pressure coefficient, C_N for components and cladding ($0.25 \leq h/L \leq 1.0$)

Open Buildings: Troughed Free Roofs ($\theta \le 45^o$)



Roof Angle θ	Effective Wind Area	C_N											
		Clear Wind Flow						Obstructed Wind Flow					
		Zone 3		Zone 2		Zone 1		Zone 3		Zone 2		Zone 1	
0°	$\le a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-3.6	0.8	-1.8	0.5	-1.2
	$>a^2, \le 4.0a^2$	1.8	-1.7	1.8	-1.7	1.1	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$>4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2
7.5°	$\le a^2$	2.4	-3.3	1.8	-1.7	1.2	-1.1	1	-4.8	0.8	-2.4	0.5	-1.6
	$>a^2, \le 4.0a^2$	1.8	-1.7	1.8	-1.7	1.2	-1.1	0.8	-2.4	0.8	-2.4	0.5	-1.6
	$>4.0a^2$	1.2	-1.1	1.2	-1.1	1.2	-1.1	0.5	-1.6	0.5	-1.6	0.5	-1.6
15°	$\le a^2$	2.2	-2.2	1.7	-1.7	1.1	-1.1	1	-2.4	0.8	-1.8	0.5	-1.2
	$>a^2, \le 4.0a^2$	1.7	-1.7	1.7	-1.7	1.1	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$>4.0a^2$	1.1	-1.1	1.1	-1.1	1.1	-1.1	0.5	-1.2	0.5	-12	0.5	-1.2
30°	$\le a^2$	1.8	-2.6	1.4	-2	0.9	-1.3	1	-2.8	0.8	-2.1	0.5	-1.4
	$>a^2, \le 4.0a^2$	1.4	-2	1.4	-2	0.9	-1.3	0.8	-2.1	0.8	-2.1	0.5	-1.4
	$>4.0a^2$	0.9	-1.3	1.9	-1.3	0.9	-1.3	0.5	-1.4	0.5	-1.4	0.5	-1.4
45°	$\le a^2$	1.6	-2.2	1.2	-1.7	0.8	-1.1	1	-2.4	0.8	-1.8	0.5	-1.2
	$>a^2, \le 4.0a^2$	1.2	-1.7	1.2	-1.7	0.8	-1.1	0.8	-1.8	0.8	-1.8	0.5	-1.2
	$>4.0a^2$	0.8	-1.1	1.8	-1.1	0.8	-1.1	0.5	-1.2	0.5	-1.2	0.5	-1.2

Notes:

1. C_N denotes net pressures (contributions from top and bottom surfaces).
2. Clear wind flow denotes relatively unobstructed wind flow with blockage less than or equal to 50%. Obstructed wind flow denotes objects below roof inhibiting wind flow (>50% blockage).
3. For values of θ other than those shown, linear interpolation is permitted.
4. Plus and minus signs signify pressures acting towards and away from the top roof surface, respectively.
5. Components and cladding elements shall be designed for positive and negative pressure coefficients shown.
6. Notation:

   α : 10% of least horizontal dimension or 0.411, whichever is smaller but not less than 4% of least horizontal dimension or 0.9 m

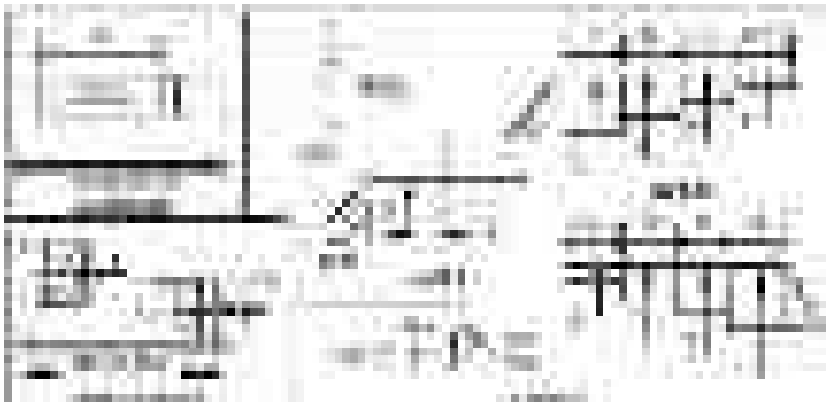
   h : mean roof height, m

   L : horizontal dimension of building, measured in along wind direction, m

   θ : angle of plane of roof from horizontal, degrees

Figure 6.2.19(c) Net pressure coefficient, C_N for components and cladding ($0.25 \le h/L \le 1.0$)

Solid Freestanding Walls & Solid Signs



C_f, CASE A & CASE B

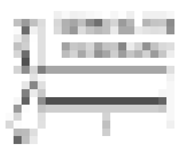
Clearance Ratio, *s/h*	Aspect Ratio, *B/s*											
	≤0.05	**0.1**	**0.2**	**0.5**	**1**	**2**	**4**	**5**	**10**	**20**	**30**	**≥45**
1	1.80	1.70	1.65	1.55	1.45	1.40	1.35	1.35	1.30	1.30	1.30	1.30
0.9	1.85	1.75	1.70	1.60	1.55	1.50	1.45	1.45	1.40	1.40	1.40	1.40
0.7	1.90	1.85	1.75	1.70	1.65	1.60	1.60	1.55	1.55	1.55	1.55	1.55
0.5	1.95	1.85	1.80	1.75	1.75	1.70	1.70	1.70	1.70	1.70	1.70	1.75
0.3	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.80	1.85	1.85	1.85
0.2	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.85	1.90	1.90	1.95
≤0.16	1.95	1.90	1.85	1.85	1.80	1.80	1.85	1.85	1.85	1.90	1.90	1.95

C_f, CASE C

Region (horizontal distance from windward edge)	**Aspect Ratio, *B/s***									**Region (horizontal distance from windward edge)**	**Aspect Ratio, B/s**	
	2	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**		**13**	**≥45**
0 to *s*	2.25	2.60	2.90	3.10*	3.30*	3.40*	3.55*	3.65*	3.75*	0 to *s*	4.00*	4.30*
s to *2s*	1.50	1.70	1.90	2.00	2.15	2.25	2.30	2.35	2.45	*s* to *2s*	2.60	2.55
2s to *3s*		1.15	1.30	1.45	1.55	1.65	1.70	1.75	1.85	*2s* to *3s*	2.00	1.95
3s to *10s*			1.10	1.05	1.05	1.05	1.05	1.00	0.95	*3s* to *4s*	1.50	1.85
										4s to 5s	1.35	1.85
										5s to 10s	0.90	1.10
										>10s	0.55	0.55

*Values shall be multiplied by the following reduction factor when a return corner is present:

L_r/s	**Reduction Factor**
0.3	0.9
1.0	0.75
≥2	0.60



Notes:

1. The term "signs" in notes below also applies to "freestanding walls".
2. Signs with openings comprising less than 30% of the gross area are classified as solid signs. Force coefficients for solid signs with openings shall be permitted to be multiplied by the reduction factor $(1 - (1 - \varepsilon)^{1.5})$.
3. To allow for both normal and oblique wind directions, the following cases shall be considered:

   For $s/h < 1$:

   CASE A: resultant force acts normal to the face of the sign through the geometric center.

   CASE B: resultant force acts normal to the face of the sign at a distance from the geometric center toward the windward edge equal to 0.2 times the average width of the sign.

   For $B/s \geq 2$, CASE C must also be considered:

   CASE C: resultant forces act normal to the face of the sign through the geometric centers of each region.

   For $s/h = 1$:

   The same cases as above except that the vertical locations of the resultant forces occur at a distance above the geometric center equal to 0.05 times the average height of the sign.
4. For CASE C where $s/h > 0.8$, force coefficients shall be multiplied by the reduction factor $(1.8 - s/h)$.
5. Linear interpolation is permitted for values of s/h, B/s and L_r/s other than shown.
6. Notation: B: horizontal dimension of sign, in meters;

   h : height of the sign, in meters;

   s: vertical dimension of the sign, in meters;

   ε: ratio of solid area to gross area;

   L_r: horizontal dimension of return corner, in meters

Figure 6.2.20 Force Coefficient, C_f for other structures - Method 2 (All heights)

Chimneys, Tanks, Rooftop Equipment, & Similar Structures

Cross-Section	Type of Surface	h/D		
		1	7	25
Square (wind normal to face)	All	1.3	1.4	2.0
Square (wind along diagomal)	All	1.0	1.1	1.5
Hexagonal or octagonal	All	1.0	1.2	1.4
Round $D\sqrt{q_z} > 5.3, D$ in m, q_z in $\mathrm{N/m^2}$	Moderately smooth	0.5	0.6	0.7
	Rough (D'/D=0.02)	0.7	0.8	0.9
	Very rough (D'/D=0.08)	0.8	1.0	0.2
Round $D\sqrt{q_z} \leq 5.3, D$ in m, q_z in $\mathrm{N/m^2}$	All	0.7	0.8	1.2

Notes:

1. The design wind force shall be calculated based on the area of the structure projected on a plane normal to the wind direction. The force shall be assumed to act parallel to the wind direction.
2. Linear interpolation is permitted for h/D values other than shown.
3. Notation:

   D: diameter of circular cross-section and least horizontal dimension of square, hexagonal or octagonal cross-section at elevation under consideration, in meters;

   D': depth of protruding element such as ribs and spoilers, in meters;

   H: height of structure, meters and

   q_z: velocity pressure evaluated at height z above ground, in N/m^2

Figure 6.2.21 Force coefficient, C_f for other structures - Method 2 (All heights)

Open Signs & Lattice Frameworks

$\in$	Flat-Sided Members	Rounded Members	
		$(D\sqrt{q_z} \leq 5.3,)$	$(D\sqrt{q_z} > 5.3,)$
<0.1	2.0	1.2	0.8
0.1 to 0.29	1.8	1.3	0.9
0.3 to 0.7	1.6	1.5	1.1

Notes:

1. Signs with openings comprising 30% or more of the gross area are classified as open signs.
2. The calculation of the design wind forces shall be based on the area of all exposed members and elements projected on a plane normal to the wind direction. Forces shall be assumed to act parallel to the wind.
3. The area A_f consistent with these force coefficients is the solid area projected normal the wind direction.
4. Notation:

   $\in$: ratio of solid area to gross area;

   D: diameter of a typical round number, in meters

   q_z: velocity pressure evaluated at height z above ground in N/m^2.

Figure 6.2.22 Force coefficient, C_f for other structures - Method 2 (All heights)

Table 6.2.8: Basic Wind Speeds, *V*, for Selected Locations in Bangladesh

Location	Basic Wind Speed (m/s)	Location	Basic Wind Speed (m/s)
Angarpota	47.8	Lalmonirhat	63.7
Bagerhat	77.5	Madaripur	68.1
Bandarban	62.5	Magura	65.0
Barguna	80.0	Manikganj	58.2
Barisal	78.7	Meherpur	58.2
Bhola	69.5	Maheshkhali	80.0
Bogra	61.9	Moulvibazar	53.0
Brahmanbaria	56.7	Munshiganj	57.1
Chandpur	50.6	Mymensingh	67.4
Chapai Nawabganj	41.4	Naogaon	55.2
Chittagong	80.0	Narail	68.6
Chuadanga	61.9	Narayanganj	61.1
Comilla	61.4	Narsinghdi	59.7
Cox’s Bazar	80.0	Natore	61.9
Dahagram	47.8	Netrokona	65.6
Dhaka	65.7	Nilphamari	44.7
Dinajpur	41.4	Noakhali	57.1
Faridpur	63.1	Pabna	63.1
Feni	64.1	Panchagarh	41.4
Gaibandha	65.6	Patuakhali	80.0
Gazipur	66.5	Pirojpur	80.0
Gopalganj	74.5	Rajbari	59.1
Habiganj	54.2	Rajshahi	49.2
Hatiya	80.0	Rangamati	56.7
Ishurdi	69.5	Rangpur	65.3
Joypurhat	56.7	Satkhira	57.6
Jamalpur	56.7	Shariatpur	61.9
Jessore	64.1	Sherpur	62.5
Jhalakati	80.0	Sirajganj	50.6
Jhenaidah	65.0	Srimangal	50.6
Khagrachhari	56.7	St. Martin’s Island	80.0
Khulna	73.3	Sunamganj	61.1
Kutubdia	80.0	Sylhet	61.1
Kishoreganj	64.7	Sandwip	80.0
Kurigram	65.6	Tangail	50.6
Kushtia	66.9	Teknaf	80.0
Lakshmipur	51.2	Thakurgaon	41.4

Open Structures: Trussed Tower

Tower Cross Section	C_f
Square	$4.0 \in^2 - 5.9 \in + 4.0$
Triangle	$3.4 \in^2 - 4.7 \in + 3.4$

Notes:

1. For all wind directions considered, the area A_f consistent with the specified force coefficients shall be the solid area of a tower face projected on the plane of that face for the tower segment under consideration.
2. The specified force coefficients are for towers with structural angles or similar flat-sided members.
3. For towers containing rounded members, it is acceptable to multiply the specified force coefficients by the following factor when determining wind forces on such members: $0.51 \in^2 + 0.57 \leq 1.0$
4. Wind forces shall be applied in the directions resulting in maximum member forces and reactions. For towers with square cross-sections, wind forces shall be multiplied by the following factor when the wind is directed along a tower diagonal:

   $1 + 0.75 \in \leq 1.2$
5. Wind forces on tower appurtenances such as ladders, conduits, lights, elevators, etc., shall be calculated using appropriate force coefficients for these elements.
6. Notation:

   $\in$: ratio of solid area to gross area of one tower face for the segment under consideration.

Figure 6.2.23 Force coefficient, C_f for other structures - Method 2 (All heights)

Table 6.2.9: Importance Factor, I (Wind Loads)

Occupancy Category[1] or Importance Class	Non-Cyclone Prone Regions and Cyclone Prone Regions with V = 38-44 m/s	Cyclone Prone Regions with V > 44 m/s
I	0.87	0.77
II	1.0	1.00
III	1.15	1.15
IV	1.15	1.15

[1] The building and structure classification categories are listed in Table 6.1.1

Table 6.2.10: Terrain Exposure Constants

Exposure	α	z_g (m)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	l (m)	$\bar{\in}$	z_{min} (m)*
A	7.0	365.76	1/7	0.84	1/4.0	0.45	0.30	97.54	1/3.0	9.14
B	9.5	274.32	1/9.5	1.00	1/6.5	0.65	0.20	152.4	1/5.0	4.57
C	11.5	213.36	1/11.5	1.07	1/9.0	0.80	0.15	198.12	1/8.0	2.13

*z_{min}= Minimum height used to ensure that the equivalent height z is greater of $0.6h$ or z_{min}.

For buildings with $h \leq z_{min}$, $\bar{z}$ shall be taken as z_{min}.

Table 6.2.11: Velocity Pressure Exposure Coefficients, K_h and K_z

Height above ground level, z	Exposure (Note 1)			
	A		B	C
(m)	Case 1	Case 2	Case 1 & 2	Case 1 & 2
0-4.6	0.70	0.57	0.85	1.03
6.1	0.70	0.62	0.90	1.08
7.6	0.70	0.66	0.94	1.12
9.1	0.70	0.70	0.98	1.16
12.2	0.76	0.76	1.04	1.22
15.2	0.81	0.81	1.09	1.27
18	0.85	0.85	1.13	1.31
21.3	0.89	0.89	1.17	1.34
24.4	0.93	0.93	1.21	1.38
27.41	0.96	0.96	1.24	1.40
30.5	0.99	0.99	1.26	1.43
36.6	1.04	1.04	1.31	1.48
42.7	1.09	1.09	1.36	1.52
48.8	1.13	1.13	1.39	1.55
54.9	1.17	1.17	1.43	1.58
61.0	1.20	1.20	1.46	1.61
76.2	1.28	1.28	1.53	1.68
91.4	1.35	1.35	1.59	1.73
106.7	1.41	1.41	1.64	1.78
121.9	1.47	1.47	1.69	1.82
137.2	1.52	1.52	1.73	1.86
152.4	1.56	1.56	1.77	1.89

Notes:

1. Case 1:

   (a) All components and cladding.

   (b) Main wind force resisting system in low-rise buildings designed using Figure 6.2.10.

   Case 2:

   (a) All main wind force resisting systems in buildings except those in low-rise buildings designed using Figure 6.2.10.

   (b) All main wind force resisting systems in other structures.

2. The velocity pressure exposure coefficient K_z may be determined from the following formula:

   For 4.57 m $\leq z \leq z_g$: $K_z = 2.01\ (z/z_g)^{2/\alpha}$

   For $z <$ 4.57 m: $K_z = 2.01\ (4.57/z_g)^{2/\alpha}$

   Note: z shall not be taken less than 9.1 m for Case 1 in exposure A.

3. α and z_g are tabulated in Table 6.2.10.

4. Linear interpolation for intermediate values of height z is acceptable.

5. Exposure categories are defined in Sec 2.4.6.3.

Table 6.2.12: Wind Directionality Factor, K_d

Structure Type	Directionality Factor K_d *	Structure Type	Directionality Factor K_d *
Buildings		Solid Signs	0.85
Main Wind Force Resisting System	0.85	Open Signs and Lattice Framework	0.85
Components and Cladding	0.85	Trussed Towers	
Arched Roofs	0.85	Triangular, square, rectangular	0.85
Chimneys, Tanks, and Similar Structures		All other cross section	0.95
Square	0.90		
Hexagonal	0.95		
Round	0.95		

* Directionality Factor K_d has been calibrated with combinations of loads specified in Sec 2.7. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.7.2 and 2.7.3.

2.5 Earthquake Loads

2.5.1 General

Minimum design earthquake forces for buildings, structures or components thereof shall be determined in accordance with the provisions of Sec 2.5. Some definitions and symbols relevant for earthquake resistant design for buildings are provided in Sections 2.1.3 and 2.1.4. Section 2.5.2 presents basic earthquake resistant design concepts. Section 2.5.3 describes procedures for soil investigations, while Sec 2.5.4 describes procedures for determining earthquake ground motion for design. Section 2.5.5 describes different types of buildings and structural systems which possess different earthquake resistant characteristics. Static analysis procedures for design are described in Sections 2.5.6, 2.5.7 and 2.5.12. Dynamic analysis procedures are dealt with in Sections 2.5.8 to 2.5.11. Section 2.5.13 presents how seismic effects are accounted in the design and combination of earthquake loading effects in different directions and with other loading effects. Section 2.5.14 deals with allowable drift and deformation limits. Section 2.5.15 addresses design of non-structural components in buildings. Section 2.5.16 presents design considerations for buildings with seismic isolation systems. Design for soft storey condition in buildings is addressed in Sec 2.5.17.

2.5.2 Earthquake Resistant Design – Basic Concepts

2.5.2.1 General principles

The purpose of earthquake resistant design provisions in this Code is to provide guidelines for the design and construction of new structures subject to earthquake ground motions in order to minimize the risk to life for all structures, to increase the expected performance of higher occupancy structures as compared to ordinary structures, and to improve the capability of essential structures to function after an earthquake. It is not economically feasible to design and construct buildings without any damage for a major earthquake event. The intent is therefore to allow inelastic deformation and structural damage at preferred locations in the structure without endangering structural integrity and to prevent structural collapse during a major earthquake.

The seismic zoning map (Fig. 6.2.24) divides the country into four seismic zones with different expected levels of intensity of ground motion. Each seismic zone has a zone coefficient which provides expected peak ground acceleration values on rock/firm soil corresponding to the maximum considered earthquake (MCE). The design basis earthquake is taken as 2/3 of the maximum considered earthquake.

The effects of the earthquake ground motion on the structure is expressed in terms of an idealized elastic design acceleration response spectrum, which depends on (a) seismic zone coefficient and local soil conditions defining ground motion and (b) importance factor and response reduction factor representing building considerations. The earthquake forces acting on the structure is reduced using the response modification/reduction factor R in order to take advantage of the inelastic energy dissipation due to inherent ductility and redundancy in the structure as well as material over-strength. The importance factor I increases design forces for important structures. The provisions of this Code for ductility and detailing need to 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. The elastic deformations calculated under these reduced design forces are multiplied by the deflection amplification factor, C_d to estimate the deformations likely to result from the design earthquake.

The seismic design guidelines presented in this Section are based on the assumption that the soil supporting the structure will not liquefy, settle or slide due to loss of strength during the earthquake. Reinforced and prestressed concrete members shall be suitably designed to ensure that premature failure due to shear or bond does not occur. Ductile detailing of reinforced concrete members is of prime importance. In steel structures, members and their connections should be so proportioned that high ductility is obtained, avoiding premature failure due to elastic or inelastic buckling of any type.

The building structure shall include complete lateral and vertical force-resisting systems capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. The design ground motions shall be assumed to occur along any horizontal direction of a building structure. The adequacy of the structural systems shall be demonstrated through the construction of a mathematical model and evaluation of this model for the effects of design ground motions.

2.5.2.2 Characteristics of Earthquake Resistant Buildings

The desirable characteristics of earthquake resistant buildings are described below:

<u>Structural Simplicity, Uniformity and Symmetry</u>:

Structural simplicity, uniformity and plan symmetry is characterized by an even distribution of mass and structural elements which allows short and direct transmission of the inertia forces created in the distributed masses of the building to its foundation. A building configuration with symmetrical layout of structural elements of the lateral force resisting system, and well-distributed in-plan, is desirable. Uniformity along the height of the building is also important, since it tends to eliminate the occurrence of sensitive zones where concentrations of stress or large ductility demands might cause premature collapse.

Some basic guidelines are given below:

(i) With respect to the lateral stiffness and mass distribution, the building structure shall be approximately symmetrical in plan with respect to two orthogonal axes.

(ii) Both the lateral stiffness and the mass of the individual storeys shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building.

(iii) All structural elements of the lateral load resisting systems, such as cores, structural walls, or frames shall run without interruption from the foundations to the top of the building.

(iv) An irregular building may be subdivided into dynamically independent regular units well separated against pounding of the individual units to achieve uniformity.

(v) The length to breadth ratio ($\lambda = L_{max}/L_{min}$) of the building in plan shall not be higher than 4, where L_{max} and L_{min} are respectively the larger and smaller in plan dimension of the building, measured in orthogonal directions.

<u>Structural Redundancy</u>:

A high degree of redundancy accompanied by redistribution capacity through ductility is desirable, enabling a more widely spread energy dissipation across the entire structure and an increased total dissipated energy. The use of evenly distributed structural elements increases redundancy. Structural systems of higher static indeterminacy may result in higher response reduction factor R.

<u>Horizontal Bi-directional Resistance and Stiffness</u>:

Horizontal earthquake motion is a bi-directional phenomenon and thus the building structure needs to resist horizontal action in any direction. The structural elements of lateral force resisting system should be arranged in an orthogonal (in plan) pattern, ensuring similar resistance and stiffness characteristics in both main directions. The stiffness characteristics of the structure should also limit the development of excessive displacements that might lead to either instabilities due to second order effects or excessive damages.

Torsional Resistance and Stiffness

Besides lateral resistance and stiffness, building structures should possess adequate torsional resistance and stiffness in order to limit the development of torsional motions which tend to stress the different structural elements in a non-uniform way. In this respect, arrangements in which the main elements resisting the seismic action are distributed close to the periphery of the building present clear advantages.

Diaphragm Behaviour

In buildings, floors (including the roof) act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action. Floor systems and the roof should be provided with in-plane stiffness and resistance and with effective connection to the vertical structural systems. Particular care should be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements, thus hindering such effective connection between the vertical and horizontal structure. The in-plane stiffness of the floors shall be sufficiently large in comparison with the lateral stiffness of the vertical structural elements, so that the deformation of the floor shall have a small effect on the distribution of the forces among the vertical structural elements.

Foundation

The design and construction of the foundation and of its connection to the superstructure shall ensure that the whole building is subjected to a uniform seismic excitation. For buildings with individual foundation elements (footings or piles), the use of a foundation slab or tie-beams between these elements in both main directions is recommended, as described in Chapter 3.

2.5.3 Investigation and Assessment of Site Conditions

2.5.3.1 Site investigation

Appropriate site investigations should be carried out to identify the ground conditions influencing the seismic action.

The ground conditions at the building site should normally be free from risks of ground rupture, slope instability and permanent settlements caused by liquefaction or densification during an earthquake. The possibility of such phenomena should be investigated in accordance with standard procedures described in Chapter 3 of this Part.

The intent of the site investigation is to classify the Site into one of types SA, SB, SC, SD, SE, S_1 and S_2 as defined in Sec 2.5.3.2. Such classification is based on site profile and evaluated soil properties (shear wave velocity, Standard Penetration Resistance, undrained shear strength, soil type). The site class is used to determine the effect of local soil conditions on the earthquake ground motion.

For sites representing special soil type S_1 or S_2, site specific special studies for the ground motion should be done. Soil type S_1, having very low shear wave velocity and low material damping, can produce anomalous seismic site amplification and soil-structure interaction effects. For S_2 soils, possibility of soil failure should be studied.

For a structure belonging to Seismic Design Category C or D (Sec 2.5.5.2), site investigation should also include determination of soil parameters for the assessment of the following:

(a) Slope instability.

(b) Potential for Liquefaction and loss of soil strength.

(c) Differential settlement.

(d) Surface displacement due to faulting or lateral spreading.

(e) Lateral pressures on basement walls and retaining walls due to earthquake ground motion.

Liquefaction potential and possible consequences should be evaluated for design earthquake ground motions consistent with peak ground accelerations. Any Settlement due to densification of loose granular soils under design earthquake motion should be studied. The occurrence and consequences of geologic hazards such as slope instability or surface faulting should also be considered. The dynamic lateral earth pressure on basement walls and retaining walls during earthquake ground shaking is to be considered as an earthquake load for use in design load combinations

2.5.3.2 Site classification

Site will be classified as type SA, SB, SC, SD, SE, S_1 and S_2 based on the provisions of this Section. Classification will be done in accordance with Table 6.2.13 based on the soil properties of upper 30 meters of the site profile. Average soil properties will be determined as given in the following equations:

$$\bar{V}_s = \sum_{i=1}^{n} d_i / \sum_{i=1}^{n} \frac{d_i}{V_{si}} \qquad (6.2.31)$$

$$\bar{N} = \sum_{i=1}^{n} d_i / \sum_{i=1}^{n} \frac{d_i}{N_i} \qquad (6.2.32)$$

$$\bar{S}_u = \sum_{i=1}^{k} d_{ci} / \sum_{i=1}^{k} \frac{d_{ci}}{S_{ui}} \qquad (6.2.33)$$

Where,

n = Number of soil layers in upper 30 m

d_i = Thickness of layer i

V_{si} = Shear wave velocity of layer i

N_i = Field (uncorrected) Standard Penetration Value for layer i

k = Number of cohesive soil layers in upper 30 m

d_{ci} = Thickness of cohesive layer i

s_{ui} = Undrained shear strength of cohesive layer i

The site profile up to a depth of 30 m is divided into n number of distinct soil or rock layers. Where some of the layers are cohesive, k is the number of cohesive layers. Hence $\sum_{i=1}^{n} d_i = 30$ m, while $\sum_{i=1}^{k} d_{ci} < 30$ m if $k < n$ in other words if there are both cohesionless and cohesive layers. The standard penetration value N as directly measured in the field without correction will be used.

The site classification should be done using average shear wave velocity $\overline{V}_s$ if this can be estimated, otherwise the value of $\overline{N}$ may be used.

Table 6.2.13: Site Classification Based on Soil Properties

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity, $\overline{V}_s$ (m/s)	SPT Value, $\overline{N}$ (blows/30cm)	Undrained shear strength, $\overline{S}_u$ (kPa)
SA	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	--	--
SB	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity, $\overline{V}_s$ (m/s)	SPT Value, $\overline{N}$ (blows/30cm)	Undrained shear strength, $\overline{S}_u$ (kPa)
SC	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
SD	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
SE	A soil profile consisting of a surface alluvium layer with V_s values of type SC or SD and thickness varying between about 5 m and 20 m, underlain by stiffer material with $V_s > 800$ m/s.	--	--	--
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	--	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types SA to SE or S_1	--	--	--

2.5.4 Earthquake Ground Motion

2.5.4.1 Regional seismicity

Bangladesh can be affected by moderate to strong earthquake events due to its proximity to the collision boundary of the Northeast moving Indian plate and Eurasian Plate. Strong historical earthquakes with magnitude greater than 7.0 have affected parts of Bangladesh in the last 150 years, some of them had their epicenters within the country. A brief description of the local geology, tectonic features and earthquake occurrence in the region is given in Appendix B.

2.5.4.2 Seismic zoning

The intent of the seismic zoning map is to give an indication of the Maximum Considered Earthquake (MCE) motion at different parts of the country. In probabilistic terms, the MCE motion may be considered to correspond to having a 2% probability of exceedance within a period of 50 years. The country has been divided into four seismic zones with different levels of ground motion. Table 6.2.14 includes a description of the four seismic zones. Figure 6.2.24 presents a map of Bangladesh showing the boundaries of the four zones. Each zone has a seismic zone coefficient (Z) which represents the maximum considered peak ground acceleration (PGA) on very stiff soil/rock (site class SA) in units of g (acceleration due to gravity). The zone coefficients (Z) of the four zones are: Z=0.12 (Zone 1), Z=0.20 (Zone 2), Z=0.28 (Zone 3) and Z=0.36 (Zone 4). Table 6.2.15 lists zone coefficients for some important towns of Bangladesh. The most severe earthquake prone zone, Zone 4 is in the northeast which includes Sylhet and has a maximum PGA value of 0.36g. Dhaka city falls in the moderate seismic intensity zone with Z=0.2, while Chittagong city falls in a severe intensity zone with Z=0.28.

2.5.4.3 Design response spectrum

The earthquake ground motion for which the building has to be designed is represented by the design response spectrum. Both static and dynamic analysis methods are based on this response spectrum. This spectrum represents the spectral acceleration for which the building has to be designed as a function of the building period, taking into account the ground motion intensity. The spectrum is based on elastic analysis but in order to account for energy dissipation due to inelastic deformation and benefits of structural redundancy, the spectral accelerations are reduced by the response modification factor *R*. For important structures, the spectral accelerations are increased by the importance factor *I*. The design basis earthquake (DBE) ground motion is selected at a ground shaking level that is 2/3 of the maximum considered earthquake (MCE) ground motion. The effect of local soil conditions on the response spectrum is incorporated in the normalized acceleration response spectrum C_s. The spectral acceleration for the design earthquake is given by the following equation:

$$S_a = \frac{2}{3}\frac{ZI}{R}C_s \qquad (6.2.34)$$

Where,

$S_a =$ Design spectral acceleration (in units of g) which shall not be less than $0.67\beta ZIS$

$\beta =$ Coefficient used to calculate lower bound for S_a. Recommended value for β is 0.11

$Z =$ Seismic zone coefficient, as defined in Sec 2.5.4.2

$I =$ Structure importance factor, as defined in Sec 2.5.5.1

$R =$ Response reduction factor which depends on the type of structural system given in Table 6.2.19. The ratio $\frac{I}{R}$ cannot be greater than one.

$C_s =$ Normalized acceleration response spectrum, which is a function of structure (building) period and soil type (site class) as defined by Equations 6.2.35a to 6.2.35d.

$$C_s = S\left(1 + \frac{T}{T_B}(2.5\eta - 1)\right) \quad \text{for} \quad 0 \le T \le T_B \qquad (6.2.35a)$$

$$C_s = 2.5S\eta \quad \text{for} \quad T_B \le T \le T_C \qquad (6.2.35b)$$

$$C_s = 2.5S\eta\left(\frac{T_C}{T}\right) \quad \text{for} \quad T_C \le T \le T_D \qquad (6.2.35c)$$

$$C_s = 2.5S\eta\left(\frac{T_C T_D}{T^2}\right) \quad \text{for} \quad T_D \le T \le 4\ sec \qquad (6.2.35d)$$

C_s depends on S and values of T_B, T_C and T_D, (Figure 6.2.25) which are all functions of the site class. Constant C_s value between periods T_B and T_C represents constant spectral acceleration.

$S =$ Soil factor which depends on site class and is given in Table 6.2.16

$T =$ Structure (building) period as defined in Sec 2.5.7.2

$T_B =$ Lower limit of the period of the constant spectral acceleration branch given in Table 6.2.16 as a function of site class.

$T_C =$ Upper limit of the period of the constant spectral acceleration branch given in Table 6.2.16 as a function of site class

T_D = Lower limit of the period of the constant spectral displacement branch given in Table 6.2.16 as a function of site class

η = Damping correction factor as a function of damping with a reference value of η=1 for 5% viscous damping. It is given by the following expression:

$$\eta = \sqrt{10/(5+\xi)} \geq 0.55 \qquad (6.2.36)$$

Where, ξ is the viscous damping ratio of the structure, expressed as a percentage of critical damping. The value of η cannot be smaller than 0.55.

The anticipated (design basis earthquake) peak ground acceleration (PGA) for rock or very stiff soil (site class SA) is $\frac{2}{3}Z$. However, for design, the ground motion is modified through the use of response reduction factor R and importance factor I, resulting in $PGA_{rock} = \frac{2}{3}\left(\frac{ZI}{R}\right)$. Figure 6.2.26 shows the normalized acceleration response spectrum C_s for 5% damping, which may be defined as the 5% damped spectral acceleration (obtained by Eq. 6.2.34) normalized with respect to PGA_{rock} . This Figure demonstrates the significant influence of site class on the response spectrum.

Design Spectrum for Elastic Analysis

For site classes SA to SE, the design acceleration response spectrum for elastic analysis methods is obtained using Eq. 6.2.34 to compute S_a (in units of g) as a function of period T. The design acceleration response spectrum represents the expected ground motion (Design Basis Earthquake) divided by the factor R/I.

Design Spectrum for Inelastic Analysis

For inelastic analysis methods, the anticipated ground motion (Design Basis Earthquake) is directly used. Corresponding real design acceleration response spectrum is used, which is obtained by using R=1 and I=1 in Eq. 6.2.34. The 'real design acceleration response spectrum' is equal to 'design acceleration response spectrum' multiplied by R/I.

Site-Specific Design Spectrum

For site class S_1 and S_2, site-specific studies are needed to obtain design response spectrum. For important projects, site-specific studies may also be carried out to determine spectrum instead of using Eq. 6.2.34. The objective of such site-specific ground-motion analysis is to determine ground motions for local seismic and site conditions with higher confidence than is possible using simplified equations.

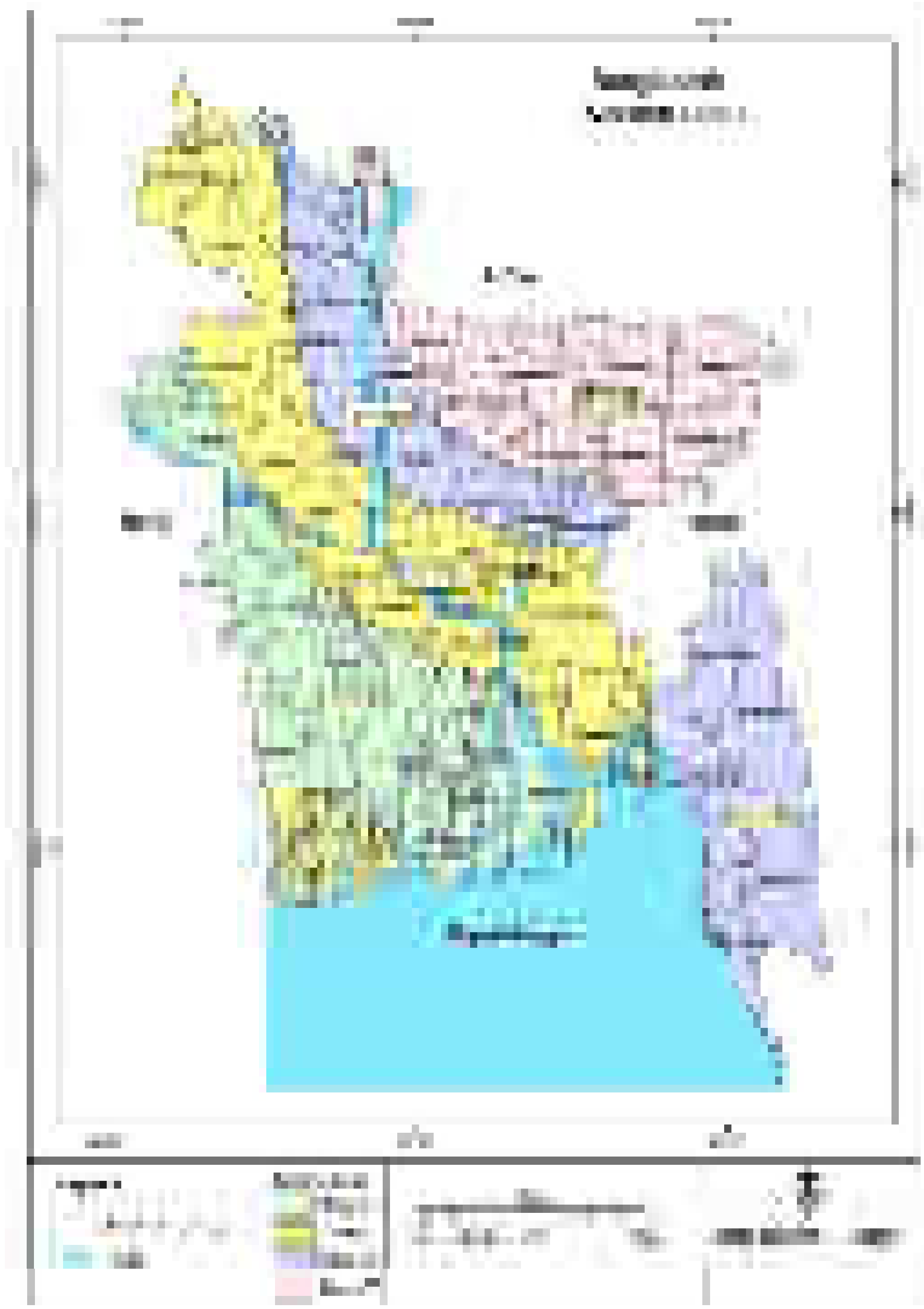


Figure 6.2.24 Seismic zoning map of Bangladesh

Table 6.2.14: Description of Seismic Zones

Seismic Zone	Location	Seismic Intensity	Seismic Zone Coefficient, Z
1	Southwestern part including Barisal, Khulna, Jessore, Rajshahi	Low	0.12
2	Lower Central and Northwestern part including Noakhali, Dhaka, Pabna, Dinajpur, as well as Southwestern corner including Sundarbans	Moderate	0.20
3	Upper Central and Northwestern part including Brahmanbaria, Sirajganj, Rangpur	Severe	0.28
4	Northeastern part including Sylhet, Mymensingh, Kurigram	Very Severe	0.36

Table 6.2.15: Seismic Zone Coefficient Z for Some Important Towns of Bangladesh

Town	Z	Town	Z	Town	Z	Town	Z
Bagerhat	0.12	Gaibandha	0.28	Magura	0.12	Patuakhali	0.12
Bandarban	0.28	Gazipur	0.20	Manikganj	0.20	Pirojpur	0.12
Barguna	0.12	Gopalganj	0.12	Maulvibazar	0.36	Rajbari	0.20
Barisal	0.12	Habiganj	0.36	Meherpur	0.12	Rajshahi	0.12
Bhola	0.12	Jaipurhat	0.20	Mongla	0.12	Rangamati	0.28
Bogra	0.28	Jamalpur	0.36	Munshiganj	0.20	Rangpur	0.28
Brahmanbaria	0.28	Jessore	0.12	Mymensingh	0.36	Satkhira	0.12
Chandpur	0.20	Jhalokati	0.12	Narail	0.12	Shariatpur	0.20
Chapainababganj	0.12	Jhenaidah	0.12	Narayanganj	0.20	Sherpur	0.36
Chittagong	0.28	Khagrachari	0.28	Narsingdi	0.28	Sirajganj	0.28
Chuadanga	0.12	Khulna	0.12	Natore	0.20	Srimangal	0.36
Comilla	0.20	Kishoreganj	0.36	Naogaon	0.20	Sunamganj	0.36
Cox's Bazar	0.28	Kurigram	0.36	Netrakona	0.36	Sylhet	0.36
Dhaka	0.20	Kushtia	0.20	Nilphamari	0.12	Tangail	0.28
Dinajpur	0.20	Lakshmipur	0.20	Noakhali	0.20	Thakurgaon	0.20
Faridpur	0.20	Lalmanirhat	0.28	Pabna	0.20		
Feni	0.20	Madaripur	0.20	Panchagarh	0.20		

Figure 6.2.25 Typical shape of the elastic response spectrum coefficient C_s

Table 6.2.16: Site Dependent Soil Factor and Other Parameters Defining Elastic Response Spectrum

Soil type	S	T_B(s)	T_C (s)	T_D (s)
SA	1.0	0.15	0.40	2.0
SB	1.2	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.4	0.15	0.50	2.0

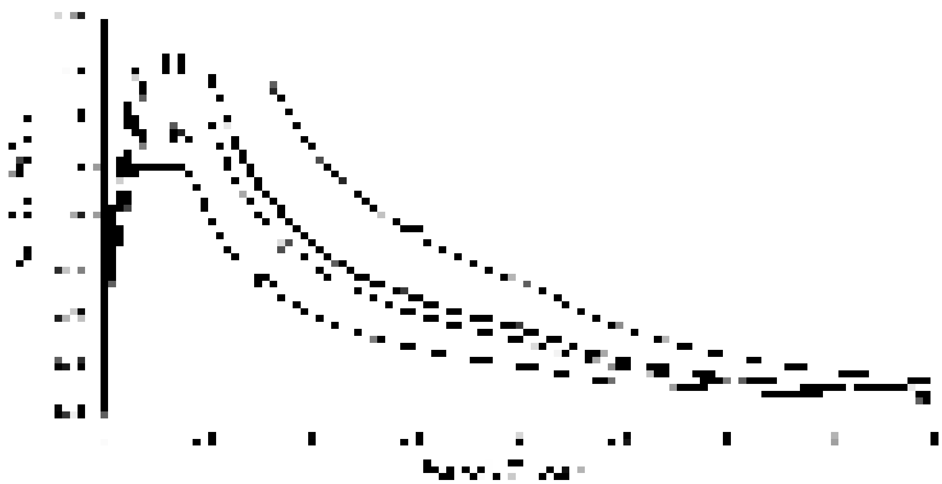


Figure 6.2.26 Normalized design acceleration response spectrum for different site classes.

2.5.5 Building Categories

2.5.5.1 Importance factor

Buildings are classified in four occupancy categories in Chapter 1 (Table 6.1.1), depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse. Depending on occupancy category, buildings may be designed for higher seismic forces using importance factor greater than one. Table 6.2.17 defines different occupancy categories and corresponding importance factor.

Table 6.2.17: Importance Factors for Buildings and Structures for Earthquake design

Occupancy Category	Importance factor I
I, II	1.00
III	1.25
IV	1.50

2.5.5.2 Seismic design category

Buildings shall be assigned a seismic design category among B, C or D based on seismic zone, local site conditions and importance class of building, as given in Table 6.2.18. Seismic design category D has the most stringent seismic design detailing, while seismic design category B has the least seismic design detailing requirements.

Table 6.2.18: Seismic Design Category of Buildings

Site Class	Occupancy Category I, II and III				Occupancy Category IV			
	Zone 1	Zone 2	Zone 3	Zone 4	Zone 1	Zone 2	Zone 3	Zone 4
SA	B	C	C	D	C	D	D	D
SB	B	C	D	D	C	D	D	D
SC	B	C	D	D	C	D	D	D
SD	C	D	D	D	D	D	D	D
SE, S_1, S_2	D	D	D	D	D	D	D	D

2.5.5.3 Building irregularity

Buildings with irregularity in plan or elevation suffer much more damage in earthquakes than buildings with regular configuration. A building may be considered as irregular, if at least one of the conditions given below are applicable:

2.5.5.3.1 Plan irregularity: Following are the different types of irregularities that may exist in the plan of a building.

(i) Torsion irregularity

To be considered for rigid floor diaphragms, when the maximum storey drift (Δ_{max}) as shown in Figure 6.2.27(a), computed including accidental torsion, at one end of the structure is more than 1.2 times the average $\left(\Delta_{avg}=\frac{\Delta_{max}+\Delta_{min}}{2}\right)$ of the storey drifts at the two ends of the structure. If $\Delta_{max}> 1.4\Delta_{avg}$ then the irregularity is termed as extreme torsional irregularity.

(ii) Re-entrant corners

Both projections of the structure beyond a re-entrant comer [Figure 6.2.27(b)] 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 [Figure 6.2.27(c)] 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, as shown in Figure 6.2.27(d).

(v) Non-parallel Systems

The vertical elements resisting the lateral force are not parallel to or symmetric [Figure 6.2.27(e)] about the major orthogonal axes of the lateral force resisting elements.

2.5.5.3.2 Vertical Irregularity: Following are different types of irregularities that may exist along vertical elevations of a building.

(i) Stiffness Irregularity - Soft Storey

A soft storey is one in which the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average lateral stiffness of the three storeys above irregularity [Figure 6.2.28(a)]. An extreme soft storey is defined where its lateral stiffness is less than 60% of that in the storey above or less than 70% of the average lateral stiffness of the three storeys above.

(ii) Mass Irregularity

The seismic weight of any storey is more than twice of that of its adjacent storeys [Figure 6.2.28(b)]. This irregularity need not be considered in case of roofs.

(iii) Vertical Geometric Irregularity

This irregularity exists for buildings with setbacks with dimensions given in Figure [6.2.28(c)].

(iv) Vertical 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 [Figure 6.2.28(d)].

(v) Discontinuity in Capacity - Weak Storey

A weak storey is one in which the storey lateral strength is less than 80% 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 [Figure 6.2.28(e)]. An extreme weak storey is one where the storey lateral strength is less than 65% of that in the storey above.

2.5.5.4 Type of structural systems

The basic lateral and vertical seismic force–resisting system shall conform to one of the types A to G indicated in Table 6.2.19. Each type is again subdivided by the types of vertical elements used to resist lateral seismic forces. A combination of systems may also be permitted as stated in Sec 2.5.5.5.

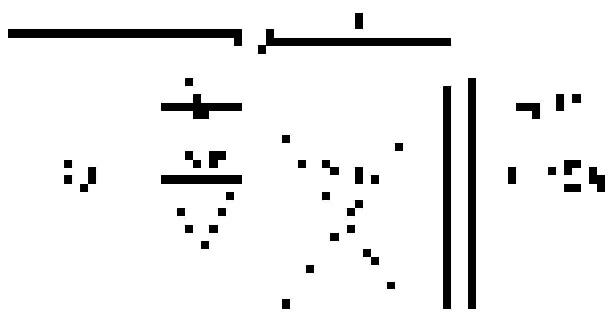
The structural system to be used shall be in accordance with the seismic design category indicated in Table 6.2.18. Structural systems that are not permitted for a certain seismic design category are indicated by “NP”. Structural systems that do not have any height restriction are indicated by “NL”. Where there is height limit, the maximum height in meters is given.

The response reduction factor, R, and the deflection amplification factor, C_d indicated in Table 6.2.19 shall be used in determining the design base shear and design story drift. The selected seismic force-resisting system shall be designed and detailed in accordance with the specific requirements for the system.

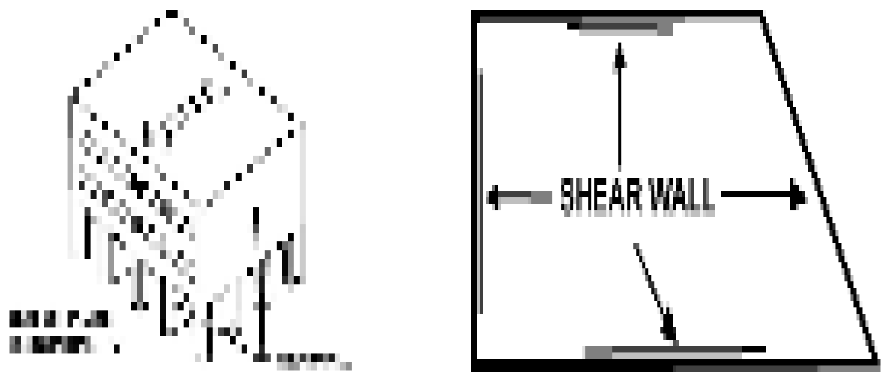
Seismic force resisting systems that are not given in Table 6.2.19 may be permitted if substantial analytical and test data are submitted that establish the dynamic characteristics and demonstrate the lateral force resistance and energy dissipation capacity to be equivalent to the structural systems listed in Table 6.2.19 for equivalent response modification coefficient, R, and deflection amplification factor, C_d values.

(a) Torsional Irregularity

(b) Re-entrant corners (A/L>0.15)

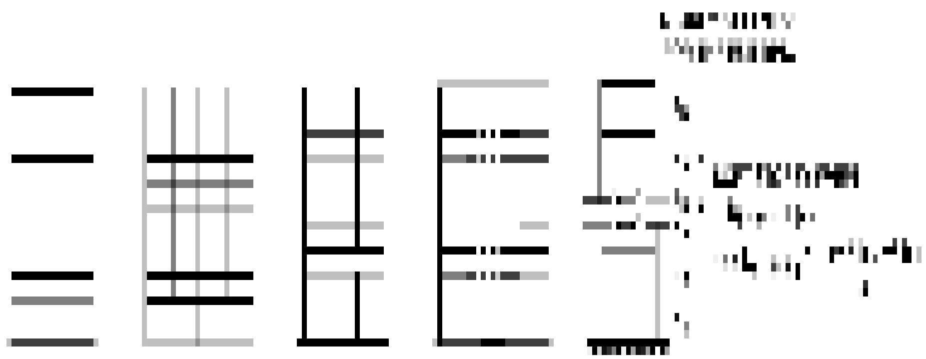


(c) Diaphragm discontinuity

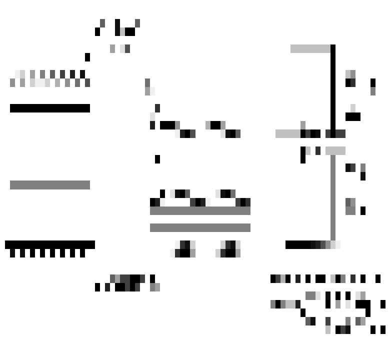


(d) Out- of-plane offsets of shear wall (e) Non-parallel systems of shear wall

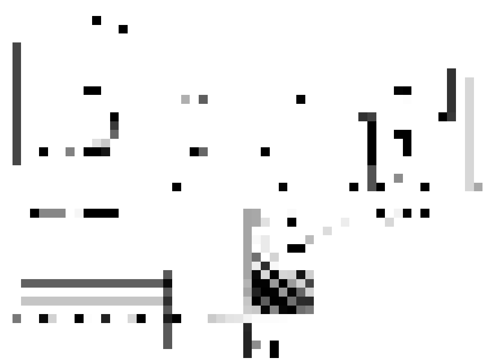
Figure 6.2.27 Different types of plan irregularities of buildings



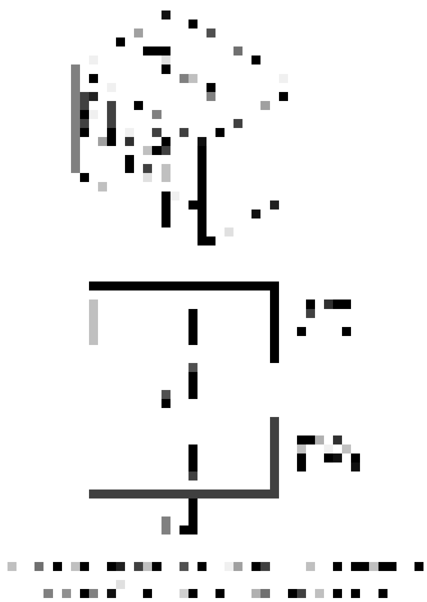
(a) Soft storey



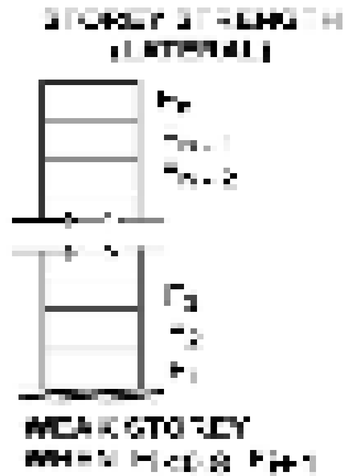
(b) Mass irregularity



(c) Vertical geometric irregularity (setback structures)



(d) Vertical In-Plane Discontinuity in Vertical Elements Resisting Lateral Force



(e) Weak storey

Figure 6.2.28 Different types of vertical irregularities of buildings

Table 6.2.19: Response Reduction Factor, Deflection Amplification Factor and Height Limitations for Different Structural Systems

Seismic Force–Resisting System	Response Reduction Factor, R	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Seismic Design Category B	Seismic Design Category C	Seismic Design Category D
				Height limit (m)		
A. BEARING WALL SYSTEMS (no frame)						
1. Special reinforced concrete shear walls	5	2.5	5	NL	NL	50
2. Ordinary reinforced concrete shear walls	4	2.5	4	NL	NL	NP
3. Ordinary reinforced masonry shear walls	2	2.5	1.75	NL	50	NP
4. Ordinary plain masonry shear walls	1.5	2.5	1.25	18	NP	NP

Seismic Force–Resisting System	Response Reduction Factor, R	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Seismic Design Category B	Seismic Design Category C	Seismic Design Category D
				Height limit (m)		
B. BUILDING FRAME SYSTEMS (with bracing or shear wall)						
1. Steel eccentrically braced frames, moment resisting connections at columns away from links	8	2	4	NL	NL	50
2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	7	2	4	NL	NL	50
3. Special steel concentrically braced frames	6	2	5	NL	NL	50
4. Ordinary steel concentrically braced frames	3.25	2	3.25	NL	NL	11
5. Special reinforced concrete shear walls	6	2.5	5	NL	NL	50
6. Ordinary reinforced concrete shear walls	5	2.5	4.25	NL	NL	NP
7. Ordinary reinforced masonry shear walls	2	2.5	2	NL	50	NP
8. Ordinary plain masonry shear walls	1.5	2.5	1.25	18	NP	NP
C. MOMENT RESISTING FRAME SYSTEMS (no shear wall)						
1. Special steel moment frames	8	3	5.5	NL	NL	NL
2. Intermediate steel moment frames	4.5	3	4	NL	NL	35
3. Ordinary steel moment frames	3.5	3	3	NL	NL	NP

Seismic Force–Resisting System	Response Reduction Factor, R	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Seismic Design Category B	Seismic Design Category C	Seismic Design Category D
				Height limit (m)		
4. Special reinforced concrete moment frames	8	3	5.5	NL	NL	NL
5. Intermediate reinforced concrete moment frames	5	3	4.5	NL	NL	NP
5. Ordinary reinforced concrete moment frames	3	3	2.5	NL	NP	NP
D. DUAL SYSTEMS: SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)						
1. Steel eccentrically braced frames	8	2.5	4	NL	NL	NL
2. Special steel concentrically braced frames	7	2.5	5.5	NL	NL	NL
3. Special reinforced concrete shear walls	7	2.5	5.5	NL	NL	NL
4. Ordinary reinforced concrete shear walls	6	2.5	5	NL	NL	NP
E. DUAL SYSTEMS: INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)						
1. Special steel concentrically braced frames	6	2.5	5	NL	NL	11
2. Special reinforced concrete shear walls	6.5	2.5	5	NL	NL	50
3. Ordinary reinforced masonry shear walls	3	3	3	NL	50	NP

Seismic Force–Resisting System	Response Reduction Factor, R	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Seismic Design Category B	Seismic Design Category C	Seismic Design Category D
				Height limit (m)		
4. Ordinary reinforced concrete shear walls	5.5	2.5	4.5	NL	NL	NP
F. DUAL SHEAR WALL-FRAME SYSTEM: ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	4.5	2.5	4	NL	NP	NP
G. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE	3	3	3	NL	NL	NP

Notes:

1. Seismic design category, NL = No height restriction, NP = Not permitted. Number represents maximum allowable height (m).
2. Dual Systems include buildings which consist of both moment resisting frame and shear walls (or braced frame) where both systems resist the total design forces in proportion to their lateral stiffness.
3. See Sec. 10.20 of Chapter 10 of this Part for additional values of R and C_d and height limits for some other types of steel structures not covered in this Table.
4. Where data specific to a structure type is not available in this Table, reference may be made to Table 12.2-1 of ASCE 7-05.

2.5.5.5 Combination of structural systems

2.5.5.5.1 Combinations of Structural Systems in Different Directions: Different seismic force–resisting systems are permitted to be used to resist seismic forces along each of the two orthogonal axes of the structure. Where different systems are used, the respective R and C_d coefficients shall apply to each system, including the limitations on system use contained in Table 6.2.19.

2.5.5.5.2 Combinations of Structural Systems in the Same Direction: Where different seismic force–resisting systems are used in combination to resist seismic forces in the same direction of structural response, other than those combinations considered as dual systems, the more stringent system limitation contained in Table 6.2.19 shall apply. The value of R used for design in that direction shall not be greater than the least value of R for any of the systems utilized in that direction. The deflection amplification factor, C_d in the direction under consideration at any story shall not be less than the largest value of this factor for the R factor used in the same direction being considered.

2.5.5.6 Provisions for Using System Overstrength Factor, Ω_o

2.5.5.6.1 Combinations of Elements Supporting Discontinuous Walls or Frames.

Columns, beams, trusses, or slabs supporting discontinuous walls or frames of structures having horizontal irregularity Type IV of Table 6.1.5 or vertical irregularity Type IV of Table 6.1.4 shall have the design strength to resist the maximum axial force that can develop in accordance with the load combinations with overstrength factor of Section 2.5.13.4. The connections of such discontinuous elements to the supporting members shall be adequate to transmit the forces for which the discontinuous elements were required to be designed.

2.5.5.6.2 Increase in Forces Due to Irregularities for Seismic Design Category D.

For structures assigned to Seismic Design Category D and having a horizontal structural irregularity of Type I.a, I.b, II, III, or IV in Table 6.1.5 or a vertical structural irregularity of Type IV in Table 6.1.4, the design forces determined from Section 2.5.7 shall be increased 25 percent for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements. Collectors and their connections also shall be designed for these increased forces unless they are designed for the load combinations with overstrength factor of Section 2.5.5.4, in accordance with Section 2.5.13.4.

2.5.5.6.3 Collector Elements Requiring Load Combinations with Overstrength Factor for Seismic Design Categories C through D.

In structures assigned to Seismic Design Category C or D, collector elements, splices, and their connections to resisting elements shall resist the load combinations with overstrength of Section 2.5.13.4.

2.5.5.6.4 Batter Piles.

Batter piles and their connections shall be capable of resisting forces and moments from the load combinations with overstrength factor of Section 2.5.13.4. Where vertical and batter piles act jointly to resist foundation forces as a group, these forces shall be distributed to the individual piles in accordance with their relative horizontal and vertical rigidities and the geometric distribution of the piles within the group.

2.5.6 Static Analysis Procedure

Although analysis of buildings subjected to dynamic earthquake loads should theoretically require dynamic analysis procedures, for certain type of building structures subjected to earthquake shaking, simplified static analysis procedures may also provide reasonably good results. The equivalent static force method is such a procedure for determining the seismic lateral forces acting on the structure. This type of analysis may be applied to buildings whose seismic response is not significantly affected by contributions from modes higher than the fundamental mode in each direction. This requirement is deemed to be satisfied in buildings which fulfill the following two conditions:

(a) The building period in the two main horizontal directions is smaller than both $4T_C$ (T_C is defined in Sec 2.5.4.3) and 2 seconds.

(b) The building does not possess irregularity in elevation as defined in Sec 2.5.5.3.

2.5.7 Equivalent Static Analysis

The evaluation of the seismic loads starts with the calculation of the design base shear which is derived from the design response spectrum presented in Sec 2.5.4.3. This Section presents different computations relevant to the equivalent static analysis procedure.

2.5.7.1 Design base shear

The seismic design base shear force in a given direction shall be determined from the following relation:

$$V = S_a W \qquad (6.2.37)$$

Where,

$S_a =$ Lateral seismic force coefficient calculated using Eq. 6.2.34 (Sec 2.5.4.3). It is the design spectral acceleration (in units of g) corresponding to the building period T (computed as per Sec 2.5.7.2).

$W =$ Total seismic weight of the building defined in Sec 2.5.7.3

Alternatively, for buildings with natural period less than or equal to 2.0 sec., the seismic design base shear can be calculated using ASCE 7-02 with seismic design parameters as given in Appendix C. However, the minimum value of S_a should not be less than 0.044 $S_{DS}I$. The values of S_{DS} are provided in Table 6.C.4 of Appendix C.

2.5.7.2 Building period

The fundamental period T of the building in the horizontal direction under consideration shall be determined using the following guidelines:

(a) Structural dynamics procedures (such as Rayleigh method or modal eigenvalue analysis), using structural properties and deformation characteristics of resisting elements, may be used to determine the fundamental period T of the building in the direction under consideration. This period shall not exceed the approximate fundamental period determined by Eq. 6.2.38 by more than 40 percent.

(b) The building period T (in sec) may be approximated by the following formula:

$$T = C_t(h_n)^m \tag{6.2.38}$$

Where,

$h_n =$ Height of building in metres from foundation or from top of rigid basement. 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. C_t and m are obtained from Table 6.2.20

(c) For masonry or concrete shear wall structures, the approximate fundamental period, T (in sec) may be determined as follows:

$$T = \frac{0.0062}{\sqrt{C_w}} h_n \tag{6.2.39}$$

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i} \right)^2 \frac{A_i}{\left[1 + 0.83 \left(\frac{h_i}{D_i} \right)^2 \right]} \tag{6.2.40}$$

Where,

A_B = area of base of structure

A_i = web area of shear wall "i"

D_i = length of shear wall "i"

h_i = height of shear wall "i"

x = number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

Table 6.2.20: Values for Coefficients to Estimate Approximate Period

Structure type	C_t	m	
Concrete moment-resisting frames	0.0466	0.9	Note: Consider moment resisting frames as frames which resist 100% of seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting under seismic forces.
Steel moment-resisting frames	0.0724	0.8	
Eccentrically braced steel frame	0.0731	0.75	
All other structural systems	0.0488	0.75	

2.5.7.3 Seismic weight

Seismic weight, W, is the total dead load of a building or a structure, including partition walls, and applicable portions of other imposed loads listed below:

(a) For live load up to and including 3 kN/m^2, a minimum of 25 percent of the live load shall be applicable.

(b) For live load above 3 kN/m^2, a minimum of 50 percent of the live load shall be applicable.

(c) Total weight (100 percent) of permanent heavy equipment or retained liquid or any imposed load sustained in nature shall be included.

Where the probable imposed loads (mass) at the time of earthquake are more correctly assessed, the designer may go for higher percentage of live load.

2.5.7.4 Vertical distribution of lateral forces

In the absence of a more rigorous procedure, the total seismic lateral force at the base level, in other words the base shear V, shall be considered as the sum of lateral forces F_x induced at different floor levels, these forces may be calculated as:

$$F_x = V \frac{w_x h_x^{\ k}}{\sum_{i=1}^{n} w_i h_i^{\ k}} \qquad (6.2.41)$$

Where,

F_x = Part of base shear force induced at level x

w_i and w_x = Part of the total effective seismic weight of the structure (W) assigned to level i or x

h_i and h_x = the height from the base to level i or x

k = 1 For structure period ≤ 0.5s

= 2 for structure period ≥ 2.5s

= linear interpolation between 1 and 2 for other periods.

n = number of stories

2.5.7.5 Storey shear and its horizontal distribution

The design storey shear V_x, at any storey x is the sum of the forces F_x in that storey and all other stories above it, given by Eq. 6.2.42:

$$V_x = \sum_{i=x}^{n} F_i \qquad (6.2.42)$$

Where, F_i = Portion of base shear induced at level i, as determined by Eq. 6.2.41.

If the floor diaphragms can be considered to be infinitely rigid in the horizontal plane, the shear V_x shall be distributed to the various elements of the lateral force resisting system in proportion to their relative lateral stiffness. For flexible diaphragms, the distribution of forces to the vertical elements shall account for the position and distribution of the masses supported.

Allowance shall also be made for the increased shear arising due to horizontal torsional moment as specified in Sec 2.5.7.6

2.5.7.6 Horizontal torsional moments

Design shall accommodate increase in storey shear forces resulting from probable horizontal torsional moments on rigid floor diaphragms. Computation of such moments shall be as follows:

2.5.7.6.1 In-built torsional effects: When there is in-built eccentricity between centre of mass and centre of rigidity (lateral resistance) at floor levels, rigid diaphragms at each level will be subject to torsional moment M_t.

2.5.7.6.2 Accidental torsional effects: In order to account for uncertainties in the location of masses and in the spatial variation of the seismic motion, accidental torsional effects need to be always considered. The accidental moment M_{ta} is determined assuming the storey mass to be displaced from the calculated centre of mass a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration. The accidental torsional moment M_{tai} at level i is given as:

$$M_{tai} = e_{ai} F_i \qquad (6.2.43)$$

Where,

e_{ai} = accidental eccentricity of floor mass at level i applied in the same direction at all floors = $\pm 0.05 L_i$

L_i = floor dimension perpendicular to the direction of seismic force considered.

Where torsional irregularity exists (Sec 2.5.5.3.1) for Seismic Design Category C or D, the irregularity effects shall be accounted for by increasing the accidental torsion M_{ta} at each level by a torsional amplification factor, A_x as illustrated in Figure 6.2.29 determined from the following equation:

$$A_x = \left[\frac{\delta_{max}}{1.2\delta_{avg}}\right]^2 \leq 3.0 \qquad (6.2.44)$$

Where,

δ_{max} = Maximum displacement at level-x computed assuming $A_x = 1$.

δ_{avg} = Average displacements at extreme points of the building at level-x computed assuming $A_x = 1$.

The accidental torsional moment need not be amplified for structures of light-frame construction. Also the torsional amplification factor (A_x) should not exceed 3.0.

2.5.7.6.3 Design for torsional effects: The torsional design moment at a given storey shall be equal to the accidental torsional moment M_{ta} plus the inbuilt torsional moment M_t (if any). Where earthquake forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass (for accidental torsion) need not be applied in both of the orthogonal directions at the same time, but shall be applied in only one direction that produces the greater effect.

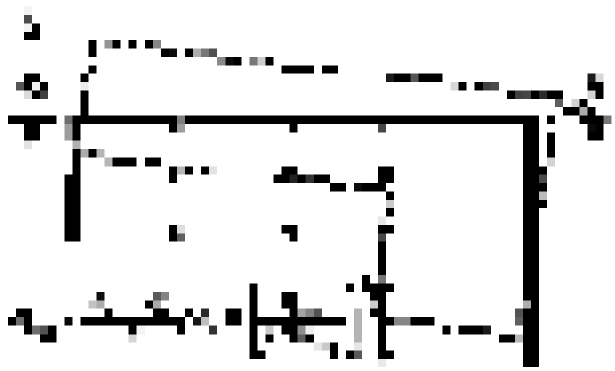


Figure 6.2.29 Torsional amplification factor A_x for plan irregularity.

2.5.7.7 Deflection and storey drift

The deflections (δ_x) of level x at the center of the mass shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad (6.2.45)$$

Where,

$C_d =$ Deflection amplification factor given in Table 6.2.19

$\delta_{xe} =$ Deflection determined by an elastic analysis

$I =$ Importance factor defined in Table 6.2.17

The design storey drift at storey x shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration:

$$\Delta_x = \delta_x - \delta_{x-1} \qquad (6.2.46)$$

2.5.7.8 Overturning effects

The structure shall be designed to resist overturning effects caused by the seismic forces determined in Sec 2.5.7.4. At any story, the increment of overturning moment in the story under consideration shall be distributed to the various vertical force resisting elements in the same proportion as the distribution of the horizontal shears to those elements. The overturning moments at level x, M_x shall be determined as follows:

$$M_x = \sum_{i=x}^{n} F_i\left(h_i - h_x\right) \tag{6.2.47}$$

Where,

F_i = Portion of the seismic base shear, V induced at level i

h_i, h_x = Height from the base to level i or x.

The foundations of structures, except inverted pendulum-type structures, shall be permitted to be designed for three-fourths of the foundation overturning design moment, M_o determined using above equation.

2.5.7.9 P-delta effects

The P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered if the stability coefficient (θ) determined by the following equation is not more than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \tag{6.2.48}$$

Where,

P_x = Total vertical design load at and above level x; where computing P_x, no individual load factor need exceed 1.0

Δ = Design story drift occurring simultaneously with V_x

V_x = Storey shear force acting between levels x and $x-1$

h_{sx} = Storey height below level x

C_d = Deflection amplification factor given in Table 6.2.19

The stability coefficient θ shall not exceed θ_{max} determined as follows:

$$\theta_{\max} = \frac{0.5}{\beta C_d} \leq 0.25 \tag{6.2.49}$$

Where, β is the ratio of shear demand to shear capacity for the story between levels x and $x - 1$. This ratio is permitted to be conservatively taken as 1.0.

Where, the stability coefficient θ is greater than 0.10 but less than or equal to θ_{max}, the incremental factor related to P-delta effects on displacements and member forces shall be determined by rational analysis. Alternatively, it is permitted to multiply displacements and member forces by $\frac{1}{(1-\theta)}$.

Where, θ is greater than θ_{max}, the structure is potentially unstable and shall be redesigned.

Where, the P-delta effect is included in an automated analysis, Eq. 6.2.49 shall still be satisfied, however, the value of θ computed from Eq. 6.2.48 using the results of the P-delta analysis is permitted to be divided by $(1 + \theta)$ before checking Eq. 6.2.49.

2.5.8 Dynamic Analysis Methods

Dynamic analysis method involves applying principles of structural dynamics to compute the response of the structure to applied dynamic (earthquake) loads.

2.5.8.1 Requirement for dynamic analysis

Dynamic analysis should 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 with height greater than 40 m in Zones 2, 3, 4 and greater than 90 m in Zone 1.

(b) Irregular buildings (as defined in Sec 2.5.5.3) with height greater than 12 m in Zones 2, 3, 4 and greater than 40 m in Zone 1.

For irregular buildings, smaller than 40 m in height in Zone 1, dynamic analysis, even though not mandatory, is recommended.

2.5.8.2 Methods of analysis

Dynamic analysis may be carried out through the following two methods:

(i) Response Spectrum Analysis method is a linear elastic analysis method using modal analysis procedures, where the structure is subjected to spectral accelerations corresponding to a design acceleration response spectrum. The design earthquake ground motion in this case is represented by its response spectrum.

(ii) Time History Analysis method is a numerical integration procedure where design ground motion time histories (acceleration record) are applied at the base of the structure. Time history analysis procedures can be two types: linear and non-linear.

2.5.9 Response Spectrum Analysis (RSA)

A response spectrum analysis shall consist of the analysis of a linear mathematical model of the structure to determine the maximum accelerations, forces, and displacements resulting from the dynamic response to ground shaking represented by the design acceleration response spectrum (presented in Sec 2.5.4.3). Response spectrum analysis is also called a modal analysis procedure because it considers different modes of vibration of the structure and combines effects of different modes.

2.5.9.1 Modeling (RSA)

A mathematical model of the structure shall be constructed that represents the spatial distribution of mass and stiffness throughout the structure. For regular structures with independent orthogonal seismic-force-resisting systems, independent two-dimensional models are permitted to be constructed to represent each system. For irregular structures or structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the structure. Where the diaphragms are not rigid compared to the vertical elements of the lateral-force-resisting system, the model should include representation of the diaphragm’s flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure’s dynamic response. The structure shall be considered to be fixed at the base or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations. In addition, the model shall comply with the following:

(a) Stiffness properties of concrete and masonry elements shall consider the effects of cracked sections

(b) The contribution of panel zone deformations to overall story drift shall be included for steel moment frame resisting systems.

2.5.9.2 Number of modes (RSA)

An analysis shall be conducted using the masses and elastic stiffnesses of the seismic-force-resisting system to determine the natural modes of vibration for the structure including the period of each mode, the modal shape vector ϕ, the modal participation factor P and modal mass M. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of two orthogonal directions.

2.5.9.3 Modal story shears and moments (RSA)

For each mode, the story shears, story overturning moments, and the shear forces and overturning moments in vertical elements of the structural system at each level due to the seismic forces shall be computed. The peak lateral force F_{ik} induced at level i in mode k is given by:

$$F_{ik} = A_k \phi_{ik} P_k W_i \qquad (6.2.50)$$

Where,

A_k = Design horizontal spectral acceleration corresponding to period of vibration T_k of mode k obtained from design response spectrum (Sec 2.5.4.3)

ϕ_{ik} = Modal shape coefficient at level i in mode k

P_k = Modal participation factor of mode k

W_i = Weight of floor i.

2.5.9.4 Structure response (RSA)

In the response spectrum analysis method, the base shear V_{rs}; each of the story shear, moment, and drift quantities; and the deflection at each level shall be determined by combining their modal values. The combination shall be carried out by taking the square root of the sum of the squares (SRSS) of each of the modal values or by the complete quadratic combination (CQC) technique. The complete quadratic combination shall be used where closely spaced periods in the translational and torsional modes result in cross-correlation of the modes.

The distribution of horizontal shear shall be in accordance with the requirements of Sec 2.5.7.5. It should be noted that amplification of accidental torsion as per Sec 2.5.7.6 is not required where accidental torsional effects are included in the dynamic analysis model by offsetting the centre of mass in each story by the required amount.

A base shear, V shall also be calculated using the equivalent static force procedure in Sec 2.5.7. Where the base shear, V_{rs} is less than 85 percent of V, all the forces but not the drifts obtained by response spectrum analysis shall be multiplied by the ratio $\frac{0.85V}{V_{rs}}$.

The displacements and drifts obtained by response spectrum analysis shall be multiplied by C_d/I to obtain design displacements and drifts, as done in equivalent static analysis procedure (Sec 2.5.7.7). The P-delta effects shall be determined in accordance with Sec 2.5.7.9.

2.5.10 Linear Time History Analysis (LTHA)

A linear time history analysis (LTHA) shall consist of an analysis of a linear mathematical model of the structure to determine its response, through direct numerical integration of the differential equations of motion, to a number of ground motion acceleration time histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with the provisions of this Section. For the purposes of analysis, the structure shall be permitted to be considered to be fixed at the base or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations. The acceleration time history (ground motion) is applied at the base of the structure. The advantage of this procedure is that the time dependent behavior of the structural response is obtained.

2.5.10.1 Modeling (LTHA)

Mathematical models shall conform to the requirements of modeling described in Sec 2.5.9.1.

2.5.10.2 Ground motion (LTHA)

At least three appropriate ground motions (acceleration time history) shall be used in the analysis. Ground motion shall conform to the requirements of this Section.

Two-dimensional analysis: Where two-dimensional analyses are performed, each ground motion shall consist of a horizontal acceleration time history selected from an actual recorded event. Appropriate acceleration histories shall be obtained from records of events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of appropriate ground motion records are not available, appropriate simulated ground motion time histories shall be used to make up the total number required. The ground motions shall be scaled such that for each period between 0.2T and 1.5T (where T is the natural period of the structure in the fundamental mode for the direction considered) the average of the five-percent-damped response spectra for the each acceleration time history is not less than the corresponding ordinate of the design acceleration response spectrum, determined in accordance with Sec 2.5.4.3.

Three-dimensional analysis: Where three-dimensional analysis is performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration time histories (in two orthogonal horizontal directions) that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of recorded ground motion pairs are not available, appropriate simulated ground motion pairs shall be used to make up the total number required. For each pair of horizontal ground motion components, an SRSS spectrum shall be constructed by taking the square root of the sum of the squares of the five-percent-damped response spectra for the components (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that for each period between 0.2T and 1.5T (where T is the natural period of the fundamental mode of the structure) the average of the SRSS spectra from all horizontal component pairs is not less than 1.3 times the corresponding ordinate of the design response spectrum, determined in accordance with Sec 2.5.4.3.

2.5.10.3 Structure response (LTHA)

For each scaled acceleration time history, the maximum values of base shear and other structure response quantities shall be obtained from the time history analysis. For three dimensional analysis, orthogonal pair of scaled motions are applied simultaneously. A base shear, V, shall also be calculated using the equivalent static force procedure described in Sec 2.5.7.1. Where the maximum base shear, V_{th} computed by linear time history analysis, is less than V, all response quantities (storey shear, moments, drifts, floor deflections, member forces etc) obtained by time history analysis shall be increased by multiplying with the ratio, $\frac{V}{V_{th}}$. If number of earthquake records (or pairs) used in the analysis is less than seven, the maximum structural response obtained corresponding to different earthquake records shall be considered as the design value. If the number is at least seven, then the average of maximum structural responses for different earthquake records shall be considered as the design value.

The displacements and drifts obtained as mentioned above shall be multiplied by $\frac{C_d}{I}$ to obtain design displacements and drifts, as done in equivalent static analysis procedure (Sec 2.5.7.7).

2.5.11 Non-Linear Time History Analysis (NTHA)

Nonlinear time history analysis (NTHA) shall consist of analysis of a mathematical model of the structure which incorporates the nonlinear hysteretic behavior of the structure's components to determine its response, through methods of numerical integration, to ground acceleration time histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with the requirements of this Section. For the purposes of analysis, the structure shall be permitted to be considered to be fixed at the base or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations. The acceleration time history (ground motion) is applied at the base of the structure. The advantage of this procedure is that actual time dependent behavior of the structural response considering inelastic deformations in the structure can be obtained.

2.5.11.1 Modeling (NTHA)

A mathematical model of the structure shall be constructed that represents the spatial distribution of mass throughout the structure. The hysteretic behavior of elements shall be modeled consistent with suitable laboratory test data and shall account for all significant yielding, strength degradation, stiffness degradation, and hysteretic pinching indicated by such test data. Strength of elements shall be based on expected values considering material over-strength, strain hardening, and hysteretic strength degradation. As a minimum, a bilinear force deformation relationship should be used at the element level. In reinforced concrete and masonry buildings, the elastic stiffness should correspond to that of cracked sections. Linear properties, consistent with the provisions of Chapter 5 shall be permitted to be used for those elements demonstrated by the analysis to remain within their linear range of response. The structure shall be assumed to have a fixed base or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness and load carrying characteristics of the foundations consistent with site-specific soils data and rational principles of engineering mechanics.

For regular structures with independent orthogonal seismic-force-resisting systems, independent two dimensional models shall be permitted to be constructed to represent each system. For structures having plan irregularity or structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis at each level of the structure shall be used. Where the diaphragms are not rigid compared to the vertical elements of the lateral-force-resisting system, the model shall include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response.

2.5.11.2 Ground motion (NTHA)

The actual time-dependent inelastic deformation of the structure is modeled. For inelastic analysis method, the real design acceleration response spectrum (Sec 2.5.4.3) is obtained using Eq. 6.2.34 with R=1 and I=1. The real design acceleration response spectrum is the true representation of the expected ground motion (design basis earthquake) including local soil effects and corresponds to a peak ground acceleration (PGA) value of $\frac{2}{3}ZS$.

At least three appropriate acceleration time histories shall be used in the analysis. Ground motion shall conform to the requirements of this Section.

Two-dimensional analysis

Where two-dimensional analyses are performed, each ground motion shall consist of a horizontal acceleration time history selected from an actual recorded event. Appropriate acceleration histories shall be obtained from records of events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of appropriate ground motion records are not available, appropriate simulated ground motion time histories shall be used to make up the total number required. The ground motions shall be scaled such that for each period between 0.2 T and 1.5 T (where T is the natural period of the structure in the fundamental mode for the direction considered) the average of the five-percent-damped response spectra for each acceleration time history is not less than the corresponding ordinate of the real design acceleration response spectrum, as defined here.

Three-dimensional analysis

Where three-dimensional analysis is performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration time histories (in two orthogonal horizontal directions) that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of recorded ground motion pairs are not available, appropriate simulated ground motion pairs shall be used to make up the total number required. For each pair of horizontal ground motion components, an SRSS spectrum shall be constructed by taking the square root of the sum of the squares of the five-percent-damped response spectra for the components (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that for each period between 0.2 T and 1.5 T (where T is the natural period of the fundamental mode of the structure) the average of the SRSS spectra from all horizontal component pairs is not less than 1.3 times the corresponding ordinate of the real design acceleration response spectrum.

2.5.11.3 Structure response (NTHA)

For each scaled acceleration time history, the maximum values of base shear and other structure response quantities shall be obtained from the nonlinear time history analysis. For three dimensional analysis, orthogonal pair of scaled motions are applied simultaneously. If number of earthquake records (or pairs) used in the analysis is less than seven, the maximum structural response obtained corresponding to different earthquake records shall be considered as the design value. If the number is at least seven, then the average of maximum structural responses for different earthquake records shall be considered as the design value. Since real expected earthquake motion input and model incorporating real nonlinear behavior of the structure is used, the results as obtained are directly used (no scaling as in LTHA or RSA is required) for interpretation and design.

2.5.11.4 Structure member design (NTHA)

The adequacy of individual members and their connections to withstand the design deformations predicted by the analyses shall be evaluated based on laboratory test data for similar components. The effects of gravity and other loads on member deformation capacity shall be considered in these evaluations. Member deformation shall not exceed two thirds of the smaller of: the value that results in loss of ability to carry gravity loads or the value at which member strength has deteriorated to less than 67 percent of peak strength.

2.5.11.5 Design review (NTHA)

Special care and expertise is needed in the use of nonlinear dynamic analysis based design. Checking of the design by competent third party is recommended. A review of the design of the seismic-force-resisting system and the supporting structural analyses shall be performed by an independent team consisting of design professionals with experience in seismic analysis methods and the theory and application of nonlinear seismic analysis and structural behavior under extreme cyclic loads. The design review shall include the following: (i) Review of development of ground motion time histories (ii) Review of acceptance criteria (including laboratory test data) used to demonstrate the adequacy of structural elements and systems to withstand the calculated force and deformation demands (iii) Review of structural design.

2.5.12 Non-Linear Static Analysis (NSA)

Nonlinear static analysis (NSA), also popularly known as pushover analysis, is a simplified method of directly evaluating nonlinear response of structures to strong earthquake ground shaking. It is an alternative to the more complex nonlinear time history analysis (NTHA). The building is subjected to monotonically increasing static horizontal loads under constant gravity load.

2.5.12.1 Modeling (NSA)

A mathematical model of the structure shall be constructed to represent the spatial distribution of mass and stiffness of the structural system considering the effects of element nonlinearity for deformation levels that exceed the proportional limit. P-Delta effects shall also be included in the analysis.

For regular structures with independent orthogonal seismic-force-resisting systems, independent two-dimensional models may be used to represent each system. For structures having plan irregularities or structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three degrees of freedom for each level of the structure, consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis, shall be used. Where the diaphragms are not rigid compared to the vertical elements of the seismic-force-resisting system, the model should include representation of the diaphragm flexibility.

Unless analysis indicates that an element remains elastic, a nonlinear force deformation model shall be used to represent the stiffness of the element before onset of yield, the yield strength, and the stiffness properties of the element after yield at various levels of deformation. Strengths of elements shall not exceed expected values considering material over-strength and strain hardening. The properties of elements and components after yielding shall account for strength and stiffness degradation due to softening, buckling, or fracture as indicated by principles of mechanics or test data.

A control point shall be selected for the model. For normal buildings, the control point shall be at the center of mass of the highest level (roof) of the structure.

2.5.12.2 Analysis procedure (NSA)

The lateral forces shall be applied at the center of mass of each level and shall be proportional to the distribution obtained from a modal analysis for the fundamental mode of response in the direction under consideration. The lateral loads shall be increased incrementally in a monotonic manner.

At the j^{th} increment of lateral loading, the total lateral force applied to the model shall be characterized by the term V_j. The incremental increases in applied lateral force should be in steps that are sufficiently small to permit significant changes in individual element behavior (such as yielding, buckling or failure) to be detected. The first increment in lateral loading shall result in linear elastic behavior. At each loading step, the total applied lateral force, V_j the lateral displacement of the control point, δ_j and the forces and deformations in each element shall be recorded. The analysis shall be continued until the displacement of the control point is at least 150 percent of the target displacement determined in accordance with Sec.2.5.12.3. The structure shall be designed so that the total applied lateral force does not decrease in any load increment for control point displacements less than or equal to 125 percent of the target displacement.

2.5.12.3 Effective period and target displacement (NSA)

A bilinear curve shall be fitted to the capacity curve, such that the first segment of the bilinear curve coincides with the capacity curve at 60 percent of the effective yield strength, the second segment coincides with the capacity curve at the target displacement, and the area under the bilinear curve equals the area under the capacity curve, between the origin and the target displacement. The effective yield strength, V_y corresponds to the total applied lateral force at the intersection of the two line segments. The effective yield displacement, δ_y corresponds to the control point displacement at the intersection of the two line segments. The effective fundamental period, T_e of the structure in the direction under consideration shall be determined using Eq. 6.2.51 as follows:

$$T_e = T_1 \sqrt{\frac{V_1/\delta_1}{V_y/\delta_y}} \tag{6.2.51}$$

Where, V_1, δ_1, and T_1 are determined for the first increment of lateral load. The target displacement of the control point, δ_T shall be determined as follows:

$$\delta_T = C_0 C_1 S_a \left(\frac{T_e}{2\pi}\right)^2 g \tag{6.2.52}$$

Where, the spectral acceleration, S_a, is determined at the effective fundamental period, T_e, using Eq. 6.2.34, g is the acceleration due to gravity. The coefficient C_o shall be calculated as :

$$C_o = \frac{\sum_{i=1}^{n} w_i \phi_i}{\sum_{i=1}^{n} w_i \phi_i^2} \tag{6.2.53}$$

Where,

w_i = the portion of the seismic weight, W, at level i, and

ϕ_i = the amplitude of the shape vector at level i.

Where the effective fundamental period, T_e, is greater than T_C (defined in Sec. 2.5.4.3), the coefficient C_1 shall be taken as 1.0. Otherwise, the value of the coefficient C_1 shall be calculated as follows:

$$C_1 = \frac{1}{R_d}\left(1 + \frac{(R_d - 1)T_s}{T_e}\right) \tag{6.2.54}$$

Where, R_d is given as follows:

$$R_d = \frac{S_a}{V_y / W} \tag{6.2.55}$$

2.5.12.4 Structure member design (NSA)

For each nonlinear static analysis the design response parameters, including the individual member forces and member deformations shall be taken as the values obtained from the analysis at the step at which the target displacement is reached.

The adequacy of individual members and their connections to withstand the member forces and member deformations shall be evaluated based on laboratory test data for similar components. The effects of gravity and other loads on member deformation capacity shall be considered in these evaluations. The deformation of a member supporting gravity loads shall not exceed (i) two-thirds of the deformation that results in loss of ability to support gravity loads, and (ii) two-thirds of the deformation at which the member strength has deteriorated to less than 70 percent of the peak strength of the component model. The deformation of a member not required for gravity load support shall not exceed two-thirds of the value at which member strength has deteriorated to less than 70 percent of the peak strength of the component model.

2.5.12.5 Design review (NSA)

Checking of the design by competent third party is recommended. An independent team composed of at least two members with experience in seismic analysis methods and the theory and application of nonlinear seismic analysis and structural behavior under earthquake loading, shall perform a review of the design of the seismic force resisting system and the supporting structural analyses. The design review shall include (i) review of any site-specific seismic criteria (if developed) employed in the analysis (ii) review of the determination of the target displacement and effective yield strength of the structure (iii) review of adequacy of structural elements and systems to withstand the calculated force and deformation demands, together with laboratory and other data (iv) review of structural design.

2.5.13 Earthquake Load Effects and Load Combinations

The seismic load effect, E, shall be determined in accordance with the following:

1. For use in load combination 5 in Section 2.7.3 or load combination 5 and 6 in Section 2.7.2, E shall be determined in accordance with the following equation,

   $E = E_h + E_v$

2. For use in load combination 7 in Section 2.7.3 or load combination 8 in Section 2.7.2, E shall be determined in accordance with following equation,

   $E = E_h - E_v$

Where,

E = total seismic load effect

E_h = effect of horizontal seismic forces as defined in Sections 2.5.7 or 2.5.9

E_v = effect of vertical seismic forces as defined in Section 2.5.13.2

2.5.13.1 Horizontal earthquake loading, E_h

The horizontal seismic load effect, E_h, shall be taken as the horizontal load effects of seismic base shear V (Sec 2.5.7 or 2.5.9) or component forces F_c (Sec 2.5.15).

The directions of application of horizontal seismic forces for design shall be those which will produce the most critical load effects. Earthquake forces act in both principal directions of the building simultaneously. In order to account for that,

(a) For structures of Seismic Design Category B, the design horizontal seismic forces are permitted to be applied independently in each of two orthogonal directions and orthogonal interaction effects are permitted to be neglected

(b) Structures of Seismic Design Category C and D shall, as a minimum, conform to the requirements of (a) for Seismic Design Category B and in addition the requirements of this Section. The structure of Seismic Design Category C with plan irregularity type V and Seismic Design Category D shall be designed for 100% of the horizontal seismic forces in one principal direction combined with 30% of the horizontal seismic forces in the orthogonal direction. Possible combinations are:

"±100% in x-direction ±30% in y-direction" or

"±30% in x-direction ±100% in y-direction"

The combination which produces most unfavourable effect for the particular action effect shall be considered. This approach may be applied to equivalent static analysis, response spectrum analysis and linear time history analysis procedure.

(c) Where three-dimensional analysis of a spatial structure model is performed as in 3D time history analysis, simultaneous application of accelerations in two directions shall be considered where the ground motions shall satisfy the conditions stated in Sections 2.5.10.2 or 2.5.11.2.

2.5.13.2 Vertical earthquake loading, E_v

The maximum vertical ground acceleration shall be taken as 50 percent of the expected horizontal peak ground acceleration (PGA). The vertical seismic load effect E_v may be determined as:

$$E_v = 0.50(a_h)D \qquad (6.2.56)$$

Where,

a_h = expected horizontal peak ground acceleration (in g) for design = $(2/3)ZS$

D = effect of dead load, S = site dependent soil factor (see Table 6.2.16).

2.5.13.3 Combination of earthquake loading with other loadings

When earthquake effect is included in the analysis and design of a building or structure, the provisions set forth in Sec 2.7 shall be followed to combine earthquake load effects, both horizontal and vertical, with other loading effects to obtain design forces etc.

2.5.13.4 Seismic Load Effect Including Overstrength Factor

Where specifically required, conditions requiring overstrength factor, Ω_o, applications shall be determined in accordance with the following,

1. For use in load combination 5 in Section 2.7.3 or load combinations 5 and 6 in Section 2.7.2, E shall be taken equal to E_m as determined in accordance with the following equation,

   $E_m = E_{mh} + E_v$

2. For use in load combination 7 in Section 2.7.3 or load combination 8 in Section 2.7.2, E shall be taken equal to E_m as determined in accordance with the following equation,

   $E_m = E_{mh} - E_v$

   where

   E_m = total seismic load effect including overstrength factor

   E_{mh} = effect of horizontal seismic forces as defined in Sections 2.5.7 or 2.5.9 including structural overstrength.

   E_v = effect of vertical seismic forces as defined in Section 2.5.13.2

The horizontal seismic load effect with overstrength factor, E_{mh}, shall be determined in accordance with the following equation:

$E_{mh} = \Omega_o E_h$

Where, Ω_o is the system overstrength factor as defined in Table 6.2.19. Like E_h, directional combinations as defined in Sec. 2.5.13.1.(b) is also applicable for calculating E_{mh}. The value of E_{mh} need not exceed the maximum force that can develop in the structure or element as determined by a rational, plastic mechanism analysis or nonlinear response analysis (static or dynamic) utilizing realistic expected values of material strengths.

2.5.13.5 Allowable Stress Increase for Load Combinations with Overstrength

Where allowable stress design methodologies are used with the seismic load effect defined in Section 2.5.13.4 applied in load combinations 5, 6, or 8 of Section 2.7.2, allowable stresses are permitted to be determined using an allowable stress increase of 1.2. This increase shall not be combined with increases in allowable stresses or load combination reductions otherwise permitted elsewhere by this standard.

2.5.13.6 Minimum Upward Force for Horizontal Cantilevers for Seismic Design Category D

In structures assigned to Seismic Design Category D, horizontal cantilever structural components shall be designed for a minimum net upward force of 0.2 times the dead load in addition to the applicable load combinations of Section 2.7.

2.5.14 Drift and Deformation

2.5.14.1 Storey drift limit

The design storey drift (Δ) of each storey, as determined in Sections 2.5.7, 2.5.9 or 2.5.10 shall not exceed the allowable storey drift (Δ_a) as obtained from Table 6.2.21 for any story.

For structures with significant torsional deflections, the maximum drift shall include torsional effects. For structures assigned to Seismic Design Category C or D having torsional irregularity, the design storey drift, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the storey under consideration. For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, the allowable storey drift for such linear elastic analysis procedures shall not exceed Δ_a/ρ where ρ is termed as a structural redundancy factor. The value of redundancy factor ρ may be considered as 1.0 with exception of structures of very low level of redundancy where ρ may be considered as 1.3.

For nonlinear time history analysis (NTHA), the storey drift obtained (Sec 2.5.11) shall not exceed 1.25 times the storey drift limit specified above for linear elastic analysis procedures.

2.5.14.2 Diaphragm deflection

The deflection in the plane of the diaphragm, as determined by engineering analysis, shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

Table 6.2.21: Allowable Storey Drift Limit (Δ_a)

Structure	Occupancy Category		
	I and II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

Notes:

1. h_{sx} is the story height below Level x.
2. There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the storey drifts.
3. Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.
4. Occupancy categories are defined in Table 6.1.1

2.5.14.3 Separation between adjacent structures

Buildings shall be protected from earthquake-induced pounding from adjacent structures or between structurally independent units of the same building maintaining safe distance between such structures as follows:

(i) for buildings, or structurally independent units, that do not belong to the same property, the distance from the property line to the potential points of impact shall not be less than the computed maximum horizontal displacement (Sec 2.5.7.7) of the building at the corresponding level.

(ii) for buildings, or structurally independent units, belonging to the same property, if the distance between them is not less than the square root of the sum of the squares (SRSS) of the computed maximum horizontal displacements (Sec 2.5.7.7) of the two buildings or units at the corresponding level.

(iii) if the floor elevations of the building or independent unit under design are the same as those of the adjacent building or unit, the above referred minimum distance may be reduced by a factor of 0.7

2.5.14.4 Special deformation requirement for seismic design category D

For structures assigned to Seismic Design Category D, every structural component not included in the seismic force–resisting system in the direction under consideration shall be designed to be adequate for the gravity load effects and the seismic forces resulting from displacement to the design story drift (Δ) as determined in accordance with Sec 2.5.7.7. Even where elements of the structure are not intended to resist seismic forces, their protection may be important. Where determining the moments and shears induced in components that are not included in the seismic force–resisting system in the direction under consideration, the stiffening effects of adjoining rigid structural and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used.

2.5.15 Seismic Design For Nonstructural Components

This Section establishes minimum design criteria for nonstructural components that are permanently attached to structures and for their supports and attachments. The following components are exempt from the requirements of this Section.

(1) Architectural components in Seismic Design Category B, other than parapets supported by bearing walls or shear walls, where the component importance factor, I_c is equal to 1.0.

(2) Mechanical and electrical components in Seismic Design Category B.

(3) Mechanical and electrical components in Seismic Design Category C where the importance factor, I_c is equal to 1.0.

(4) Mechanical and electrical components in Seismic Design Category D where the component importance factor, I_c is equal to 1.0 and either (a) flexible connections between the components and associated ductwork, piping, and conduit are provided, or (b) components are mounted at 1.2 m or less above a floor level and weigh 1780 N or less.

(5) Mechanical and electrical components in Seismic Design Category C or D where the component importance factor, I_c is equal to 1.0 and (a) flexible connections between the components and associated ductwork, piping, and conduit are provided, and (b) the components weigh 89 N or less or, for distribution systems, which weigh 73 N/m or less.

Where the individual weight of supported components and non-building structures with periods greater than 0.06 seconds exceeds 25 percent of the total seismic weight W, the structure shall be designed considering interaction effects between the structure and the supported components.

Testing shall be permitted to be used in lieu of analysis methods outlined in this Chapter to determine the seismic capacity of components and their supports and attachments.

2.5.15.1 Component importance factor

All components shall be assigned a component importance factor. The component importance factor, I_c shall be taken as 1.5 if any of the following conditions apply:

(1) The component is required to function after an earthquake,

(2) The component contains hazardous materials, or

(3) The component is in or attached to a occupancy category IV building and it is needed for continued operation of the facility.

All other components shall be assigned a component importance factor, I_c equal to 1.0.

2.5.15.2 Component force transfer

Components shall be attached such that the component forces are transferred to the structure. Component attachments that are intended to resist seismic forces shall be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity. A continuous load path of sufficient strength and stiffness between the component and the supporting structure shall be verified. Local elements of the supporting structure shall be designed for the component forces where such forces control the design of the elements or their connections. In this instance, the component forces shall be those determined in Sec 2.5.15.3, except that modifications to F_p and R_p due to anchorage conditions need not be considered. The design documents shall include sufficient information concerning the attachments to verify compliance with the requirements of these Provisions.

2.5.15.3 Seismic design force

The seismic design force, F_c, applied in the horizontal direction shall be centered at the component's center of gravity and distributed relative to the component's mass distribution and shall be determined as follows:

$$F_c = \frac{\alpha_c a_h W_c I_c}{R_c}\left(1+2\frac{z}{h}\right) \qquad (6.2.57)$$

Where,

$0.75 a_h W_c I_c \leq F_c \leq 1.5 a_h W_c I_c$

α_c = component amplification factor which varies from 1.0 to 2.5 (Table 6.2.22 or Table 6.2.23).

a_h = expected horizontal peak ground acceleration (in g) for design = 0.67ZS

W_c = weight of component

R_c = component response reduction factor which varies from 1.0 to 12.0 (Table 6.2.22 or Table 6.2.23)

z = height above the base of the point of attachment of the component, but z shall not be taken less than 0 and the value of z/h need not exceed 1.0

h = roof height of structure above the base

The force F_c shall be independently applied in at least two orthogonal horizontal directions in combination with service loads associated with the component. In addition, the component shall also be designed for a concurrent vertical force of $\pm 0.5 a_h W_c$.

Where non-seismic loads on nonstructural components exceed F_c such loads shall govern the strength design, but the seismic detailing requirements and limitations shall apply.

2.5.15.4 Seismic relative displacements

The relative seismic displacement, D_c for two connection points on the same structure A, one at a height h_x and other at height h_y, for use in component design shall be determined as follows:

$$D_c = \delta_{xA} - \delta_{yA} \qquad (6.2.58)$$

D_c shall not exceed $D_{c\,max}$ given by:

$$D_{c\,\max} = \frac{(h_x - h_y)\Delta_{aA}}{h_{sx}} \tag{6.2.59}$$

Where,

δ_{xA} = Deflection at level x of structure A

δ_{yA} = Deflection at level y of structure A

Δ_{aA} = Allowable story drift for structure A

h_x = Height (above base) of level x to which upper connection point is attached.

h_y = Height (above base) of level y to which lower connection point is attached.

h_{sx} = Story height used in the definition of the allowable drift Δ_a

For two connection points on separate structures, A and B, or separate structural systems, one at level x and the other at level y, D_c shall be determined as follows:

$$D_c = |\delta_{xA}| + |\delta_{yB}| \tag{6.2.60}$$

D_c shall not exceed $D_{c\,max}$ given by:

$$D_{c\,\max} = \frac{h_x \Delta_{aA}}{h_{sx}} + \frac{h_y \Delta_{aB}}{h_{sx}} \tag{6.2.61}$$

Where,

δ_{yB} = Deflection at level y of structure B

Δ_{aB} = Allowable story drift for structure B

The effects of relative seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate.

2.5.16 Design For Seismically Isolated Buildings

Buildings that use special seismic isolation systems for protection against earthquakes shall be called seismically isolated or base isolated buildings. Seismically isolated structure and every portion thereof shall be designed and constructed in accordance with the requirements of provisions presented in this Section.

2.5.16.1 General requirements for isolation system

The isolation system to be used in seismically isolated structures shall satisfy the following requirements:

(1) Design of isolation system shall consider variations in seismic isolator material properties over the projected life of structure including changes due to ageing, contamination, exposure to moisture, loadings, temperature, creep, fatigue, etc.

(2) Isolated structures shall resist design wind loads at all levels above the isolation interface. At the isolation interface, a wind restraint system shall be provided to limit lateral displacement in the isolation system to a value equal to that required between floors of the structure above the isolation interface.

(3) The fire resistance rating for the isolation system shall be consistent with the requirements of columns, walls, or other such elements in the same area of the structure.

(4) The isolation system shall be configured to produce a lateral restoring force such that the lateral force at the total design displacement is at least 0.025 W greater than the lateral force at 50% of the total design displacement.

(5) The isolation system shall not be configured to include a displacement restraint that limits lateral displacement due to the maximum considered earthquake to less than the total maximum displacement unless it is demonstrated by analysis that such engagement of restraint does not result in unsatisfactory performance of the structure.

(6) Each element of the isolation system shall be designed to be stable under the design vertical load when subjected to a horizontal displacement equal to the total maximum displacement.

(7) The factor of safety against global structural overturning at the isolation interface shall not be less than 1.0 for required load combinations. All gravity and seismic loading conditions shall be investigated. Seismic forces for overturning calculations shall be based on the maximum considered earthquake and the vertical restoring force shall be based on the seismic weight above the isolation interface.

(8) Local uplift of individual units of isolation system is permitted if the resulting deflections do not cause overstress or instability of the isolator units or other elements of the structure.

(9) Access for inspection and replacement of all components of the isolation system shall be provided.

(10) The designer of the isolation system shall establish a quality control testing program for isolator units. Each isolator unit before installation shall be tested under specified vertical and horizontal loads.

(11) After completion of construction, a design professional shall complete a final series of inspections or observations of structure separation areas and components that cross the isolation interface. Such inspections and observations shall confirm that existing conditions allow free and unhindered displacement of the structure to maximum design levels and that all components that cross the isolation interface as installed are able to accommodate the stipulated displacements.

(12) The designer of the isolation system shall establish a periodic monitoring, inspection, and maintenance program for such system.

(13) Remodeling, repair, or retrofitting at the isolation interface, including that of components that cross the isolation interface, shall be performed under the direction of a design professional experienced in seismic isolation systems.

Table 6.2.22: Coefficients α_c and R_c for Architectural Components

Architectural Component or Element	$\alpha_c{}^a$	R_c
Interior Nonstructural Walls and Partitions		
Plain (unreinforced) masonry walls	1.0	1.5
All other walls and partitions	1.0	2.5
Cantilever Elements (Unbraced or braced to structural frame below its center of mass) Parapets and cantilever interior nonstructural walls	2.5	2.5
Chimneys and stacks where laterally braced or supported by the structural frame	2.5	2.5
Cantilever Elements (Braced to structural frame above its center of mass)		
Parapets	1.0	2.5
Chimneys and Stacks	1.0	2.5
Exterior Nonstructural Walls	1.0	2.5

Architectural Component or Element	α_c[a]	R_c
Exterior Nonstructural Wall Elements and Connections		
Wall Element	1.0	2.5
Body of wall panel connections	1.0	2.5
Fasteners of the connecting system	1.25	1.0
Veneer		
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
Penthouses (except where framed by an extension of the building frame)	2.5	3.5
Ceilings		
All	1.0	2.5
Cabinets		
Storage cabinets and laboratory equipment	1.0	2.5
Access Floors		
Special access floors	1.0	2.5
All other	1.0	1.5
Appendages and Ornamentations	2.5	2.5
Signs and Billboards	2.5	2.5
Other Rigid Components		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability materials and attachments	1.0	1.5
Other Flexible Components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability materials and attachments	2.5	1.5

[a] A lower value for α_c is permitted where justified by detailed dynamic analysis. The value for α_c shall not be less than 1.0. The value of α_c equal to 1.0 is for rigid components and rigidly attached components. The value of α_c equal to 2.5 is for flexible components and flexibly attached components.

Table 6.2.23: Coefficients α_c and R_c for Mechanical and Electrical Components

Mechanical and Electrical Components	$\alpha_c{}^a$	R_c
Air-side HVAC, fans, air handlers, air conditioning units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal framing.	2.5	6.0
Wet-side HVAC, boilers, furnaces, atmospheric tanks and bins, chillers, water heaters, heat exchangers, evaporators, air separators, manufacturing or process equipment, and other mechanical components constructed of high-deformability materials.	1.0	2.5
Engines, turbines, pumps, compressors, and pressure vessels not supported on skirts and not within the scope of Chapter 15.	1.0	2.5
Skirt-supported pressure vessels	2.5	2.5
Elevator and escalator components.	1.0	2.5
Generators, batteries, inverters, motors, transformers, and other electrical components constructed of high deformability materials.	1.0	2.5
Motor control centers, panel boards, switch gear, instrumentation cabinets, and other components constructed of sheet metal framing.	2.5	6.0
Communication equipment, computers, instrumentation, and controls.	1.0	2.5
Roof-mounted chimneys, stacks, cooling and electrical towers laterally braced below their center of mass.	2.5	3.0
Roof-mounted chimneys, stacks, cooling and electrical towers laterally braced above their center of mass.	1.0	2.5
Lighting fixtures.	1.0	1.5
Other mechanical or electrical components.	1.0	1.5
Vibration Isolated Components and Systems^b		
Components and systems isolated using neoprene elements and neoprene isolated floors with built-in or separate elastomeric snubbing devices or resilient perimeter stops.	2.5	2.5
Spring isolated components and systems and vibration isolated floors closely restrained using built-in or separate elastomeric snubbing devices or resilient perimeter stops.	2.5	2.0
Internally isolated components and systems.	2.5	2.0
Suspended vibration isolated equipment including in-line duct devices and suspended internally isolated components.	2.5	2.5
Mechanical and Electrical Components	$\alpha_c{}^a$	R_c
Air-side HVAC, fans, air handlers, air conditioning units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal framing.	2.5	6.0
Wet-side HVAC, boilers, furnaces, atmospheric tanks and bins, chillers, water heaters, heat exchangers, evaporators, air separators, manufacturing or process equipment, and other mechanical components constructed of high-deformability materials.	1.0	2.5

Mechanical and Electrical Components	$\alpha_c{}^a$	R_c
Engines, turbines, pumps, compressors, and pressure vessels not supported on skirts and not within the scope of Chapter 15.	1.0	2.5
Skirt-supported pressure vessels	2.5	2.5
Distribution Systems		
Piping in accordance with ASME B31, including in-line components with joints made by welding or brazing.	2.5	12.0
Piping in accordance with ASME B31, including in-line components, constructed of high or limited deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings.	2.5	6.0
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing.	2.5	9.0
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high- or limited-deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings.	2.5	4.5
Piping and tubing constructed of low-deformability materials, such as cast iron, glass, and non-ductile plastics.	2.5	3.0
Ductwork, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing.	2.5	9.0
Ductwork, including in-line components, constructed of high- or limited-deformability materials with joints made by means other than welding or brazing.	2.5	6.0
Ductwork, including in-line components, constructed of low-deformability materials, such as cast iron, glass, and non-ductile plastics.	2.5	3.0
Electrical conduit, bus ducts, rigidly mounted cable trays, and plumbing.	1.0	2.5
Manufacturing or process conveyors (non-personnel).	2.5	3.0
Suspended cable trays.	2.5	6.0

[a] A lower value for α_c is permitted where justified by detailed dynamic analysis. The value for α_c shall not be less than 1.0. The value of α_c equal to 1.0 is for rigid components and rigidly attached components. The value of α_c equal to 2.5 is for flexible components and flexibly attached components.

[b] Components mounted on vibration isolators shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_c$ if the nominal clearance (air gap) between the equipment support frame and restraint is greater than 6 mm. If the nominal clearance specified on the construction documents is not greater than 6 mm, the design force may be taken as F_c.

2.5.16.2 Equivalent static analysis

The equivalent static analysis procedure is permitted to be used for design of a seismically isolated structure provided that:

(1) The structure is located on Site Class SA, SB, SC, SD or SE site;

(2) The structure above the isolation interface is not more than four stories or 20 m in height

(3) Effective period of the isolated structure at the maximum displacement, T_M, is less than or equal to 3.0 sec.

(4) The effective period of the isolated structure at the design displacement, T_D, is greater than three times the elastic, fixed-base period of the structure above the isolation system as determined in Sec. 2.5.7.2

(5) The structure above the isolation system is of regular configuration; and

(6) The isolation system meets all of the following criteria:

(a) The effective stiffness of the isolation system at the design displacement is greater than one third of the effective stiffness at 20 percent of the design displacement,

(b) The isolation system is capable of producing a restoring force as specified in Sec. 2.5.16.1,

(c) The isolation system does not limit maximum considered earthquake displacement to less than the total maximum displacement.

Where the equivalent lateral force procedure is used to design seismically isolated structures, the requirements of this Section shall apply.

2.5.16.2.1 Displacement of isolation system: The isolation system shall be designed and constructed to withstand minimum lateral earthquake displacements that act in the direction of each of the main horizontal axes of the structure and such displacements shall be calculated as follows:

$$D_D = \frac{S_a g}{4\pi^2}\left(\frac{T_D^2}{B_D}\right) \qquad (6.2.62)$$

Where,

S_a = Design spectral acceleration (in units of g), calculated using Eq. 6.2.34 for period T_D and assuming $R=1$, $I=1$, $\eta=1$ (Sec 2.5.4.3) for the design basis earthquake (DBE).

g = acceleration due to gravity

B_D= damping coefficient related to the effective damping β_D of the isolation system at the design displacement, as set forth in Table 6.2.24.

T_D = effective period of seismically isolated structure at the design displacement in the direction under consideration, as prescribed by Eq. 6.2.63:

$$T_D = 2\pi\sqrt{\frac{W}{k_{D\min}g}} \tag{6.2.63}$$

Where,

W = seismic weight above the isolation interface

k_{Dmin} = minimum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

Table 6.2.24: Damping Coefficient, B_D or B_M

Effective Damping, β_D or β_M [a, b] (%)	**B_D or B_M**
≤ 2	0.8
5	1.0
10	1.2
20	1.5
30	1.7
40	1.9
≥ 50	2.0

[a] The damping coefficient shall be based on the effective damping of the isolation system

[b] The damping coefficient shall be based on linear interpolation for effective damping values other than those given.

The maximum displacement of the isolation system, D_M, in the most critical direction of horizontal response shall be calculated in accordance with the following formula:

$$D_M = \frac{S_{aM} g}{4\pi^2}\left(\frac{T_M^2}{B_M}\right) \qquad (6.2.64)$$

Where:

S_{aM} = Maximum spectral acceleration (in units of g), calculated using Eq. 6.2.34 for period T_D and assuming $R=1$, $I=1$, $\eta=1$ (Sec 2.5.4.3) for the maximum considered earthquake (MCE).

B_M = numerical coefficient related to the effective damping β_M of the isolation system at the maximum displacement, as set forth in Table 6.2.24.

T_M = effective period of seismic-isolated structure at the maximum displacement in the direction under consideration as prescribed by:

$$T_M = 2\pi\sqrt{\frac{W}{k_{M\,\min} g}} \qquad (6.2.65)$$

Where,

$k_{M\,min}$ = minimum effective stiffness of the isolation system at the maximum displacement in the horizontal direction under consideration.

The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, of elements of the isolation system shall include additional displacement due to inherent and accidental torsion calculated considering the spatial distribution of the lateral stiffness of the isolation system and the most disadvantageous location of eccentric mass.

2.5.16.2.2 Lateral seismic forces: The structure above the isolation system shall be designed and constructed to withstand a minimum lateral force, V_s, using all of the appropriate provisions for a non-isolated structure. The importance factor for all isolated structures shall be considered as 1.0, also the response reduction factor R_I considered here (for computing design seismic forces) is in the range of 1.0 to 2.0. V_s shall be determined in accordance with Eq. 6.2.66 as follows:

$$V_s = \frac{k_{D\max} D_D}{R_I} \qquad (6.2.66)$$

Where,

$k_{D\,max} =$ maximum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

$D_D =$ design displacement at the center of rigidity of the isolation system in the direction under consideration as prescribed by Eq. 6.2.62.

$R_I =$ response reduction factor related to the type of seismic-force-resisting system above the isolation system. R_I shall be based on the type of seismic-force-resisting system used for the structure above the isolation system and shall be taken as the lesser of $\frac{3}{8}R$ (Table 6.2.19) or 2.0, but need not be taken less than 1.0.

In no case shall V_s be taken less than the following:

(1) The lateral force required by Sec 2.5.7 for a fixed-base structure of the same weight, W, and a period equal to the isolated period, T_D;

(2) The base shear corresponding to the factored design wind load; and

(3) The lateral force required to fully activate the isolation system (e.g., the yield level of a softening system, the ultimate capacity of a sacrificial wind-restraint system, or the break-away friction level of a sliding system) multiplied by 1.5.

The isolation system, the foundation, and all structural elements below the isolation system shall be designed and constructed to withstand a minimum lateral force, V_b using all of the appropriate provisions for a non-isolated structure. V_b shall be determined in accordance with Eq. 6.2.67 as follows:

$$V_b = k_{Dmax} D_D \qquad (6.2.67)$$

In all cases, V_b shall not be taken less than the maximum force in the isolation system at any displacement up to and including the design displacement.

2.5.16.2.3 Vertical distribution of lateral forces: The total lateral force shall be distributed over the height of the structure above the isolation interface in accordance with Eq. 6.2.68 as follows:

$$F_x = V_s \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (6.2.68)$$

Where:

$V_s =$ Total seismic lateral design force on elements above the isolation system.

$h_i, h_x =$ Height above the base, to Level i or Level x, respectively.

$w_i, w_x =$ Portion of W that is located at or assigned to Level i or Level x, respectively.

At each Level x the force, F_x shall be applied over the area of the structure in accordance with the distribution of mass at the level. Stresses in each structural element shall be determined by applying the lateral forces, F_x at all levels above the base to an analytical model.

2.5.16.2.4 Storey drift: The storey drift shall be calculated as in Sec 2.5.7.7 except that C_d for the isolated structure shall be taken equal to R_I and importance factor equal to 1.0. The maximum storey drift of the structure above the isolation system shall not exceed $0.015h_{sx}$.

2.5.16.3 Dynamic analysis

Response spectrum analysis may be conducted if the behavior of the isolation system can be considered as equivalent linear. Otherwise, non-linear time history analysis shall be used where the true non-linear behaviour of the isolation system can be modeled. The mathematical models of the isolated structure including the isolation system shall be along guidelines given in Sections 2.5.9.1 and 2.5.11.1, and other requirements given in Sec 2.5.16.

The isolation system shall be modeled using deformational characteristics developed and verified by testing. The structure model shall account for: (i) spatial distribution of isolator units; (ii) consideration of translation in both horizontal directions, and torsion of the structure above the isolation interface considering the most disadvantageous location of eccentric mass; (iii) overturning/uplift forces on individual isolator units; and (iv) effects of vertical load, bilateral load, and the rate of loading if the force-deflection properties of the isolation system are dependent on such attributes.

A linear elastic model of the isolated structure (above isolation system) may be used provided that: (i) stiffness properties assumed for the nonlinear components of the isolation system are based on the maximum effective stiffness of the isolation system, and (ii) all elements of the seismic-force-resisting system of the structure above the isolation system behave linearly.

2.5.16.3.1 Response Spectrum Analysis: Response spectrum analysis shall be performed using a modal damping value for the fundamental mode in the direction of interest not greater than the effective damping of the isolation system or 30 percent of critical, whichever is less. Modal damping values for higher modes shall be selected consistent with those that would be appropriate for response spectrum analysis of the structure above the isolation system assuming a fixed base.

Response spectrum analysis used to determine the total design displacement and the total maximum displacement shall include simultaneous excitation of the model by 100 percent of the ground motion in the critical direction and 30 percent of the ground motion in the perpendicular, horizontal direction. The design basis earthquake shall be used for the design displacement, while the maximum considered earthquake shall be used for the maximum displacement. The maximum displacement of the isolation system shall be calculated as the vectorial sum of the two orthogonal displacements.

For the design displacement, structures that do not require site-specific ground motion evaluation, shall be analyzed using the design acceleration response spectrum in accordance with Sec 2.5.4.3. The maximum design spectrum to be used for the maximum considered earthquake shall not be less than 1.5 times the design acceleration response spectrum.

The response spectrum procedure is based on an equivalent linear model, where the effective stiffness and effective damping is a function of the displacement, this formulation is thus an iterative process. The effective stiffness must be estimated, based on assumed displacement, and then adjusted till obtained displacement agree with assumed displacement.

The design shear at any story shall not be less than the story shear resulting from application of the story forces calculated using Eq. 6.2.68 with a value of V_S equal to the base shear obtained from the response spectrum analysis in the direction of interest.

2.5.16.3.2 Nonlinear Time History Analysis: Where a time history analysis procedure is performed, not fewer than three appropriate ground motions shall be used in the analysis as described below.

Ground motions shall consist of pairs of appropriate horizontal ground motion acceleration components that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. If required number of recorded ground motion pairs are not available, appropriate simulated ground motion pairs shall be used to make up the total number required. For each pair of horizontal ground-motion components, a square root of the sum of the squares (SRSS) spectrum shall be constructed by taking the SRSS of the 5 percent damped response spectra for the scaled components (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that for each period between $0.5T_D$ and $1.25T_M$ (where T_D and T_M are defined in Sec 2.5.16.2.1) the average of the SRSS spectra from all horizontal component pairs does not fall below 1.3 times the corresponding ordinate of the design response spectrum (Sec 2.5.16.4), by more than 10 percent.

Each pair of ground motion components shall be applied simultaneously to the model considering the most disadvantageous location of eccentric mass. The maximum displacement of the isolation system shall be calculated from the vectorial sum of the two orthogonal displacements at each time step.

The parameters of interest shall be calculated for each ground motion used for the time history analysis. If at least seven ground motions are used for the time history analysis, the average value of the response parameter of interest is permitted to be used for design. If fewer than seven ground motions are analyzed, the maximum value of the response parameter of interest shall be used for design.

2.5.16.3.3 Storey drift: Maximum story drift corresponding to the design lateral force including displacement due to vertical deformation of the isolation system shall not exceed the following limits:

1. The maximum story drift of the structure above the isolation system calculated by response spectrum analysis shall not exceed $0.015h_{sx}$.

2. The maximum story drift of the structure above the isolation system calculated by nonlinear time history analysis shall not exceed $0.020h_{sx}$.

The storey drift shall be calculated as in Sec 2.5.7.7 except that C_d for the isolated structure shall be taken equal to R_I and importance factor equal to 1.0.

2.5.16.4 Testing

The deformation characteristics and damping values of the isolation system used in the design and analysis of seismically isolated structures shall be based on test results of isolator units. The tests are for establishing and validating the design properties of the isolation system and shall not be considered as satisfying the manufacturing quality control tests.

The following sequence of tests shall be performed on isolator units for the prescribed number of cycles at a vertical load equal to the average dead load plus one-half the effects due to live load on all isolator units of a common type and size:

(1) Twenty fully reversed cycles of loading at a lateral force corresponding to the wind design force.

(2) Three fully reversed cycles of loading at each of the following increments of the total design displacement-0.25D_D, 0.5D_D, 1.0D_D, and 1.0D_M where D_D and D_M are as determined in Sec 2.5.16.2.1.

(3) Three fully reversed cycles of loading at the total maximum displacement, 1.0D_{TM}.

(4) Not less than ten fully reversed cycles of loading at 1.0 times the total design displacement, 1.0D_{TD}.

For each cycle of each test, the force-deflection and hysteretic behavior of each isolator unit shall be recorded. The effective stiffness is obtained as the secant value of stiffness at design displacement while the effective damping is determined from the area of hysteretic loop at the design displacement.

2.5.16.5 Design review

A design review of the isolation system and related test programs shall be performed by an independent team of design professionals experienced in seismic analysis methods and the application of seismic isolation. Isolation system design review shall include, but need not be limited to, the following:

(1) Review of site-specific seismic criteria including the development of site-specific spectra and ground motion time histories and all other design criteria developed specifically for the project;

(2) Review of the preliminary design including the determination of the total design displacement of the isolation system and the lateral force design level;

(3) Overview and observation of prototype (isolator unit) testing

(4) Review of the final design of the entire structural system and all supporting analyses; and

(5) Review of the isolation system quality control testing program.

2.5.17 Buildings with Soft Storey

Buildings with possible soft storey action at ground level for providing open parking spaces belong to structures with major vertical irregularity [Figure 6.2.28(a)]. Special arrangement is needed to increase the lateral strength and stiffness of the soft/open storey. The following two approaches may be considered:

(1) Dynamic analysis of such building may be carried out incorporating the strength and stiffness of infill walls and inelastic deformations in the members, particularly those in the soft storey, and the members designed accordingly.

(2) Alternatively, when system overstrength factor, Ω_o, is not included in determining seismic load effects, the following design criteria are to be adopted after carrying out the earthquake analysis, neglecting the effect of infill walls in other storeys. Structural elements (e.g columns and beams) of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads neglecting effect of infill walls. Shear walls placed symmetrically in both directions of the building as far away from the centre of the building as feasible are to be designed exclusively for 1.5 times the lateral shear force calculated before.

2.5.18 Non-Building Structures

Calculation of seismic design forces on non-building structures (e.g. chimney, self-supported overhead water/fluid tank, silo, trussed tower, storage tank, cooling tower, monument and other structures not covered in Sec 2.5) shall be in accordance with "Chapter 15: Seismic Design Requirements for Non-Building Structures, Minimum Design Loads for Buildings and Other Structures, ASCE Standard ASCE/SEI 7-05" complying with the requirements of Sec 2.5 of this Code.

2.6 Miscellaneous Loads

2.6.1 General

The procedures and limitations for the determination of selected miscellaneous loads are provided in this Section. Loads that are not specified in this Section or elsewhere in this Chapter, may be determined based on information from reliable references or specialist advice may be sought.

2.6.2 Rain Loads

Rain loads shall be determined in accordance with the following provisions.

2.6.2.1 Blocked drains

Each portion of a roof shall be designed to sustain the load from all rainwater that could be accumulated on it if the primary drainage system for that portion is undersized or blocked. Ponding instability shall be considered in this situation.

2.6.2.2 Controlled drainage

Roofs equipped with controlled drainage provisions shall be designed to sustain all rainwater loads on them to the elevation of the secondary drainage system plus 0.25 kN/m^2. Ponding instability shall be considered in this situation.

2.6.3 Loads Due to Flood and Surge

For the determination of flood and surge loads on a structural member, consideration shall be given to both hydrostatic and hydrodynamic effects. Required loading shall be determined in accordance with the established principles of mechanics based on site specific criteria and in compliance with the following provisions of this Section. For essential facilities like cyclone and flood shelters and for hazardous facilities specified in Table 6.1.1, values of maximum flood elevation, surge height, wind velocities etc., required for the determination of flood and surge load, shall be taken corresponding to 100-year return period. For structures other than essential and hazardous facilities, these values shall be based on 50-year return period.

2.6.3.1 Flood loads on structures at inland areas

For structures sited at inland areas subject to flood, loads due to flood shall be determined considering hydrostatic effects which shall be calculated based on the flood elevation of 50-year return period. For river-side structures such as that under Exposure C specified in Sec 2.4.6.3, hydrodynamic forces, arising due to approaching wind-generated waves shall also be determined in addition to the hydrostatic load on them. In this case, the amplitude of such wind-induced water waves shall be obtained from site-specific data.

2.6.3.2 Flood and surge loads on structures at coastal areas

Coastal area of Bangladesh has been delineated as Risk Area (RA) and High Risk Area (HRA) based on the possible extend of the inland intrusion of the cyclone storm surge as shown in Figure 6.2.30. To be classified as coastal RISK AREA, the principal source of flooding must be sea tides, storm surge, and not riverine flood. The RA extends from the coast line to an inland limit up to which surge water can reach. The HRA includes a strip of land within the RA. It extends from the coast line up to the limit where the depth of storm surge inundation may exceed 1m.Entire area of the off-shore islands except the Maheshkhali area is included in the HRA. A part of Maheshkhali is covered by hills and therefore free from inundation. However, the western and northern parts of Maheshkhali are of low elevation and risk inundation. For structures sited in coastal areas (Risk Areas), the hydrostatic and hydrodynamic loads shall be determined as follows:

2.6.3.2.1 Hydrostatic Loads

The hydrostatic loads on structural elements and foundations shall be determined based on the maximum static height of water, H_m, produced by floods or surges as given by the relation:

$$H_m = max(h_s, h_f) \quad (6.2.69)$$

$$h_f = y_T - y_g \quad (6.2.70)$$

Where,

h_s = Maximum surge height as specified in (i) below.

y_T = Elevation of the extreme surface water level corresponding to a T-year return period specified in (ii) below, meters

y_g = Elevation of ground level at site, meters.

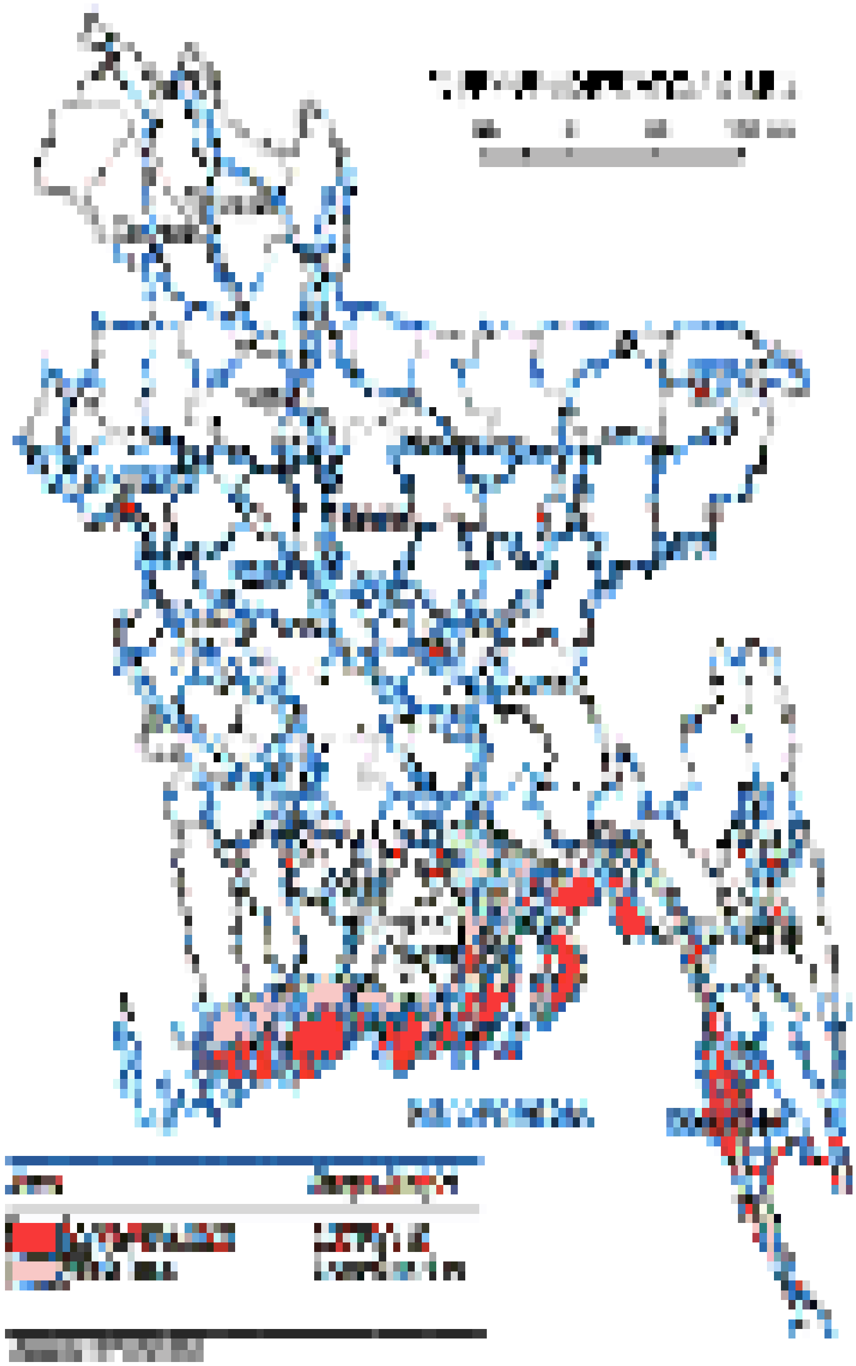


Figure 6.2.30 Coastal risk areas (RA) and high risk areas (HRA) of Bangladesh

(i) Maximum Surge Height, h_s: The maximum surge height, h_s, associated with cyclones, shall be that corresponding to a 50-year or a 100-year return period as may be applicable, based on site specific analysis. In the absence of a more rigorous site specific analysis, the following relation may be used:

$$h_s = h_T - (x - 1)k \qquad (6.2.71)$$

Where, h_T = design surge height corresponding to a return period of T-years at sea coast, in metres, given in Table 6.2.25.

x = distance of the structure site measured from the spring tide high-water limit on the sea coast, in km; x= 1, if x<1.

k = rate of decrease in surge height in meter/km; the value of k may be taken as 0.5 for Chittagong-Cox's Bazar-Teknaf coast and as 0.33 for other coastal areas.

(ii) Extreme Surface Water Level, y_T: The elevation of the extreme surface water level, y_T for a site, which may not be associated with a cyclonic storm surge, shall be that obtained from a site specific analysis corresponding to a 50-year or a 100-year return period. Values of y_T are given in Table 6.2.26 for selected coastal locations which may be used in the absence of any site specific data.

Hydrostatic loads caused by a depth of water to the level of the H_m shall be applied over all surfaces involved, both above and below ground level, except that for surfaces exposed to free water, the design depth H_m shall be increased by 0.30 m. Reduced uplift and lateral loads on surfaces of enclosed spaces below the H_m shall apply only if provision is made for entry and exit of floodwater.

Table 6.2.25: Design Surge Heights at the Sea Coast, h_T*

Coastal Region	**Surge Height at the Sea Coast, h_T (m)**	
	***T* = 50-year(1)**	***T* = 100-year(2)**
Teknaf to Cox's Bazar	4.5	5.8
Chakaria to Anwara, and Maheshkhali-Kutubdia Islands	7.1	8.6
Chittagong to Noakhali	7.9	9.6
Sandwip, Hatiya and all islands in this region	7.9	9.6
Bhola to Barguna	6.2	7.7
Sarankhola to Shyamnagar	5.3	6.4

Notes:

* Values prepared from information obtained from Annex-D3, MCSP.

(1) These values may be used in the absence of site specific data for structures other than essential facilities listed in Table 6.1.1.

(2) These values may be used in the absence of site specific data for essential facilities listed in Table 6.1.1.

Table 6.2.26: Extreme Surface Water Levels above PWD Datum, y_T^* at Coastal Areas during Monsoon

Coastal Area		y_T (m)	
Location	**Thana**	**T = 50 years(1)**	**T = 100 years(2)**
Teknaf	Teknaf	2.33	2.44
Cox's Bazar	Cox's Bazar	3.84	3.88
Shaflapur	Moheshkhali	4.67	4.87
Lemsikhali	Kutubdia	4.95	5.19
Banigram	Patiya	5.05	5.24
Chittagong	Bandar	4.72	4.88
Patenga	Bandar	4.08	4.16
Sonapur	Sonagazi	7.02	7.11
Sandwip	Sandwip	6.09	6.2
Companyganj	Companyganj	7.53	7.94
Hatiya	Hatiya	5.55	5.76
Daulatkhan	Daulatkhan	4.62	4.72
Dashmina	Dashmina	3.60	3.73
Galachipa	Galachipa	3.79	3.92
Patuakhali	Patuakhali	2.87	3.03
Khepupara	Kalapara	2.93	3.02
Bamna	Bamna	3.32	3.37
Patharghata	Patharghata	3.65	3.84
Raenda	Sarankhola	3.66	3.75
Chardouni	Patharghata	4.41	4.66
Mongla	Mongla port	3.23	3.36
Kobodak (river estuary)	Shyamnagar	3.51	3.87
Kaikhali	Shyamnagar	3.94	4.12

Notes:

* Values prepared from information obtained from Annex -D3, MCSP

(1) These values may be used in the absence of site specific data for structures in Structure Occupancy Category IV listed Table 6.1.1.

(2) These values may be used in the absence of site specific data for structures in Structure Occupancy Categories I, II and III listed in Table 6.1.1.

2.6.3.2.2 Hydrodynamic loads

The hydrodynamic load applied on a structural element due to wind-induced local waves of water, shall be determined by a rational analysis using an established method of fluid mechanics and based on site specific data. In the absence of a site-specific data the amplitude of the local wave, to be used in the rational analysis, shall be taken as $h_w = \frac{h_s}{4} \geq 1$ m, where, h_s is given in Sec 2.6.3.2.1. Such forces shall be calculated based on 50-year or 100-year return period of flood or surge. The corresponding wind velocities shall be 80 m/s or 90 m/s (3-sec gust) respectively.

Exception:

Where water velocities do not exceed 3.0 m/s, dynamic effects of moving water shall be permitted to be converted into equivalent hydrostatic loads by increasing H_m for design purposes by an equivalent surcharge depth, d_h, on the headwater side and above the ground level only, equal to

$$d_h = \frac{aV^2}{2g} \qquad (6.2.72)$$

Where,

V = average velocity of water in m/s

g = acceleration due to gravity, 9.81 m/s^2

a = coefficient of drag or shape factor (not less than 1.25)

In absence of more authentic site specific data, the velocity of water, V, may be estimated such that $d_s \leq V \leq \sqrt{gd_s}$ where g is the acceleration due to gravity and d_s is defined in Sec 2.6.3.4. Selection of the correct value of drag-coefficient a in Eq. 6.2.72 will depend upon the shape and roughness of the object exposed to flood flow, as well as the flow condition. As a general rule, the smoother and more streamlined the object, the lower the drag coefficient (shape factor). Drag coefficients for elements common in buildings and structures (round or square piles, columns, and rectangular shapes) will range from approximately 1.0 to 2.0, depending upon flow conditions. However, given the uncertainty surrounding flow conditions at a particular site, it is recommended that a minimum value of 1.25 be used. Fluid mechanics texts should be consulted for more information on when to apply drag coefficients above 1.25.

The equivalent surcharge depth, d_h, shall be added to the design depth H_m and the resultant hydrostatic pressures applied to, and uniformly distributed across, the vertical projected area of the building or structure that is perpendicular to the flow. Surfaces parallel to the flow or surfaces wetted by the tail water shall be subject to the hydrostatic pressures for depths to the H_m only.

2.6.3.3 Breakaway walls

Walls and partitions required to break away, including their connections to the structure, shall be designed for the largest of the following loads acting perpendicular to the plane of the wall:

(i) The wind load specified in Sec. 2.4.

(ii) The earthquake load specified in Sec. 2.5.

(iii) 0.50 kN/m^2 pressure.

The loading at which breakaway walls are intended to collapse shall not exceed 1.0 kN/m^2 unless the design meets the following conditions:

(i) Breakaway wall collapse is designed to result from a flood load less than that which occurs during the base flood.

(ii) The supporting foundation and the elevated portion of the building shall be designed against collapse, permanent lateral displacement, and other structural damage due to the effects of flood loads in combination with other loads as specified elsewhere in this Chapter.

2.6.3.4 Wave loads

Wave loads shall be determined by one of the following three methods: (1) by using the analytical procedures outlined in this Section, (2) by more advanced numerical modeling procedures, or (3) by laboratory test procedures (physical modeling).

Wave loads are those loads that result from water waves propagating over the water surface and striking a building or other structure. Design and construction of buildings and other structures subject to wave loads shall account for the following loads: a) waves breaking on any portion of the building or structure; b) uplift forces caused by shoaling waves beneath a building or structure, or portion thereof; c) wave runup striking any portion of the building or structure; d) wave-induced drag and inertia forces; and e) wave-induced scour at the base of a building or structure, or its foundation. Nonbreaking and broken wave loads shall be calculated using the procedures described in Sections 2.6.3.2.1 and 2.6.3.2.2 that show how to calculate hydrostatic and hydrodynamic loads.

Breaking wave loads shall be calculated using the procedures described in Sections 2.6.3.4.1 to 2.6.3.4.4. Breaking wave heights used in the procedures described in these Sections shall be calculated for using Equations 6.2.73 and 6.2.74.

$$H_b = 0.78\, d_s \qquad (6.2.73)$$

Where,

H_b = breaking wave height in meter.

d_s = local still water depth in meter.

The local still water depth shall be calculated using Eq. 6.2.74 unless more advanced procedures or laboratory tests permitted by this Section are used.

$$d_s = 0.65 H_m \qquad (6.2.74)$$

2.6.3.4.1 Breaking wave loads on vertical pilings and columns

The net force resulting from a breaking wave acting on a rigid vertical pile or column shall be assumed to act at the still water elevation and shall be calculated by the following:

$$F_D = 0.5 \gamma_w C_D D H_b^2 \qquad (6.2.75)$$

Where,

F_D = net wave force, in kN.

γ_w = unit weight of water, in kN/m^3 = 9.80 kN/m^3 for fresh water and 10.05 kN/m^3 or salt water.

C_D = coefficient of drag for breaking waves, = 1.75 for round piles or columns, and = 2.25 for square piles or columns.

D = pile or column diameter, in meter for circular sections, or for a square pile or column, 1.4 times the width of the pile or column in meter.

H_b = breaking wave height, in meter.

2.6.3.4.2 Breaking wave loads on vertical walls

Maximum pressures and net forces resulting from a normally incident breaking wave (depth-limited in size, with $H_b = 0.78d_s$ acting on a rigid vertical wall shall be calculated by the following:

$$P_{max} = C_p\gamma_w d_s + 1.2\gamma_w d_s \tag{6.2.76}$$

$$F_t = 1.1C_p\gamma_w d_s^2 + 2.4\gamma_w d_s^2 \tag{6.2.77}$$

Where,

P_{max} = maximum combined dynamic $\left(C_p\gamma_w d_s\right)$ and static $(1.2\gamma_w d_s)$ wave pressures, also referred to as shock pressures in kN/m^2.

F_t = net breaking wave force per unit length of structure, also referred to as shock, impulse, or wave impact force in kN/m, acting near the still water elevation.

C_p = dynamic pressure coefficient. It shall be taken as 1.6, 2.8, 3.2 or 3.5 for building occupancy categories I, II, III or IV respectively.

γ_w = unit weight of water, in kN/m^3 = 9.80 kN/m^3 for fresh water and 10.05 kN/m^3 for salt water

d_s = still water depth in meter at base of building or other structure where the wave breaks.

This procedure assumes the vertical wall causes a reflected or standing wave against the water ward side of the wall with the crest of the wave at a height of $1.2d_s$ above the still water level. Thus, the dynamic static and total pressure distributions against the wall are as shown in Figure 6.2.31.

This procedure also assumes the space behind the vertical wall is dry, with no fluid balancing the static component of the wave force on the outside of the wall. If free water exists behind the wall, a portion of the hydrostatic component of the wave pressure and force disappears (Figure 6.2.32) and the net force shall be computed by Eq. 6.2.78 (the maximum combined wave pressure is still computed with Eq. 6.2.76).

$$F_t = 1.1C_p\gamma_w d_s^2 + 1.9\gamma_w d_s^2 \tag{6.2.78}$$

Where,

F_t = net breaking wave force per unit length of structure, also referred to as shock, impulse, or wave impact force in kN/m, acting near the still water elevation.

C_p = dynamic pressure coefficient. It shall be taken as 1.6, 2.8, 3.2 or 3.5 for building occupancy categories I, II, III or IV respectively.

γ_w = unit weight of water, in kN/m^3 = 9.80 kN/m^3 for fresh water and 10.05 kN/m^3 for salt water

d_s = still water depth in meter at base of building or other structure where the wave breaks.

2.6.3.4.3 Breaking wave loads on nonvertical walls

Breaking wave forces given by Equations 6.2.77 and 6.2.78 shall be modified in instances where the walls or surfaces upon which the breaking waves act are nonvertical. The horizontal component of breaking wave force shall be given by

$$F_{nv} = F_t \sin^2\alpha \qquad (6.2.79)$$

Where,

F_{nv} = horizontal component of breaking wave force in kN/m.

F_t = net breaking wave force acting on a vertical surface in kN/m.

α = vertical angle between nonvertical surface and the horizontal.

2.6.3.4.4 Breaking Wave Loads from Obliquely Incident Waves.

Breaking wave forces given by Equations 6.2.77 and 6.2.78 shall be modified in instances where waves are obliquely incident. Breaking wave forces from non-normally incident waves shall be given by

$$F_{oi} = F_t \sin^2\alpha \qquad (6.2.80)$$

Where,

F_{oi} = horizontal component of obliquely incident breaking wave force in kN/m.

F_t = net breaking wave force (normally incident waves) acting on a vertical surface in kN/m.

α = horizontal angle between the direction of wave approach and the vertical surface.

2.6.3.5 Impact loads

Impact loads are those that result from debris, ice, and any object transported by floodwaters striking against buildings and structures, or parts thereof. Impact loads shall be determined using a rational approach as concentrated loads acting horizontally at the most critical location at or below H_m (Eq. 6.2.69). Eq. 6.2.81 provides a rational approach for calculating the magnitude of the impact load.

$$F = \frac{\pi W V_b C_I C_O C_D C_B R_{max}}{2g\Delta t} \qquad (6.2.81)$$

Where,

F = impact force in N

W = debris weight in N, to be taken equal to 4448 N unless more specific data is available.

V_b = velocity of the debris, m/s, assumed equal to the velocity of water V defined in Sec. 2.6.3.2.2.

g = acceleration due to gravity, 9.81 m/s^2

Δt = duration of impact, which may be taken as 0.03 second

C_I = importance co-efficient = 0.6, 1.0, 1.2 or 1.3 for building occupancy categories I, II, III or IV respectively

C_O = orientation co-efficient = 0.8

C_D = depth co-efficient, to be taken equal to 0.0 for water depth 0.3m or less and equal to 1.0 for water depth 1.5m or more. Linear interpolation shall be made for intermediate water depth values.

C_B= blockage co-efficient, to be taken equal to 0.0 for upstream flow channel width 1.5m or less and equal to 1.0 for upstream flow channel width 9.1 m or more. Linear interpolation shall be made for intermediate values of upstream flow channel width. The upstream shall extend 30.0 m from the building.

R_{max} = maximum response ratio for impulsive load (half sine wave type) to be obtained from Table 6.2.27.

2.6.4 Temperature Effects

Temperature effects, if significant, shall be considered in the design of structures or components thereof in accordance with the provision of this Section. In determining the temperature effects on a structure, the following provisions shall be considered:

(a) The temperatures indicated, shall be the air temperature in the shade. The range of the variation in temperature for a building site shall be taken into consideration.

(b) Effects of the variation of temperature within the material of a structural element shall be accounted for by one of the following methods.

  (i) Relieve the stresses by providing adequate numbers of expansion or contraction joints,

  (ii) Design the structural element to sustain additional stresses due to temperature effects.

(c) when the method b(ii) above is considered to be applicable, the structural analysis shall take into account the following :

  (i) The variation in temperature within the material of the structural element, exposure condition of the element and the rate at which the material absorb or radiate heat.

  (ii) The warping or any other distortion caused due to temperature changes and temperature gradient in the structural element.

(d) When it can be demonstrated by established principle of mechanics or by any other means that neglecting some or all of the effects of temperature, does not affect the safety and serviceability of the structure, the temperature effect can be considered insignificant and need not be considered in design.

Figure 6.2.31 Normally incident breaking wave pressures against a vertical wall (space behind vertical wall is dry)

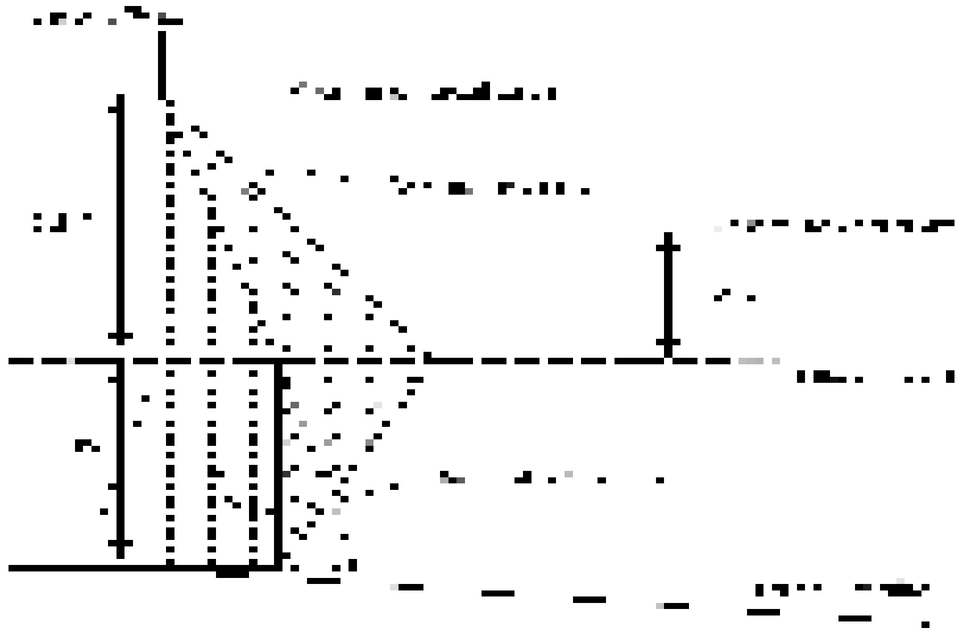


Figure 6.2.32 Normally incident breaking wave pressures against a vertical wall (still water level equal on both sides of wall)

Table 6.2.27: Values of response ratio, R_{max}, for impulsive loads

Ratio of impulse duration (Δt) to natural period (Sec. 2.5) of structure	R_{max}	Ratio of impulse duration (Δt) to natural period (Sec. 2.5) of structure	R_{max}
0	0	0.8	1.8
0.1	0.4	0.9	1.8
0.2	0.8	1	1.7
0.3	1.1	1.1	1.7
0.4	1.4	1.2	1.6
0.5	1.5	1.3	1.6
0.6	1.7	≥1.4	1.5
0.7	1.8		

2.6.5 Soil and Hydrostatic Pressure

For structures or portions thereof, lying below ground level, loads due to soil and hydrostatic pressure shall be determined in accordance with the provisions of this Section and applied in addition to all other applicable loads.

2.6.5.1 Pressure on basement wall:

In the design of basement walls and similar vertical or nearly vertical structures below grade, provision shall be made for the lateral pressure of adjacent soil. Allowance shall be made for possible surcharge due to fixed or moving loads. When a portion or the whole of the adjacent soil is below the surrounding water table, computations shall be based on the submerged unit weight of soil, plus full hydrostatic pressure.

2.6.5.2 Uplift on floors:

In the design of basement floors and similar horizontal or nearly horizontal construction below grade, the upward pressure of water, if any, shall be taken as the full hydrostatic pressure applied over the entire area. The hydrostatic head shall be measured from the underside of the construction.

2.6.6 Loads due to Explosions

Loads on buildings or portions thereof, shall be assessed in accordance with the provisions of this Section.

2.6.6.1 Explosion effects in closed rooms

(a) Determination of Loads and Response: Internal overpressure developed from an internal explosion such as that due to leaks in gas pipes, evaporation of volatile liquids, internal dust explosion etc., 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 :

   (i) The overpressure, q_O provided in Figure 6.2.33(a) shall be assumed to depend on a factor A_O/v, where, A_O is the total window area in m^2 and v is the volume in m^3 of the room considered,

   (ii) The internal pressure shall be assumed to act simultaneously upon all walls and floors in one closed room, and

   (iii) The action q_O obtained from Figure 6.2.33(a) may be taken as static action.

When a time dependent response is required, an impulsive force function similar to that shown in Figure 6.2.33(b) shall be used in a dynamic analysis, 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 shall be chosen in relation to the dynamic properties of the structures. However, the values shall be chosen within the intervals as given in Figure 6.2.33(b).

The pressure may be applied solely in one room or in more than one room 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 as ventilation areas even though certain limited structural parts break at pressures less than q_o.

(b) Limitations : Procedure for determining explosion loads given in (a) above shall have the following limitations:

(i) Values of q_o given in Figure 6.2.33(a) are based on tests with gas explosions in room corresponding to ordinary residential flats, and may be applied to considerably different conditions with caution after appropriate adjustment of the values based on more accurate information.

(ii) Figures 6.2.33(a) and 6.2.33(b) shall be taken as guides only, and probability of occurrence of an explosion shall be checked in each case using appropriate values.

Figure 6.2.33 Magnitude and distribution of internal pressure in a building due to internal gas explosion

2.6.6.2 Minimum design pressure

Walls, floors and roofs and their supporting members separating a use from an explosion exposure, shall be designed to sustain the anticipated maximum load effects resulting from such use including any dynamic effects, but for a minimum internal pressure or suction of 5 kN/m^2, in addition to all other loads specified in this Chapter.

2.6.6.3 Design pressure on relief vents

When pressure-relief vents are used, such vents shall be designed to relieve at a maximum internal pressure of 1.0 kN/m^2.

2.6.6.4 Loads due to other explosions

Loads arising from other types of explosions, such as those from external gas cloud explosions, external explosions due to high explosives (TNT) etc. shall be determined, for specific cases, by rational analyses based on information from reliable references or specialist advice shall be sought.

2.6.7 Vertical Forces on Air Raid Shelters

For the design of air raid shelters located in a building e.g. in the basement below ground level, the characteristic vertical load shall be determined in accordance with provisions of Sec 2.6.7.1 below.

2.6.7.1 Characteristic vertical loads

Buildings in which the individual floors are acted upon by a total distributed live load of up to 5.0 kN/m^2, vertical forces on air raid shelters generally located below ground level, such as a basement, shall be considered to have the characteristic values provided in Table 6.2.27. In the case of buildings having floors that are acted upon by a live load larger than 5.0 kN/m^2, above values shall be increased by the difference between the average live loads on all storeys above the one used as the shelter and 5.0 kN/m^2.

Table 6.2.28: Characteristic Vertical Loads for an Air Raid Shelter in a Building

No. of Storeys[1] above the Air Raid Shelter	Vertical Load, kN/m^2
$\leq$ 2	28
3 - 4	34
$\geq$ 4	41
Buildings of particularly stable construction irrespective of the number of storeys	28[2]

Notes:

[1] Storeys shall mean every usable storey above the shelter floor

[2] Buildings of particularly stable construction shall mean buildings having bearing structural elements made from reinforced in-situ concrete.

2.6.8 Loads on Helicopter Landing Areas

In addition to all other applicable loads provided in this Chapter, including the dead load, the minimum live load on helicopter landing or touch down areas shall be one of the loads L_1, L_2 or L_3 as given below producing the most unfavourable effect:

$$L_1 = W_1 \tag{6.2.82a}$$

$$L_2 = kW_2 \tag{6.2.82b}$$

$$L_3 = w \tag{6.2.82c}$$

Where,

W_1 = Actual weight of the helicopter in kN,

W_2 = Fully loaded weight of the helicopter in kN,

w = A distributed load of 5.0 kN/m^2,

k = 0.75 for helicopters equipped with hydraulic - type shock absorbers, and

= 1.5 for helicopters with rigid or skid-type landing gear.

The live load, L_1 shall be applied over the actual areas of contact of landing. The load, L_2 shall be a single concentrated load including impact applied over a 300 mm x 300 mm area. The loads L_1 and L_2 may be applied anywhere within the landing area to produce the most unfavourable effects of load.

2.6.9 Erection and Construction Loads

All loads required to be sustained by a structure or any portion thereof due to placing or storage of construction materials and erection equipment including those due to operation of such equipment shall be considered as erection loads. Provisions shall be made in design to account for all stresses due to such loads.

2.7 Combinations of Loads

2.7.1 General

Buildings, foundations and structural members shall be investigated for adequate strength to resist the most unfavorable effect resulting from the various combinations of loads provided in this Section. The combination of loads may be selected using the provisions of either Sec 2.7.2 or Sec 2.7.3 whichever is applicable. However, once Sec 2.7.2 or Sec 2.7.3 is selected for a particular construction material, it must be used exclusively for proportioning elements of that material throughout the structure.

In addition to the load combinations given in Sections 2.7.2 and 2.7.3 any other specific load combination provided elsewhere in this Code shall also be investigated to determine the most unfavourable effect.

The most unfavourable effect of loads may also occur when one or more of the contributing loads are absent, or act in the reverse direction. Loads such as F, H or S shall be considered in design when their effects are significant. Floor live loads shall not be considered where their inclusion results in lower stresses in the member under consideration. The most unfavourable effects from both wind and earthquake loads shall be considered where appropriate, but they need not be assumed to act simultaneously.

2.7.2 Combinations of Load effects for Allowable Stress/Strength Design Method

2.7.2.1 Basic combinations

Provisions of this Section shall apply to all construction materials permitting their use in proportioning structural members by allowable stress/strength design method. When this method is used in designing structural members, all loads listed herein shall be considered to act in the following combinations. The combination that produces the most unfavorable effect shall be used in design.

1. $D + F$
2. $D + H + F + L + T$
3. $D + H + F + (L_r \text{ or } R)$
4. $D + H + F + 0.75(L + T) + (L_r \text{ or } R)$
5. $D + H + F + (W \text{ or } 0.7E)$
6. $D + H + F + 0.75(W \text{ or } 0.7E) + 0.75L + 0.75(L_r \text{ or } R)$
7. $0.6D + W + H$
8. $0.6D + 0.7E + H$

When a structure is located in a flood zone or in tidal surge zone, the following load combinations shall be considered:

1. In Coastal Zones vulnerable to tidal surges, $1.5F_a$ shall be added to other loads in combinations (5), (6); E shall be set equal to zero in (5) and (6).
2. In non-coastal Zones, $0.75F_a$ shall be added to combinations (5), (6) and (7); E shall be set equal to zero in (5) and (6).

2.7.2.2 Stress increase

Unless permitted elsewhere in this Code, increases in allowable stress shall not be used with the loads or load combinations given above in Sec 2.7.2.1.

2.7.3 Combinations of Load effects for Strength Design Method

When strength design method is used, structural members and foundations shall be designed to have strength not less than that required to resist the most unfavorable effect of the combinations of factored loads listed in the following Sections:

2.7.3.1 Basic combinations

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \; or \; R)$
3. $1.2D + 1.6(L_r \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } R)$
5. $1.2D + 1.0E + 1.0L$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

Each relevant strength limit state shall be investigated. Effects of one or more loads not acting shall be investigated. The most unfavorable effect from both wind and earthquake loads shall be investigated, where appropriate, but they need not be considered to act simultaneously.

Exceptions:

1. The load factor on live load L in combinations (3), (4), and (5) is permitted to be reduced to 0.5 for all occupancies in which minimum specified uniformly distributed live load is less than or equal to 5.0 kN/m^2, with the exception of garages or areas occupied as places of public assembly.
2. The load factor on H shall be set equal to zero in combinations (6) and (7) if the structural action due to H counteracts that due to W or E. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.
3. For structures designed in accordance with the provisions of Chapter 6, Part 6 of this Code (reinforced concrete structures), where wind load W has not been reduced by a directionality factor, it shall be permitted to use $1.3W$ in place of $1.6W$ in (4) and (6) above.

When a structure is located in a flood zone or in tidal surge zone, the following load combinations shall be considered:

1. In Coastal Zones vulnerable to tidal surges, $1.6W$ shall be replaced by $1.6W$+$2.0F_a$ in combinations (4) and (6).
2. In Non-coastal Zones, $1.6W$ shall be replaced by $0.8W$+$1.0F_a$ in combinations (4) and (6).

2.7.4 Load Combinations for Extraordinary Events

Where required by the applicable Code, standard, or the authority having jurisdiction, strength and stability shall be checked to ensure that structures are capable of withstanding the effects of extraordinary (i.e., low-probability) events, such as fires, explosions, and vehicular impact.

2.7.5 Load Combination for Serviceability

Serviceability limit states of buildings and structures shall be checked for the load combinations set forth in this Section as well as mentioned elsewhere in this Code. For serviceability limit states involving visually objectionable deformations, repairable cracking or other damage to interior finishes, and other short term effects, the suggested load combinations for checking vertical deflection due to gravity load is

1. $D + L$

For serviceability limit states involving creep, settlement, or similar long-term or permanent effects, the suggested load combination is:

2. $D + 0.5L$

The dead load effect, D, used in applying combinations 1 and 2 above may be that portion of dead load that occurs following attachment of nonstructural elements. In applying combination 2 above to account for long term creep effect, the immediate (e.g. elastic) deflection may be multiplied by a creep factor ranging from 1.5 to 2.0. Serviceability against gravity loads (vertical deflections) shall be checked against the limits set forth in Sec 1.2.5 Chapter 1 of this Part as well as mentioned elsewhere in this Code.

For serviceability limit state against lateral deflection of buildings and structures due to wind effect, the following combination shall be used:

3. $D + 0.5L + 0.7W$

Due to its transient nature, wind load need not be considered in analyzing the effects of creep or other long-term actions. Serviceability against wind load using load combination 3 above shall be checked in accordance with the limit set forth in Sec 1.5.6.2 Chapter 1 of this Part.

2.8 List of Related Appendices

Appendix A	Equivalence of Nonhomogenous Equations in SI-Metric, MKS-Metric, and U.S. Customary Units
Appendix B	Local Geology, Tectonic Features and Earthquake Occurrence in the Region
Appendix C	Seismic Design Parameters for Alternative Method of Base Shear Calculation

PART VI
Chapter 3
Soils And Foundations

3.1 General

The Soils and Foundations Chapter of the Code is divided into the following three distinct Divisions:

Division A: Site Investigations, Soil Classifications, Materials and Foundation Types

Division B: Service Load Design Method of Foundations

Division C: Additional Considerations in Planning, Design and Construction of Building Foundations

Division A (Site Investigations, Soil Classifications, Materials and Foundation Types) consists of the following Sections:

- Site Investigations
- Identification, Classification and Description of Soils
- Materials
- Types of Foundation

Division B (Service Load Design Method of Foundations) has the sections as under:

- Shallow Foundations
- Geotechnical Design of Shallow Foundations
- Geotechnical Design of Deep Foundations
- Field Tests for Driven Piles and Drilled Shafts

Division C (Additional Considerations in Planning, Design and Construction of Building Foundations) deals with the following sections:

- Excavation
- Dewatering
- Slope Stability of Adjoining Building
- Fills
- Retaining Walls for Foundations
- Waterproofing and Damp-proofing
- Foundation on Slopes
- Foundations on Fill and Problematic Soils
- Foundation Design for Dynamic Forces
- Geo-hazards for Buildings

3.2 Scope

The provisions of this Chapter shall be applicable to the design and construction of foundations of buildings and structures for the safe support of dead and superimposed loads without exceeding the allowable bearing stresses, permissible settlements and design capability. Because of uncertainties and randomness involved in sub-soil characteristics, Geotechnical Engineering requires a high degree of engineering judgment. As such the Code provisions of this Chapter provided here under, are kept elaborative for better understanding of the readers. Provisions that are stated in imperative form using "shall" are mandatory. Other provisions of this Chapter should be followed using sound Geotechnical Engineering judgment.

3.3 Definitions and Symbols

3.3.1 Definitions

For the terms used in this Chapter, the following definitions shall apply.

ALLOWABLE BEARING CAPACITY	It is the minimum of the safe bearing capacity and safe settlement pressure, so that the foundation/ structure is safe and stable under both shear failure and settlement criteria. It may be denoted by symbol q_{allow}. The lateral dimensions of the foundation (width or diameter and the length) are designed on the basis of allowable bearing capacity. Also known as Allowable Bearing Pressure.
ALLOWABLE LOAD	The maximum load that may be safely applied to a foundation unit, considering both the strength and settlement of the soil, under expected loading and soil conditions.
ANGULAR DISTORTION	Angle between the horizontal and any two foundations or two points in a single foundation.
AUGUR PILE	Same as SCREW PILE.
BATTER PILE	The pile which is installed at an angle to the vertical in order to carry lateral loads along with the vertical loads. This is also known as RAKER PILE.
BEARING CAPACITY	The general term used to describe the load carrying capacity of foundation soil or rock in terms of average pressure that enables it to bear and transmit loads from a structure.

BEARING SURFACE	The contact surface between a foundation unit and the soil or rock upon which the foundation rests.
BORED PILE	A pile formed into a preformed hole of ground, usually of reinforced concrete having a diameter smaller than 600 mm.
BOULDER	Particles of rock that will not pass a 12 inch. (300 mm) square opening.
CAISSON	A deep foundation unit, relatively large section, sunk down (not driven) to the ground. This is also called WELL FOUNDATION.
CAST IN-SITU PILE	Same as BORED PILE.
CLAY	A natural aggregate of microscopic and submicroscopic mineral grains less than 0.002 mm in size and plastic in moderate to wide range of water contents.
CLAY MINERAL	A small group of minerals, commonly known as clay minerals, essentially composed of hydrous aluminium silicates with magnesium or iron replacing wholly or in part some of the aluminium.
CLAY SOIL	Same as CLAY.
COBBLE	Particles of rock that will pass a 12-in. (300-mm) square opening and be retained on a 3-in. (75-mm) sieve.
COLLAPSIBLE SOIL	Consists predominant of sand and silt size particles arranged in a loose honeycomb structure. These soils are dry and strong in their natural state and consolidate or collapse quickly if they become wet.
CONSOLIDATION SETTLEMENT	A time dependent settlement resulting from gradual reduction of volume of saturated soils because of squeezing out of water from the pores due to increase in effective stress and hence pore water pressure. It is also known as primary consolidation settlement. It is thus a time related process involving compression, stress transfer and water drainage.

DEEP FOUNDATION	A foundation unit that provides support for a structure transferring loads by end bearing and/or by shaft resistance at considerable depth below the ground. Generally, the depth is at least five times the least dimension of the foundation.
DESIGN BEARING CAPACITY	The maximum net average pressure applied to a soil or rock by a foundation unit that the foundation soil or rock will safely carry without the risk of both shear failure and exceedance of permissible settlement. It is equal to the least of the two values of net allowable bearing capacity and safe bearing pressure. This may also be called ALLOWABLE BEARING PRESSURE.
DESIGN LOAD	The expected un-factored load to a foundation unit.
DIFFERENTIAL SETTEMENT	The difference in the total settlements between two foundations or two points in the same foundation.
DISPERSIVE SOIL	Soils that are structurally unstable and disperse in water into basic particles i.e. sand, silt and clay. Dispersible soils tend to be highly erodible. Dispersive soils usually have a high Exchangeable Sodium Percentage (ESP).
DISPLACEMENT PILE	Same as DRIVEN PILE.
DISTORTION SETTLEMENT	Same as ELASTIC SETTLEMENT.
DOWNDRAG	The transfer of load (drag load) to a deep foundation, when soil settles in relation to the foundation. This is also known as NEGATIVE SKIN FRICTION.
DRILLED PIER	A deep foundation generally of large diameter shaft usually more than 600 mm and constructed by drilling and excavating into the soil.
DRILLED SHAFT	Same as DRILLED PIER.
DRIVEN PILE	A pile foundation pre-manufactured and placed in ground by driving, jacking, jetting or screwing.

EFFECTIVE STRESS	The pressure transmitted through grain to grain at the contact point through a soil mass is termed as effective stress or effective pressure.
ELASTIC SETTLEMENT	It is attributed due to lateral spreading or elastic deformation of dry, moist or saturated soil without a change in the water content and volume.
END BEARING	The load being transmitted to the toe of a deep foundation and resisted by the bearing capacity of the soil beneath the toe.
EXCAVATION	The space created by the removal of soil or rock for the purpose of construction.
EXPANSIVE SOIL	These are clay soils expand when they become wetted and contract when dried. These are formed of clay minerals like montmorillonite and illite.
FACTOR OF SAFETY	The ratio of ultimate capacity to design (working) capacity of the foundation unit.
FILL	Man-made deposits of natural earth materials (soil, rock) and/or waste materials.
FOOTING	A foundation constructed of masonry, concrete or other material under the base of a wall or one or more columns for the purpose of spreading the load over a larger area at shallower depth of ground surface.
FOUNDATION	Lower part of the structure which is in direct contact with the soil and transmits loads to the ground.
FOUNDATION ENGINEER	A graduate Engineer with at least five years of experience in civil engineering particularly in foundation design or construction.
GEOTECHNICAL ENGINEER	Engineer with Master’s degree in geotechnical engineering having at least 2 (two) years of experience in geotechnical design/construction or graduate in civil engineering/engineering geology having 10 (ten) years of experience in geotechnical design/construction.

GRAVEL	Particles of rock that will pass a 3-in. (75-mm) sieve and be retained on a No. 4 (4.75-mm) sieve.
GROSS PRESSURE	The total pressure at the base of a footing due to the weight of the superstructure and the original overburden pressure.
GROSS ALLOWABLE BEARING PRESSURE	The maximum gross average pressure of loading that the soil can safely carry with a factor of safety considering risk of shear failure. This may be calculated by dividing gross ultimate bearing capacity with a factor of safety.
GROUND WATER TABLE	The level of water at which porewater pressure is equal to atmospheric pressure. It is the top surface of a free body of water (piezometric water level) in the ground.
IMMEDIATE SETTLEMENT	This vertical compression occurs immediately after the application of loading either on account of elastic behaviour that produces distortion at constant volume and on account of compression of air void. For sands, even the consolidation component is immediate.
INORGANIC SOIL	Soil of mineral origin having small amount usually less than 5 percent of organic matter content.
LATERALLY LOADED PILE	A pile that is installed vertically to carry mainly the lateral loads.
MAT FOUNDATION	See RAFT.
NEGATIVE SKIN FRICTION	See DOWNDRAG.
NET PRESSURE	The gross pressure minus the surcharge pressure i.e. the overburden pressure of the soil at the foundation level.
NET SAFE BEARING CAPACITY	The maximum net pressure that can be safely applied from the foundation on the soil at its base, and at which the shear failure of the soil is avoided with a suitable factor of safety (FS). It is denoted by symbol q_{ns}. Thus, $q_{ns} = \frac{q_{nu}}{FS}$.

NET ULTIMATE BEARING CAPACITY	The minimum net pressure at the base of the foundation, excluding the weight of the overburden, at which the soil fails in shear due to the load on the foundation from superstructure. It is denoted by the symbol q_{nu}. Thus, $q_{nu} = q_{ult} - q'$ where, q' is the effective stress at foundation level due to overburden soil.
ORGANIC SOIL	Soil having appreciable/significant amount of organic matter content to influence the soil properties.
OVERCONSOLIDATION RATIO (OCR)	The ratio of the preconsolidation pressure (maximum past pressure) to the existing effective overburden pressure of the soil.
PEAT SOIL	An organic soil with high organic content, usually more than 75% by weight, composed primarily of vegetable tissue in various stages of decomposition usually with an organic odor, a dark brown to black color, a spongy consistency, and a texture ranging from fibrous to amorphous. Fully decomposed organic soils are known as MUCK.
PILE	A slender deep foundation unit made of materials such as steel, concrete, wood, or combination thereof that transmits the load to the ground by skin friction, end bearing and lateral soil resistance.
PILE CAP	A pile cap is a special footing needed to transmit the column load to a group or cluster of piles.
PILE HEAD	The upper small length of a pile. Also known as pile top.
PILE SHOE	A separate reinforcement or steel form attached to the bottom end (pile toe) of a pile to facilitate driving, to protect the pile toe, and/or to improve the toe resistance of the pile.
PILE TOE	The bottom end of a pile. Also known as pile tip.
PORE WATER PRESSURE	The pressure induced in the water or vapour and water filling the pores of soil. This is also known as neutral stress.

PRESUMPTIVE BEARING CAPACITY	The net approximate pressure prescribed as appropriate for the particular type of ground to be used in preliminary designs of foundations
RAFT	A relatively large spread foundation supporting an arrangement of columns or walls in a regular or irregular layout transmitting the loads to the soil by means of a continuous slab and/or beams, with or without depressions or openings. This is also known as MAT FOUNDATION.
RAKER PILE	See BATTER PILE.
ROCK	A natural aggregate of one or more minerals that are connected by strong and permanent cohesive forces.
ROTATION	It is the angle between the horizontal and any two foundations or two points in a single foundation.
RELATIVE ROTATION	Same as ANGULAR DISTORTION
REPLACEMENT PILE	Same as BORED PILE.
SAFE BEARING CAPACITY	It is the maximum gross pressure that can carry safely, without shear failure. It is denoted by symbol q_{safe}. Thus, $q_{safe} = q_{ns} + q'$. When the excavation for foundation is backfilled, $q_{safe} = q_{ns}$.
SAFE SETTLEMENT PRESSURE	The maximum pressure that can be applied from the foundation on the soil at its base such that the settlement of the foundation/structure is less than or equal to the permissible settlement. It may be denoted by symbol q_{sp}.
SAND	Aggregates of rounded, sub-rounded, angular, sub-angular or flat fragments of more or less unaltered rock or minerals which is larger than 75 μm and smaller than 4.75 mm in size.
SCREW PILE	A pre-manufactured pile consisting of steel helical blades and a shaft placed into ground by screwing.
SECONDARY CONSOLDATION SETTLEMENT	This is the settlement speculated to be due to the plastic deformation of the soil as a result of some complex colloidal-chemical processes or creep under imposed long term loading.

SERVICE LOAD	The expected un-factored load to a foundation unit.
SETTLEMENT	The downward vertical movement of foundation under load. When settlement occurs over a large area, it is sometimes called subsidence.
SHAFT RESISTANCE	The resistance mobilized on the shaft (side) of a deep foundation. Upward resistance is called positive shaft resistance. Downward force on the shaft is called negative shaft resistance.
SHALLOW FOUNDATION	A foundation unit that provides support for a structure transferring loads at a small depth below the ground. Generally, the depth is less than two times the least dimension of the foundation.
SILT	Soil passing a No. 200 (75-µm) sieve either non-plastic or plastic.
SOIL	A loose or soft deposit of particles of mineral and/or organic origin that can be separated by such gentle mechanical means as agitation in water.
SOIL PARTICLE SIZE	The sizes of particles that make up soil varying over a wide range. Soil particles are generally gravel, sand, silt and clay, though the terms boulder and cobble can be used to describe larger sizes of gravel.
TILT	Rotation of the entire superstructure or at least a well-defined part of it.
TOTAL SETTLEMENT	The total downward vertical displacement of a foundation base under load from its as-constructed position. It is the summation of immediate settlement, consolidation settlement and secondary consolidation settlement of the soil.
ULTIMATE BEARING CAPACITY	The minimum gross pressure at the base of the foundation at which the soil fails in shear due to the load on the foundation from superstructure. It is denoted by the symbol q_{ult} and obtained from bearing capacity equation containing soil/ground properties, depth of foundation, foundation dimensions and shapes, and loading conditions. Also known as Gross Ultimate Bearing Capacity.

3.3.2 Symbols and Notation

Every symbol used in this Chapter is explained where it first appears in the text. However, for convenience of the reader, a list of main symbols and notation is provided as under. Other common symbols and notation like those of soil classifications are not included in this list.

A	=	Cross sectional area of pile
A_b	=	End bearing area of pile
A_s	=	Skin friction area (perimeter area) of pile
B	=	Width of footing/foundation (Sec 3.9.6, Sec 3.20.2)
B	=	Smallest dimension of pile group (Sec 3.10.5)
B_p	=	Width of plate
B_r	=	Reference width (300 mm) for computation of pile settlement
CEC	=	Cation exchange capacity
CRR	=	Cyclic resistance ratio
CSR	=	Cyclic stress ratio
C_c	=	Compression index of soil
C_p	=	Empirical coefficient used for pile settlement computation
C_u	=	Uniformity coefficient
C_z	=	Coefficient of curvature
D	=	Diameter or width of pile
D_b	=	Diameter of pile at base
D_c	=	Critical depth of soil layer
D_{10}	=	Effective grain size; the size of soil particle from which 10 percent of the soil is finer
D_{30}	=	The size of soil particle from which 30 percent of the soil is finer
D_{60}	=	The size of soil particle from which 60 percent of the soil is finer
EI	=	Flexural rigidity of footing
Em_aP	=	Exchangeable magnesium percentage
E_p	=	Modulus of elasticity of pile material
E_s	=	Modulus of elasticity of soil
ESP	=	Exchangeable sodium percentage

F_L	=	Factor of safety against liquefaction
FS	=	Factor of safety
G	=	Modulus of rigidity
H	=	Height of wall from foundation footing (Sec 3.9.4)
H	=	Layer thickness (Sec 3.10.5)
H	=	Thickness of sample (Sec 3.5.6)
H'	=	Final thickness of sample (Sec 3.5.6)
I_p	=	Plasticity index
I_{subs}	=	Relative subsidence
K	=	Coefficient of earth pressure
K_o	=	Coefficient of earth pressure at rest
L	=	Length of pile (Sec 3.10)
L	=	Length of deflected part of wall/raft or centre to centre distance between columns. (Sec 3.9.4)
LL		Liquid limit
N	=	Standard penetration test value (SPT)
N_{60}	=	Corrected SPT value for field procedures
$\bar{N}_{60}$	=	Average SPT N_{60} value
$(N_1)_{60}$	=	Corrected SPT value for overburden pressure (for sandy soil)
N_c, N_q, N_γ	=	Bearing capacity factors
OCR	=	Overconsolidation ratio
PI	=	Plasticity index; same as I_p
Q_{allow}	=	Allowable load
Q_p	=	End bearing at the base or tip of the pile
Q_p	=	Load transferred to the soil at pile tip level
Q_s	=	Skin friction or shaft friction or side shear
Q_{ult}	=	Ultimate bearing/load carrying capacity
R_s	=	Group settlement ratio of pile group
S_{ax}	=	Settlement due to axial deformation
S_g	=	Settlement of pile group
S_{pt}	=	Settlement at pile tip

S_{sf}	=	Settlement of pile due to skin friction
S_r	=	Degree of saturation
$S_{t(sinale)}$	=	Total settlement of a single pile
W	=	Weight of the pile
WPI	=	Weighted plasticity index
a_{max}	=	Peak horizontal acceleration on the ground surface
c	=	Apparent cohesion of soil
c_u	=	Undrained cohesion of soil
d_p	=	Diameter of pile
e	=	Void ratio
e_c	=	Critical void ratio
e_L	=	Void ratio at liquid limit
e_P	=	Void ratio at plastic limit
e_i	=	Initial void ratio
e_o	=	Initial void ratio; same as e_i
f_b	=	End bearing resistance on unit tip area of pile
f_n	=	Natural frequency
f_s	=	Skin frictional resistance on unit surface area of pile
f_s	=	Adhesive stress (Sec. 3.10.1.12)
g	=	Gravitational acceleration
k	=	Modulus of sub-grade reaction
k_p	=	Stiffness of soil
k_s	=	Coefficient of horizontal soil stress
m	=	Total mass of machine foundation system
m_f	=	Mass of foundation block
m_s	=	Mass of soil
n	=	Number of pile in a group
q_{allow}	=	Allowable bearing capacity of shallow foundation
q_o	=	Ultimate end bearing capacity pile
q_{ns}	=	Net safe ultimate bearing capacity of shallow foundation
q_{nu}	=	Net ultimate bearing capacity of shallow foundation

q_{safe}	=	Safe ultimate bearing capacity
q_{sp}	=	Safe settlement pressure of shallow foundation
q_u	=	Unconfined compressive strength
q_{ult}	=	Ultimate bearing capacity of shallow foundation
r_d	=	Stress reduction coefficient to allow for the deformability of the soil column
s_u	=	undrained shear strength; same as c_u
w_L	=	Liquid limit; same as LL
z	=	Depth
Δz_i	=	Thickness of any (i^{th}) layer
α	=	Adhesion factor
β	=	Ratio of footing length to width (Sec 3.9.6.8)
β	=	Friction factor due to overburden (3.10.1)
γ, γ_t	=	Unit weight of the soil
γ_w	=	Unit weight of water
δ	=	Total settlement
δ_c	=	Consolidation settlement
δ_e	=	Elastic settlement
δ_i	=	Immediate settlement
δ_s	=	Secondary consolidation settlement
μ	=	Poisson's ratio of soil
$\sigma_o^{'}$	=	Initial effective stress at mid-point of a soil layer
$\sigma_p^{'}$	=	Increase in effective stress at mid-point of a soil layer due to increase in stress
$\sigma_r^{'}$	=	Reference stress (100 kPa) for computation of pile settlement
σ_v	=	The total vertical stress
$\sigma_v^{'}$	=	Effective vertical stress
$\sigma_z^{'}$	=	Effective vertical stress; same as $\sigma_v^{'}$
τ_{max}	=	Maximum shear stress
ϕ	=	Apparent angle of internal fiction
$\phi^{'}$	=	Effective/drained angle of internal fiction
ϕ_s	=	Soil shaft interface friction angle
ω_n	=	natural circular frequency

Division A: Site Investigations, Soil Classifications, Materials and Foundation Types (Sections 3.4 to 3.7)

3.4 Site Investigations

3.4.1 Sub-Surface Survey

Depending on the type of project thorough investigations has to be carried out for identification, location, alignment and depth of various utilities, e.g., pipelines, cables, sewerage lines, water mains etc. below the surface of existing ground level. Detailed survey may also be conducted to ascertain the topography of existing ground.

3.4.2 Sub-Soil Investigations

Sub soil investigation shall be done describing the character, nature, load bearing capacity and settlement capacity of the soil before constructing a new building and structure or for alteration of the foundation of an existing structure. The aims of a geotechnical investigation are to establish the soil, rock and groundwater conditions, to determine the properties of the soil and rock, and to gather additional relevant knowledge about the site. Careful collection, recording and interpretation of geotechnical information shall be made. This information shall include ground conditions, geology, geomorphology, seismicity and hydrology, as relevant. Indications of the variability of the ground shall be taken into account.

An engineering geological study may be an important consideration to establish the physiographic setting and stratigraphic sequences of soil strata of the area. Geological and agricultural soil maps of the area may give valuable information of site conditions.

During the various phases of sub-soil investigations, e.g. drilling of boreholes, field tests, sampling, groundwater measurements, etc. a competent graduate engineer having experiences in supervising sub-soil exploration works shall be employed by the drilling contractor.

3.4.3 Methods of Exploration

Sub soil exploration process may be grouped into three types of activities such as: reconnaissance, exploration and detailed investigations. The reconnaissance method includes geophysical measurements, sounding or probing, while exploratory methods involve various drilling techniques. Field investigations should comprise :

(i) Drilling and/or excavations (test pits including exploratory boreholes) for sampling;

(ii) Groundwater measurements;

(iii) Field tests.

Examples of the various types of field investigations are:

(i) Field testing (e.g. CPT, SPT, dynamic probing, WST, pressuremeter tests, dilatometer tests, plate load tests, field vane tests and permeability tests);

(ii) Soil sampling for description of the soil and laboratory tests;

(iii) Groundwater measurements to determine the groundwater table or the pore pressure profile and their fluctuations

(iv) Geophysical investigations (e.g. seismic profiling, ground penetrating radar, resistivity measurements and down hole logging);

(v) Large scale tests, for example to determine the bearing capacity or the behaviour directly on prototype elements, such as anchors.

Where ground contamination or soil gas is expected, information shall be gathered from the relevant sources. This information shall be taken into account when planning the ground investigation. Some of the common methods of exploration, sampling and ground water measurements in soils are described in Appendix D.

3.4.4 Number and Location of Investigation Points

The locations of investigation points, e.g., pits and boreholes shall be selected on the basis of the preliminary investigations as a function of the geological conditions, the dimensions of the structure and the engineering problems involved. When selecting the locations of investigation points, the following should be observed:

(i) The investigation points should be arranged in such a pattern that the stratification can be assessed across the site;

(ii) The investigation points for a building or structure should be placed at critical points relative to the shape, structural behaviour and expected load distribution (e.g. at the corners of the foundation area);

(iii) For linear structures, investigation points should be arranged at adequate offsets to the centre line, depending on the overall width of the structure, such as an embankment footprint or a cutting;

(iv) For structures on or near slopes and steps in the terrain (including excavations), investigation points should also be arranged outside the project area, these being located so that the stability of the slope or cut can be assessed. Where anchorages are installed, due consideration should be given to the likely stresses in their load transfer zone;

(v) The investigation points should be arranged so that they do not present a hazard to the structure, the construction work, or the surroundings (e.g. as a result of the changes they may cause to the ground and groundwater conditions);

(vi) The area considered in the design investigations should extend into the neighbouring area to a distance where no harmful influence on the neighbouring area is expected. Where ground conditions are relatively uniform or the ground is known to have sufficient strength and stiffness properties, wider spacing or fewer investigation points may be applied. In either case, this choice should be justified by local experience.

(vii) The locations and spacing of sounding, pits and boreholes shall be such that the soil profiles obtained will permit a reasonably accurate estimate of the extent and character of the intervening soil or rock masses and will disclose important irregularities in subsurface conditions.

(viii) For building structures, the following guidelines shall be followed:

On uniform soils, at least three borings, not in one line, should be made for small buildings and at least five borings one at each corner and one at the middle should be made for large buildings. As far as possible the boreholes should be drilled closed to the proposed foundations but outside their outlines.

Spacing of exploration depends upon nature and condition of soil, nature and size of the project. In uniform soil, spacing of exploration (boring) may be 30 m to 100 m apart or more and in very erratic soil conditions, spacing of 10 m or less may be required. The following chart gives an approximate idea about spacing of boring required for small and multistoried buildings having different horizontal stratification of soil.

Type of Building	**Spacing of Bore Holes (m)**		
	Type of Soil in Horizontal Stratification		
	Uniform	**Average**	**Erratic**
Small buildings	60	30	15
Multistoried buildings	45	30	15

(ix) For large areas covering industrial and residential colonies, the geological nature of the terrain will help in deciding the number of boreholes or trial pits. The whole area may be divided into grid pattern with Cone Penetration Tests (Appendix D) performed at every 100 m grid points. The number of boreholes or trial pits shall be decided by examining the variation in penetration curves. At least 67% of the required number of borings or trial pits shall be located within the area under the building.

3.4.5 Depth of Exploration

The depth of investigations shall be extended to all strata that will affect the project or are affected by the construction. The depth of exploration shall depend to some extent on the site and type of the proposed structure, and on certain design considerations such as safety against foundation failure, excessive settlement, seepage and earth pressure. Cognizance shall be taken of the character and sequence of the subsurface strata. The site investigation should be carried to such a depth that the entire zone of soil or rock affected by the changes caused by the building or the construction will be adequately explored. A rule of thumb used for this purpose is to extend the borings to a depth where the additional load resulting from the proposed building is less than 10% of the average load of the structure, or less than 5% of the effective stress in the soil at that depth. Where the depth of investigation cannot be related to background information, the following guide lines are suggested to determine the depth of exploration:

(i) Where substructure units will be supported on spread footings, the minimum depth boring should extend below the anticipated bearing level a minimum of two footing widths for isolated, individual footings where length $\leq$ 2 times of width, and four footing widths for footings where length > 5 times of width. For intermediate footing lengths, the minimum depth of boring may be estimated by linear interpolation as a function of length between depths of two times width and five times width below the bearing level. Greater depth may be required where warranted by local conditions.

(ii) For more heavily loaded structures, such as multistoried structures and for framed structures, at least 50% of the borings should be extended to a depth equal to 1.5 times the width of the building below the lowest part of the foundation.

(iii) Normally the depth of exploration shall be 1.5 times the estimated width or the least dimension of the footing below the foundation level. If the pressure bulbs for a number of loaded areas overlap, the whole area may be considered as loaded and exploration shall be carried down to one and a half times the least dimension. In weak soils, the exploration shall be continued to a depth at which the loads can be carried by the stratum in question without undesirable settlement or shear failure.

(iv) Where substructure units will be supported on deep foundations, the depth boring should extend a minimum of 6 m below the anticipated pile of shaft tip elevation. Where pile or shaft groups will be used, the boring should extend at least two times the maximum pile or shaft group dimension below the anticipated tip elevation, unless the foundation will be end bearing on or in rock.

(v) For piles bearing on rock, a minimum of 1.5 m of rock core should be obtained at each boring location to ensure the boring has not been terminated in a boulder.

(vi) For shafts supported on or extending into rock, a minimum of 1.5 m of rock core, or a length of rock core equal to at least three times the shaft diameter for isolated shafts or two times the maximum shaft group dimension for a shaft group, whichever is greater, should be obtained to ensure that the boring had not been terminated in a boulder and to determine the physical properties of rock within the zone of foundation influence for design.

(vii) The depth, to which weathering process affects the deposit, shall be regarded as the minimum depth of exploration for a site. However, in no case shall this depth be less than 2 m, but where industrial processes affect the soil characteristics, this depth may be more.

(viii) At least one boring should be carried out to bedrock, or to well below the anticipated level of influence of the building. Bedrock should be ascertained by coring into it to a minimum depth of 3 m.

3.4.6 Sounding and Penetration Tests

Subsurface soundings are used for exploring soil strata of an erratic nature. They are useful to determine the presence of any soft pockets between drill holes and also to determine the density index of cohesionless soils and the consistency of cohesive soils at desired depths. A field test called Vane Shear Test may be used to determine the shearing strength of the soil located at a depth below the ground.

Penetration tests consist of driving or pushing a standard sampling tube or a cone. The devices are also termed as penetrometers, since they penetrate the subsoil with a view to measuring the resistance to penetrate the soil strata. If a sampling tube is used to penetrate the soil, the test is referred to as Standard Penetration Test (or simply SPT). If a cone is used, the test is called a Cone Penetration Test. If the penetrometer is pushed steadily into the soil, the procedure is known as Static Penetration Test. If driven into the soil, it is known as Dynamic Penetration Test. Details of sounding and penetrations tests are presented in Appendix D.

3.4.7 Geotechnical Investigation Report

The results of a geotechnical investigation shall be compiled in the Geotechnical Investigation Report which shall form a part of the Geotechnical Design Report. The Geotechnical Investigation Report shall consist of the following :

(i) A presentation of all appropriate geotechnical information on field and laboratory tests including geological features and relevant data;

(ii) A geotechnical evaluation of the information, stating the assumptions made in the interpretation of the test results.

The Geotechnical Investigation Report shall state known limitations of the results, if appropriate. The Geotechnical Investigation Report should propose necessary further field and laboratory investigations, with comments justifying the need for this further work. Such proposals should be accompanied by a detailed programme for the further investigations to be carried out. The presentation of geotechnical information shall include a factual account of all field and laboratory investigations. The factual account should include the following information :

(i) The purpose and scope of the geotechnical investigation including a description of the site and its topography, of the planned structure and the stage of the planning the account is referring to;

(ii) The names of all consultants and contractors;

(iii) The dates between which field and laboratory investigations were performed;

(iv) The field reconnaissance of the site of the project and the surrounding area noting particularly :

- evidence of groundwater;
- behaviour of neighbouring structures;
- exposures in quarries and borrow areas;

- areas of instability;
- difficulties during excavation;
- history of the site;
- geology of the site,
- survey data with plans showing the structure and the location of all investigation points;
- local experience in the area;
- information on the seismicity of the area.

The presentation of geotechnical information shall also include documentation of the methods, procedures and results including all relevant reports of :

(i) desk studies;

(ii) field investigations, such as sampling, field tests, groundwater measurements and technical specifications of field equipment used

(iii) laboratory tests and test standard followed

The results of the field and laboratory investigations shall be presented and reported according to the requirements defined in the ASTM or equivalent standards applied in the investigations.

3.5 Identification, Classification and Description af Soils

3.5.1 Identification of Soils

Samples and trial pits should be inspected visually and compared with field logs of the drillings so that the preliminary ground profile can be established. For soil samples, the visual inspection should be supported by simple manual tests to identify the soil and to give a first impression of its consistency and mechanical behaviour. A standard visual-manual procedure of describing and identifying soils may be followed.

Soil classification tests should be performed to determine the composition and index properties of each stratum. The samples for the classification tests should be selected in such a way that the tests are approximately equally distributed over the complete area and the full depth of the strata relevant for design.

3.5.2 Particle Size Classification of Soils

Depending on particle sizes, main soil types are gravel, sand, silt and clay. However, the larger gravels can be further classified as cobble and boulder. The soil particle size shall be classified in accordance with Table 6.3.1.

Table 6.3.1: Particle Size Ranges of Soils

Soil Type		Particle Size Range (mm)	Retained on Mesh Size/ Sieve No.
Boulder		> 300	12″
Cobble		300 – 75	3″
Gravel:	Coarse Gravel	75 – 19	3/4″
	Medium Gravel	19 – 9.5	3/8″
	Fine Gravel	9.5 – 4.75	No. 4
Sand:	Coarse Sand	4.75 – 2.00	No. 10
	Medium Sand	2.00 – 0.425	No. 40
	Fine Sand	0.425 – 0.075	No. 200
Silt		0.075 – 0.002	---
Clay		< 0.002	---

3.5.3 Engineering Classification of Soils

Soils are divided into three major groups, coarse grained, fine grained and organic. The classification is based on classification test results namely grain size analysis and consistency test. The coarse grained soils shall be classified using Table 6.3.2. Outlines of organic and inorganic soil separations are also provided in Table 6.3.2. The fine grained soils shall be classified using the plasticity chart shown in Figure 6.3.1. In this context, this Code adopts the provisions of ASTM D2487. In addition to these classifications, a soil shall be described by its colour, particle angularity (for coarse grained soils) and consistency. Further to the above classification soils exhibiting swelling or collapsing characteristic shall be recorded. For undisturbed soils information on stratification, compactness, cementation, moisture conditions and drainage characteristics shall be included.

Table 6.3.2: Engineering Classification of Soils (Criteria for Assigning Group Symbols and Names using Laboratory Tests ^{A})

Classification (For particles smaller than 75 mm and based on estimated weights)			Group Symbol	Group Name ^{B}	Laboratory Classification		
					Percent finer than 0.075 mm	Other Criteria	
Coarse grained soils (More than 50% of the material retained on No. 200 sieve (0.075 mm))	**Gravels** (More than 50%of coarse fraction retained on No. 4 sieve (4.75 mm))	Clean gravels	GW	Well graded gravels, sandy gravels, sand gravel mixture, little or no fines.^{D}	$< 5^{E}$	$C_u \geq 4$ and $1 \leq C_z \leq 3^{C}$	
			GP	Poorly graded gravels, sandy gravels, Sand gravel mixture, little or no fines. ^{D}		$C_u < 4$ and/or $1 > C_z > 3^{C}$	
		Gravel with fines	GM	Silty gravels, silty sandy gravels. $^{D, F, G}$	$> 12^{E}$	Ip< 4 or the limit values below 'A' line of plasticity chart	For 4> Ip > 7 and limit values above 'A' line, dual symbol required*
			GC	Clayey gravels, silty clayey gravels. . $^{D, F, G}$		Ip >7 and the limit values above 'A' line of Plasticity Chart	
	Sands (over 50% of coarse fraction smaller than 4.75 mm)	Clean Sands	SW	Well graded sand, gravelly sand, little or no fines. ^{H}	$< 5^{E}$	$C_u \geq 6$ and $1 \leq Cz \leq 3^{C}$	
			SP	Poorly graded sands, gravelly sand, little or no fines. ^{H}		$C_u < 6$ and/or $1 > C_z > 3^{C}$	
		Sands with fines	SM	Silty sand, poorly graded sand silt mixtures. $^{F, G, H}$	$> 12^{E}$	Ip < 4 or the limit values below 'A' line of Plasticity chart	For 4 > Ip >7 and limit values above A-line, dual symbols required.
			SC	Clayey sand, sand clay mixtures. $^{F, G, H}$		Ip >7 and the limit values above 'A' line of plasticity chart	
Fine grained soils (Over 50% of the material smaller than 0.075 mm)	**Silts & Clays** $w_L < 50$	Inorganic	ML	Silt of low to medium compressibility, very fine sands, rock flour, silt with sand. $^{K, L, M}$	Limit values on or below 'A' line of plasticity chart & Ip <4		
			CL	Clays of low to medium plasticity, gravelly clay, sandy clay, silty clay, lean clay. $^{K, L, M}$	Limit values above 'A' line of plasticity chart and/or Ip > 4		
		Organic	OL	Organic clay $^{K, L, M, N}$ and Organic silt $^{K, L, M, O}$ of low to medium plasticity	$\frac{\text{Liquid limit (oven dried)}}{\text{Liquid limit (undried)}} < 0.75$		

Classification (For particles smaller than 75 mm and based on estimated weights)			**Group Symbol**	**Group Name** [B]	**Laboratory Classification**	
					Percent finer than 0.075 mm	**Other Criteria**
Fine grained soils (Over 50% of the material smaller than 0.075 mm)	**Silts & Clays** $w_L \geq 50$	Inorganic	MH	Silt of high plasticity, micaceous fine sandy or silty soil, elastic silt. [K, L, M]	Limit values on or below 'A' line of plasticity chart	
			CH	High plastic clay, fat clay. [K, L, M]	Limit values above 'A' line of plasticity chart	
		Organic	OH	Organic clay of high plasticity. [K, L, M, P]	$\frac{\text{Liquid limit (oven dried)}}{\text{Liquid limit (undried)}} < 0.75$	
Soils of high organic origin			PT	Peat and highly organic soils. [K, L, M, Q]	Identified by colour, odour, fibrous texture and spongy characteristics.	

Notes:

[A] Based on the material passing the 3-in. (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add “with cobbles or boulders, or both” to group name.

[C] $C_u = D_{60}/D_{10}$, $C_Z = (D_{30})^2 / (D_{10} \times D_{60})$

[D] If soil contains ≥ 15 % sand, add “with sand” to group name.

[E] Gravels with 5 to 12 % fines require dual symbols:

GW-GM well-graded gravel with silt

GW-GC well-graded gravel with clay

GP-GM poorly graded gravel with silt

GP-GC poorly graded gravel with clay

[F] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[G] If fines are organic, add “with organic fines” to group name.

[H] If soil contains ≥ 15 % gravel, add “with gravel” to group name.

[I] Sands with 5 to 12 % fines require dual symbols:

SW-SM well-graded sand with silt

SW-SC well-graded sand with clay

SP-SM poorly graded sand with silt

SP-SC poorly graded sand with clay.

[J] If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29 % plus No. 200, add “with sand” or “with gravel,” whichever is predominant.

[L] If soil contains ≥30 % plus No. 200, predominantly sand, add “sand ” to group name.

[M] If soil contains ≥ 30 % plus No. 200, predominantly gravel, add “gravelly” to group name.

[N] PI ≥ 4 and plots on or above “A” line.

[O] PI < 4 or plots below“ A” line.

[P] PI plots on or above “A” line.

[Q] PI plots below “A” line.

If desired, the percentages of gravel, sand, and fines may be stated in terms indicating a range of percentages, as follows:

Trace –	Particles are present but estimated to be less than 5 %
Few –	5 to 10 %
Little –	15 to 25 %
Some –	30 to 45 %
Mostly –	50 to 100 %

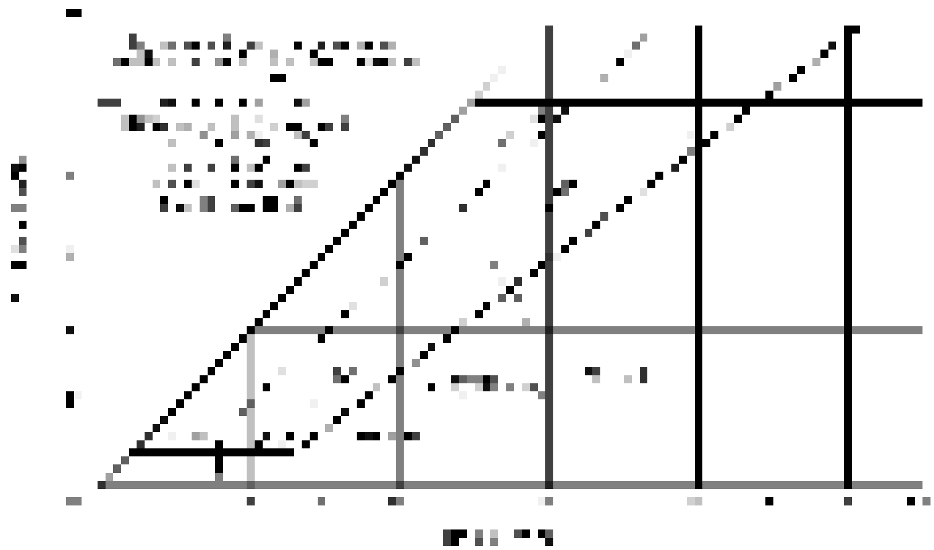


Figure 6.3.1 Plasticity chart (based on materials passing 425 μm sieve)

3.5.4 Identification and Classification of Organic Soils

The presence of organic matter can have undesirable effects on the engineering behaviour of soil. For example, the bearing capacity is reduced, the compressibility is increased and, swelling and shrinkage potential is increased due to organic content. Organic content tests are used to classify the soil. In soil with little or no clay particles and carbonate content, the organic content is often determined from the loss on ignition at a controlled temperature. Other suitable tests can also be used. For example, organic content can be determined from the mass loss on treatment with hydrogen peroxide (H_2O_2), which provides a more specific measure of organics. Organic deposits are due to decomposition of organic matters and found usually in topsoil and marshy place. A soil deposit in organic origin is said to peat if it is at the higher end of the organic content scale (75% or more), organic soil at the low end, and muck in between. Peat soil is usually formed of fossilized plant minerals and characterized by fiber content and lower decomposition. The peats have certain characteristics that set them apart from moist mineral soils and required special considerations for construction over them. This special characteristic includes, extremely high natural moisture content, high compressibility including significant secondary and even tertiary compression and very low undrained shear strength at natural moisture content.

However, there are many other criteria existed to classify the organic deposits and it remains still as controversial issue with numerous approaches available for varying purpose of classification. A possible approach is being considered by the American society for Testing and Materials for classifying organic soils having varying amount of organic matter contents. The classification is given in Table 6.3.3.

Table 6.3.3: Classification and Description of Organic Soils (after Edil, 1997)

Organic Content (ASTM D2974-07a)	Description
< 5 %	Little effect on behavior; considered inorganic soil.
6 ~ 20 %	Effects properties but behavior is still like mineral soils; organic silts and clays.
21 ~ 74 %	Organic matter governs properties; traditional soil mechanics may be applicable; silty or clayey organic soils.
> 75 %	Displays behavior distinct from traditional soil mechanics especially at low stress.

3.5.5 Identification and Classification of Expansive Soils

Expansive soils are those which swell considerably on absorption of water and shrink on the removal of water. In monsoon seasons, expansive soils imbibe water, become soft and swell. In drier seasons, these soils shrink or reduce in volume due to evaporation of water and become harder. As such, the seasonal moisture variation in such soil deposits around and beneath the structure results into subsequent upward and downward movements of structures leading to structural damage, in the form of wide cracks in the wall and distortion of floors. For identification and classification of expansive soils parameters like liquid limit, plasticity index, shrinkage limit, free swell, free swell index, linear shrinkage, swelling potential, swelling pressure and volume change from air dry to saturate condition should be evaluated experimentally or from available geotechnical correlation. Various recommended criteria for identification and classification of expansive soils are presented in Appendix E.

3.5.6 Identification and Classification of Collapsible Soils

Soil deposits most likely to collapse are; (i) loose fills, (ii) altered wind-blown sands, (iii) hill wash of loose consistency and (iv) decomposed granite or other acid igneous rocks.

A very simple test for recognizing collapsible soil is the ″sauges test″. Two undisturbed cylindrical samples (sausages) of the same diameter and length (volume) are carved from the soil. One sample is then wetted and kneaded to form a cylinder of the original diameter. A decrease in length as compared to the original, undisturbed cylinder will confirm a collapsible grain structure. Collapse is probable when the natural void ratio, collapsible grain structure. Collapse is probable when the natural void ratio, e_i is higher than a critical void ratio, e_c that depends on void ratios e_L and e_P at liquid limit and plastic limits respectively. The following formula should be used to estimate the critical void ratio.

$$e_c = 0.85\,e_L + 0.15\,e_P \qquad (6.3.1)$$

Collapsible soils (with a degree of saturation, $S_r \leq 0.6$) should satisfy the following condition:

$$\frac{e_L - e_i}{1 + e_i} \leq 0.10 \qquad (6.3.2)$$

A consolidation test is to be performed on an undisturbed specimen at natural moisture content and to record the thickness, “H” on consolidation under a pressure “p” equal to overburden pressure plus the external pressure likely to be exerted on the soil. The specimen is then submerged under the same pressure and the final thickness H’ recorded. Relative subsidence, I_{subs} is found as:

$$I_{subs} = \frac{H - H'}{H} \qquad (6.3.3)$$

Soils having $I_{subs} \geq 0.02$ are considered to be collapsible.

3.5.7 Identification and Classification of Dispersive Soils

Dispersive nature of a soil is a measure of erosion. Dispersive soil is due to the dispersed structure of a soil matrix. An identification of dispersive soils can be made on the basis of pinhole test.

The pinhole test was developed to directly measure dispersive potential of compacted fine grained soils in which water is made to flow through a small hole in a soil specimen, where water flow through the pinhole simulates water flow through a crack or other concentrated leakage channel in the impervious core of a dam or other structure. The test is run under 50, 180, 380 and 1020 mm heads and the soil is classified as follows in Table 6.3.4.

Table 6.3.4: Classification of Dispersive Soil on the Basis of Pinhole Test (Sherard et. al. 1976)

Test Observation	Type of Soil	Class of Soil
Fails rapidly under 50 mm head.	Dispersive soils	D_1 and D_2
Erode slowly under 50 mm or 180 mm head	Intermediate soils	ND_4 and ND_3
No colloidal erosion under 380 mm or 1020 mm head	Non-dispersive soils	ND_2 and ND_1

Another method of identification is to first determine the pH of a 1:2.5 soil/water suspension. If the pH is above 7.8, the soil may contain enough sodium to disperse the mass. Then determine: (i) total excahangable bases, that is, K^{+}, Ca^{2+}, $Mg^{2+}$and Na_{+} (milliequivalent per 100g of air dried soil) and (ii) cation exchange capacity (CEC) of soil (milliequivalent per 100g of air dried soil). The Exchangeable Sodium Percentage ESP is calculated from the relation:

$$ESP = \frac{N_a}{CEC} \times 100(\%) \qquad (6.3.4)$$

EM_gP is given by:

$$EM_gP = \frac{Mg}{CEC} \times 100(\%) \qquad (6.3.5)$$

If the ESP is above 8 percent and ESP plus EM_gP is above 15, dispersion will take place. The soils with ESP =7 to 10 are moderately dispersive in combination with reservoir waters of low dissolved salts. Soils with ESP greater than 15 have serious piping potential. Dispersive soils do not actually present any problems with building structures. However, dispersive soil can lead to catastrophic failures of earth embankment dams as well as severe distress of road embankments.

3.5.8 Identification and Classification of Soft Inorganic Soils

No standard definition exists for soft clays in terms of conventional soil parameters, mineralogy or geological origin. It is, however, commonly understood that soft clays give shear strength, compressibility and severe time related settlement problems. In near surface clays, where form a crust, partial saturation and overconsolidation occur together and the overconsolidation is a result of the drying out of the clay due to changes in water table.

In below surface clays, overconsolidation may have taken place when the clay was previously at, or close to the ground surface and above the water table, but due to subsequent deposition the strata may now be below the surface, saturated and overconsolidated. Partial saturation does not in itself cause engineering problems, but may lead to laboratory testing difficulties. Soft clays have undrained shear strengths between about 10kPa and 40kPa, in other words, from exuding between the fingers when squeezed to being easily moulded in the fingers.

Soft clays present very special problems of engineering design and construction. Foundation failures in soft clays are comparatively common. The construction of buildings in soft clays has always been associated with stability problems and settlement. Shallow foundations inevitably results in large settlements which must be accommodated for in the design, and which invariably necessitate long-term maintenance of engineered facilities. The following relationship among N-values obtained from SPT, consistency and undrained shear strength of soft clays may be used as guides.

N-value	Consistency	Undrained Shear Strength (kN/m^2)
Below 2	Very soft	Less than 20
2 – 4	Soft	20 – 40

Undrained shear strength is half of unconfined compressive strength as determined from unconfined compression test or half of the peak deviator stress as obtained from unconsolidated undrained (UU) triaxial compression test.

3.6 Materials

All materials for the construction of foundations shall conform to the requirements of Part 5 of this Code.

3.6.1 Concrete

All concrete materials and steel reinforcement used in foundations shall conform to the requirements specified in Chapter 5 unless otherwise specified in this Section. For different types of foundation the recommended concrete properties are shown in Table 6.3.5. However, special considerations should be given for hostile environment (salinity, acidic environment).

Table 6.3.5: Properties of Concrete for Different Types of Foundations

Foundation Type	Minimum cement content (kg/m^3)	Specified Min. 28 days Cylinder Strength (MPa)	Slump (mm)	Remarks
Footing/raft	350	20	25 to 125	Retarder and plasticizer
Drilled shaft/Cast-in-situ pile (tremie concrete)	400	18	125 to 200	recommended.
Driven pile	350	25	25 to 125	Slump test shall be performed as per ASTM C143.

3.6.2 Steel

All steel reinforcement and steel materials used in foundations shall conform to the requirements specified in Chapter 5 unless otherwise specified in this Section. However, this Section considers the corrosivity of soil that is described as under.

Corrosion in soil, water or moist out-door environment is caused by electro-chemical processes. The process takes place in corrosion cells on the steel surface, which consists of an anodic surface, a cathodic surface (where oxygen is reduced) and the electrolyte, which reacts with these surfaces. In the case of general corrosion, the surface erosion is relatively even across the entire surface. Local corrosion however is concentrated to a limited surface area. Pronounced cavity erosion is rather unusual on unprotected carbon steel in soil or water.

In many circumstances, steel corrosion rates are low and steel piles may be used for permanent works in an unprotected condition. The degree of corrosion and whether protection is required depend upon the working environment which can be variable, even within a single installation. Underground corrosion of steel piles driven into undisturbed soils is negligible irrespective of the soi1 type and characteristics. The insignificant corrosion attack is attributed to the low oxygen levels present in undisturbed soil. For the purpose of calculations, a maximum corrosion rate of 0.015

mm per side per year may be used. In recent-fill soils or industrial waste soils, where corrosion rates may be higher, protection systems should be considered.

(a) Atmospheric Corrosion

Atmospheric corrosion of steel of 0.035 mm/side per year may be used for most atmospheric environments.

(b) Corrosion in Fresh Water

Corrosion losses in fresh water immersion zones are generally lower than for sea water so the effective life of steel piles is normally proportionately longer. However, fresh waters are variable and no general advice can be given to quantify the increase in the length of life.

(c) Corrosion in Marine Environment

Marine environments may include several exposure zones with different aggressivity and different corrosion performance.

(i) Below the bed level: Where piles are below the bed level little corrosion occurs and the corrosion rate given for underground corrosion is applicable, that is, 0.015 mm/side per year.

(ii) Seawater immersion zone: Corrosion of steel pilling in immersion conditions is normally low, with a mean corrosion rate of 0.035 mm/side per year.

(iii) Tidal zones: Marine growths in this zone give significant protection to the piling, by sheltering the steel from wave action between tides and by limiting the oxygen supply to the steel surface. The corrosion rate of steels in the tidal zone is similar to that of immersion zone corrosion, i.e. 0.035 mm/side per year. Protection should be provided where necessary, to the steel surfaces to prevent the removal or damage of the marine growth.

(iv) Low water zone: In tidal waters, the low water level and the splash zone are reasons of highest thickness losses, where a mean corrosion rate of 0.075 mm/side per year occurs. Occasionally higher corrosion rates are encountered at the lower water level because of specific local conditions.

(v) Splash and atmospheric zones: In the splash zone, which is a more aggressive environment than the atmospheric zone, corrosion rates are similar to the low water level, i.e. 0.075 mm/side per year. In this zone thick stratified rust layers may develop and at thicknesses greater than 10 mm this tend to spall from steel especially on curved parts of the piles such as the shoulders and the clutches. Rust has a much greater volume than the steel from which it is derived so that the steel corrosion losses are represented by some 10 % to 20 % of the rust thickness. The boundary between splash and atmospheric zones is not well defined, however, corrosion rates diminish rapidly with distance above peak wave height and mean atmospheric corrosion rate of 0.035 mm/side per year can be used.

(d) Method of Assessing Soil Corrosivity

The following variables attributes to accelerated corrosion: (i) acidity and alkalinity; (ii) soluable salts; (iii) bacteria (sulphates usually promote bacteria; (iv) resistivity; (v) moisture content; (vi) pH; and so on. The following charts, Tables 6.3.6a and 6.3.6b provide guides in assessing the corrosivity of soils. The parameters should be measured following relevant Standards of ASTM.

Table 6.3.6a: Soil Corrosivity Scores for Various Parameters

Item/Parameter	Measured value	Score/Mark
Soil composition	Calcareous, marly limestone, sandy marl, non-stratified sand	+2
	Sandy silt, sandy clay, clayey silt	0
	Clay, silty clay	-2
	Peat, marshy soil	-4
Ground water	None	0
	Exist	-1
	Vary	-2

Item/Parameter	Measured value	Score/Mark
Resistivity	10,000 ohm-cm or more	0
	10,000-5,000	-1
	5,000-2,300	-2
	2,300-1,000	-3
	1,000 or less	-4
Moisture content	20% or less	0
	More than 20%	-1
pH	6 or more	0
	Less than 6	-2
Sulphide and hydrogen sulphide	None	0
	Trace	-2
	Exist	-4
Carbonate	5% or more	+2
	5% - 1%	+1
	Less than 1%	0
Chloride	100 mg/kg or less	0
	More than 100 mg/kg	+1
Sulphate	200 mg/kg or less	0
	200 – 500 mg/kg	-1
	500 – 1000 mg/kg	-2
	More than 1000 mg/kg	-3
Cinder and coke	None	0
	Exist	-4

Table 6.3.6b: Soil Corrosivity Rating

Score/Mark	Corrosivity Rating
0 and above	Non-corrosive
0 to -4	Slightly corrosive
-5 to -10	Corrosive
-10 or less	Highly corrosive

(e) Methods of Increasing Effective Life

The effective life of unpainted or otherwise unprotected steel piling depends upon the combined effects of imposed stresses and corrosion. Where measures for increasing the effective life of a structure are necessary, the following should be considered; introduction of a corrosion allowance (i.e. oversized cross-sections of piles, high yield steel etc), anti-corrosion painting, application of a polyethylene (PE) coating (on steel tube piles), zinc coating, electro-chemical (cathodic) protection, casting in cement mortar or concrete, and use of atmospheric corrosion resistant steel products instead of ordinary carbon steel in any foundation work involving steel.

(i) Use of a heavier section: Effective life may be increased by the use of additional steel thickness as a corrosion allowance. Maximum corrosion seldom occurs at the same position as the maximum bending moment. Accordingly, the use of a corrosion allowance is a cost effective method of increasing effective life. It is preferable to use atmospheric corrosion resistant high strength low alloy steel.

(ii) Use of a high yield steel: An alternative to using mild steel in a heavier section is to use a higher yield steel and retain the same section.

(iii) Zinc coatings: Steel piles should normally be coated under shop conditions. Paints should be applied to the cleaned surface by airless spraying and then cured rapidly to produce the required coating thickness in as few coats as possible. Hot zinc-coating of steel piles in soil can achieve normally long-lasting protection, provided that the zinc layer has sufficient thickness. In some soils, especially those with low pH-values, the corrosion of zinc can be high, thereby shortening the protection duration. Low pH-values occur normally in the aerated zone above the lowest ground water level. In such a case, it is recommended to apply protection paint on top of the zinc layer.

(iv) Concrete encasement: Concrete encasement may be used to protect steel piles in marine environment. The use of concrete may be restricted to the splash zone by extending the concrete cope to below the mean high water level, both splash and tidal zones may be protected by extending the cope to below the lowest water level. The concrete itself should be a quantity sufficient to resist seawater attack.

(v) Cathodic protection: The design and application of cathodic protection systems to marine piles structures is a complex operation requiring the experience of specialist firms. Cathodic protection with electric current applied to steel sheet pile wall. Rod-type anodes are connected directly with steel sheet pile. Cathodic protection is considered to be fully effective only up to the half-tide mark. For zones above this level, including the splash zone, alternative methods of protection may be required, in addition to cathodic protection. Where cathodic protection is used on marine structures, provision should be made for earthing ships and buried services to the quay.

(vi) Polyetheline coating: Steel tube piles can be protected effectively by application of a PE-cover of a few millimeter of thickness. This cover can be applied in the factory and is usually placed on a coating of epoxy. Steel tube piles in water, where the mechanical wear is low, can in this way be protected for long time periods. When the steel tube piles with the PE-cover are driven into coarse-grained soil, the effect of damaging the protection layer must be taken into consideration.

(vii) Properly executed anti-corrosion measures, using high-quality methods can protect steel piles in soil or water over periods of 15 to 20 years. PE-cover in combination with epoxy coating can achieve even longer protection times.

3.6.3 Timber

Timber may be used only for foundation of temporary structure and shall conform to the standards specified in Sec 2.9 of Part 5 of this Code. Where timber is exposed to soil or used as load bearing pile above ground water level, it shall be treated in accordance with BDS 819:1975.

3.7 Types of Foundation

3.7.1 Shallow Foundations

Shallow foundations spread the load to the ground at shallow depth. Generally, the capacity of this foundation is derived from bearing.

3.7.2 Footing

Footings are foundations that spread the load to the ground at shallow depths. These include individual column footings, continuous wall footings, and combined footings. Footings shall be provided under walls, pilasters, columns, piers, chimneys etc. bearing on soil or rock, except that footings may be omitted under pier or monolithic concrete walls if safe bearing capacity of the soil or rock is not exceeded.

3.7.3 Raft/Mat

A foundation consisting of continuous slab that covers the entire area beneath the structure and supports all walls and columns is considered as a raft or mat foundation. A raft foundation may be one of the following types:

(i) Flat plate or concrete slab of uniform thickness usually supporting columns spaced uniformly and resting on soils of low compressibility.

(ii) Flat plates as in (a) but thickened under columns to provide adequate shear and moment resistance.

(iii) Two way slab and beam system supporting largely spaced columns on compressible soil.

(iv) Cellular raft or rigid frames consisting of slabs and basement walls, usually used for heavy structures.

3.7.4 Deep Foundations

A cylindrical/box foundation having a ratio of depth to base width greater than 5 is considered a Deep Foundation. Generally, its capacity is derived from friction and end bearing.

3.7.5 Driven Piles

A slender deep foundation unit made of materials such as steel, concrete, wood, or combination thereof, which is pre-manufactured and placed by driving, jacking, jetting or screwing and displacing the soil.

(i) Driven Precast Concrete Piles: Pile structure capable of being driven into the ground and able to resist handling stresses shall be used for this category of piles.

(ii) Driven Cast-in-situ Concrete Piles : A pile formed by driving a steel casing or concrete shell in one or more pieces, which may remain in place after driving or withdrawn, with the inside filled with concrete, falls in this category of piles. Sometimes an enlarged base may be formed by driving out a concrete plug.

(iii) Driven Prestressed Concrete Pile: A pile constructed in prestressed concrete in a casting yard and subsequently driven in the ground when it has attained sufficient strength.

(iv) Timber Piles: Structural timber (Sec 2.9 Part 5) shall be used as piles for temporary structures for directly transmitting the imposed load to soil. Driven timber poles are used to compact and improve the deposit.

3.7.6 Bored Piles/Cast-in-Situ Piles

A deep foundation of generally small diameter, usually less than 600 mm, constructed using percussion or rotary drilling into the soil. These are constructed by concreting bore holes formed by auguring, rotary drilling or percussion drilling with or without using bentonite mud circulation. Excavation or drilling shall be carried out in a manner that will not impair the carrying capacity of the foundations already in place or will not damage adjacent foundations. These foundations may be tested for capacity by load test or for integrity by sonic response or other suitable method. Under-reaming drilled piers can be constructed in cohesive soils to increase the end bearing.

3.7.7 Drilled Pier/Drilled Shafts

Drilled pier is a bored pile with larger diameter (more than 600 mm) constructed by excavating the soil or sinking the foundation.

3.7.8 Caisson/Well

A caisson or well foundation is a deep foundation of large diameter relative to its length that is generally a hollow shaft or box which is sunk to position. It differs from other types of deep foundation in the sense that it undergoes rigid body movement under lateral load, whereas the others are flexible like a beam under such loads. This type of foundation is usually used for bridges and massive structures.

Division B: Design of Foundations (Sections 3.8 to 3.11)

3.8 Shallow Foundation

This Section shall be applicable to isolated Footings, Combined Footings and Raft/Mats.

3.8.1 Distribution of Bearing Pressure

Footing shall be designed to keep the maximum imposed load within the safe bearing values of soil and rock. To prevent unequal settlement footing shall be designed to keep the bearing pressure as nearly uniform as practical. For raft design, distribution of soil pressures should be consistent with the properties of the foundation materials (subsoil) and the structure (raft thickness) and with the principles of geotechnical engineering.

Mat or raft and floating foundations shall only be used when the applied load of building or structure is so arranged as to result in practically uniformly balanced loading, and the soil immediately below the mat is of uniform bearing capacity.

3.8.2 Dimension of Footings

Footings shall generally be proportioned from the allowable bearing pressure and stress limitations imposed by limiting settlement.

The angle of spread of the load from the wall base to outer edge of the ground bearing shall not exceed the following:

Brick or stone masonry	$\frac{1}{2}$ horizontal to 1 vertical
Lime concrete	$\frac{2}{3}$ horizontal to 1 vertical
Cement concrete	1 horizontal to 1 vertical

A footing shall be placed to depth so that:

(a) adequate bearing capacity is achieved,

(b) in case of clayey soil , shrinkage and swelling due to seasonal weather change is not significant,

(c) it is below possible excavation close by, and

(d) it is at least 500 mm below natural ground level unless rock or other weather resistant material is at the surface.

Where footings are to be founded on a slope, the distance of the sloping surface at the base level of the footing measured from the centre of the footing shall not be less than twice the width of the footing.

When adjacent footings are to be placed at different levels, the distance between the edges of footings shall be such as to prevent undesirable overlapping of structures in soil and disturbance of the soil under the higher footing due to excavation of the lower footing.

On a sloping site, footing shall be on a horizontal bearing and stepped. At all changes of levels, footings shall be lapped for a distance of at least equal to the thickness of foundation or three times the height of step, whichever is greater. Adequate precautions shall be taken to prevent tendency for the upper layers of soil to move downhill.

3.8.3 Thickness of Footing

The minimum thickness for different types of footing for light structures (two stories or less in occupancy category A, B, C and D), shall be as follows:

Type of Footing	Minimum Thickness	Remark
Masonry	250 mm; twice the maximum projection from the face of the wall	Greater of the two values shall be selected
Plain concrete	200 mm, or twice the maximum offset in a stepped footing	-
Reinforced concrete	150 mm	Resting on soil
(depth above bottom reinforcement)	300 mm	Resting on pile

3.8.4 Footings in Fill Soil

Footings located in fill are subject to the same bearing capacity, settlement, and dynamic ground stability considerations as footings in natural soil. The behavior of both fill and underlying natural soil should be considered.

3.8.5 Soil and Rock Property Selection

Soil and rock properties defining the strength and compressibility characteristics of foundation materials are required for footing design. Foundation stability and settlement analysis for design shall be conducted using soil and rock properties based on the results of field and laboratory testing.

3.8.6 Minimum Depth of Foundation

The minimum depth of foundation shall be 1.5 m for exterior footing of permanent structures in cohesive soils and 2 m in cohesionless soils. For temporary structures the minimum depth of exterior footing shall be 400 mm. In case of expansive and soils susceptible to weathering effects, the above mentioned minimum depths will be not applicable and may have to be increased.

3.8.7 Scour

Footings supported on soil shall be embedded sufficiently below the maximum computed scour depth or protected with a scour countermeasure.

3.8.8 Mass Movement of Ground in Unstable Areas

In certain areas mass movement of ground may occur from causes independent of the loads applied to the foundation. These include mining subsidence, landslides on unstable slopes and creep on clay slopes. In areas of ground subsidence, foundations and structures should be made sufficiently rigid and strong to withstand the probable worst loading conditions. The construction of structures on slopes which are suspected of being unstable and subject to landslip shall be avoided. Spread foundations on such slopes shall be on a horizontal bearing and stepped. For foundations on clay slopes, the stability of the foundation should be investigated.

3.8.9 Foundation Excavation

Foundation excavation below ground water table particularly in sand shall be made such that the hydraulic gradient at the bottom of the excavation is not increased to a magnitude that would case the foundation soils to loosen due to upward flow of water. Further, footing excavations shall be made such that hydraulic gradients and material removal do not adversely affect adjacent structures. Seepage forces and gradients may be evaluated by standard flow net procedures. Dewatering or cutoff methods to control seepage shall be used when necessary. In case of soil excavation for raft foundations, the following issues should be additionally taken into consideration:

(i) Protection for the excavation using shore or sheet piles and/or retaining system with or without bracing, anchors etc.

(ii) Consideration of the additional bearing capacity of the raft for the depth of the soil excavated.

(iii) Consideration of the reduction of bearing capacity for any upward buoyancy pressure of water.

(iv) Other considerations as mentioned in Sec 3.12.

3.8.10 Design Considerations for Raft foundation

Design provisions given in Sec 3.9.2 shall generally apply. In case the raft supports structure consisting of several parts with varying loads and height, it is advisable to provide separate joints between these parts. Joints shall also be provided wherever there is a change in the direction of the raft. The minimum depth of foundation shall generally be not less than 1.5 m in cohesive soil and 2 m in cohesionless soils. Foundations subject to heavy vibratory loads shall preferably be isolated.

3.8.10.1 Dimensioning

The size and shape of the foundation shall be decided taking into consideration the magnitude of subgrade modulus, the long term deformation of the supporting soil and the distribution of contact pressure.

Distribution of contact pressure underneath a raft is affected by the physical characteristics of the supporting soil. Consideration shall be given to the increased contact pressure developed along the edges of foundation on cohesive soils and the decrease in pressure on granular soils. Both long term and short term deformation and settlement effects shall be considered in the design.

3.8.10.2 Eccentricity

Since raft foundation usually occupies the entire area of a building, it may not be feasible to proportion the raft so that the centroid of the raft coincides with the line of action of the resultant force due to building. In such cases, the effect of eccentricity on the contact pressure distribution shall be considered in the design.

3.8.10.3 Rigidity of Foundation

The rigidity of foundation affects soil pressure distribution which in turn produces additional stresses in the raft due to moments etc. A rigid foundation also generates high secondary stresses. The effects of such rigidity shall be taken into consideration in designing rafts.

3.8.10.4 Methods of Analysis

The essential part of analysis of a raft foundation is the determination of distribution of contact pressure below the mat which is a complex function of the rigidity of raft, and the rigidity of the superstructure and the supporting soil. Any analytical method shall therefore use simplifying assumptions which are reasonably valid for the condition analysed. Choice of a particular method shall therefore be governed by the validity of the assumptions in the particular case.

3.9 Geotechnical Design of Shallow Foundations

3.9.1 General

Shallow foundations on soil shall be designed to support the design loads with adequate bearing and structural capacity and with tolerable settlements. In addition, the capacity of footings subjected to seismic and dynamic loads shall be appropriately evaluated. The location of the resultant pressure on the base of the footings should be maintained preferably within B/6 of the centre of the footing.

3.9.2 Design Load

(a) Shallow foundation design considering bearing capacity due to shear strength shall consider the most unfavourable effect of the following combinations of loading:

(i) Full Dead Load + Normal Live Load

(ii) Full Dead Load + Normal Live Load + Wind Load or Seismic Load

(iii) 0.9 ×(Full Dead Load) + Buoyancy Pressure

(b) Shallow foundation design considering settlement shall consider the most unfavourable effect of the following combinations of loading:

SAND

(i) Full Dead Load + Normal Live Load

(ii) Full Dead Load + Normal Live Load + Wind Load or Seismic Load

CLAY

Full Dead Load + 0.5× Normal Live Load

Normal Live Load is a live load considering floor area reduction factor as used in column design (Sec 2.3.13).

3.9.3 Bearing Capacity of Shallow Foundations

When physical characteristics such as cohesion, angle of internal friction, density etc. are available, the bearing capacity shall be calculated from stability considerations. Established bearing capacity equations shall be used for calculating bearing capacity.

A factor of safety of between 2.0 to 3.0 (depending on engineering judgement on the extent of soil exploration, quality control and monitoring of construction) shall be adopted to obtain allowable bearing pressure when dead load and normal live load is used. Thirty three percent (33%) overstressing above allowable pressure shall be allowed in case of design considering wind or seismic loading. Allowable load shall also limit settlement between supporting elements to a tolerable limit**.**

3.9.3.1 Presumptive bearing capacity for preliminary design

For lightly loaded and small sized structures (two storied or less in occupancy category A, B, C & D) and for preliminary design of any structure, the presumptive bearing values (allowable) as given in Table 6.3.7 may be assumed for uniform soil in the absence of test results.

3.9.3.2 Allowable increase of bearing pressure due to wind and earthquake forces

The allowable bearing pressure of the soil determined in accordance with this Section may be increased by 33 percent when lateral forces due to wind or earthquake act simultaneously with gravity loads. No increase in allowable bearing pressure shall be permitted for gravity loads acting alone. In a zone where seismic forces exist, possibility of liquefaction in loose sand, silt and sandy soils shall be investigated.

Table 6.3.7: Presumptive Values of Bearing Capacity for Lightly Loaded Structures*

Soil Type	Soil Description	Safe Bearing Capacity, kPa
1	Soft Rock or Shale	440
2	Gravel, sandy gravel, silty sandy gravel; very dense and offer high resistance to penetration during excavation (soil shall include the groups GW, GP, GM, GC)	400**
3	Sand (other than fine sand), gravelly sand, silty sand; dry (soil shall include the groups SW, SP, SM, SC)	200**
4	Fine sand; loose & dry (soil shall include the groups SW, SP)	100**

5	Silt, clayey silt, clayey sand; dry lumps which can be easily crushed by finger (soil shall include the groups ML, SC & MH)	150
6	Clay, sandy clay; can be indented with strong thumb pressure (soil shall include the groups CL & CH)	150
7	Soft clay; can be indented with modest thumb pressure (soil shall include the groups CL & CH)	100
8	Very soft clay; can be penetrated several centimeters with thumb pressure (soil shall include the groups CL & CH)	50
9	Organic clay & Peat (soil shall include the groups OH, OL, Pt)	To be determined after investigation.
10	Fills	To be determined after investigation.

* Two stories or less (Occupancy category A, B, C and D)

** 50% of these values shall be used where water table is above the base, or below it within a distance equal to the least dimension of foundation

3.9.4 Settlement of Shallow Foundation

Foundation shall be so designed that the allowable bearing capacity is not exceeded, and the total and differential settlement are within permissible values. Foundations can settle in various ways and each affects the performance of the structure. The simplest mode consists of the entire structure settling uniformly. This mode does not distort the structure. Any damage done is related to the interface between the structure and adjacent ground or adjacent structures. Shearing of utility lines could be a problem. Another possibility is that one side of the structure settles much more than the opposite side and the portions in between settle proportionately. This causes the structure to tilt, but it still does not distort. A nominal tilt will not affect the performance of the structure, although it may create aesthetic and public confidence problems. However, as a result of difference in foundation settlement the structure may settle and distort causing cracks in walls and floors, jamming of doors and windows and overloading of structural members.

3.9.4.1 Total settlement

Total settlement (δ) is the absolute vertical movement of the foundation from its as-constructed position to its loaded position. Total settlement of foundation due to net imposed load shall be estimated in accordance with established engineering principle. An estimate of settlement with respect to the following shall be made.

(i) Elastic compression of the underlying soil below the foundation and of the foundation.

(ii) Consolidation settlement.

(iii) Secondary consolidation/compression of the underlying soil.

(iv) Compression and volume change due to change in effective stress or soil migration associated with lowering or movement of ground water.

(v) Seasonal swelling and shrinkage of expansive clays.

(vi) Ground movement on earth slopes, such as surface erosion, creep or landslide.

(vii) Settlement due to adjacent excavation, mining subsidence and underground erosion.

In normal circumstances of inorganic and organic soil deposits the total settlement is attributed due to the first three factors as mentioned above. The other factors are regarded as special cases. Because soil settlement can have both time-depended and notime-dependent components, it is often categorized in terms short-term settlement (or immediate settlement) which occurs as quickly as the load is applied, and long-term settlement (or delayed settlement), which occurs over some longer period. Many engineers associate consolidation settlement solely with the long term settlement of clay. However, this is not strictly true. Consolidation is related to volume change due to change in effective stress regardless of the type of soil or the time required for the volume change.

3.9.4.2 Elastic/distortion settlement

Elastic Settlement δ_e of foundation soils results from lateral movements of the soil without volume change in response to changes in effective vertical stress. This is non-time dependent phenomenon and similar to the Poisson's effect where an object is loaded in the vertical direction expands laterally. Elastic or distortion settlements primarily occur when the load is confined to a small area, such as a structural foundation, or near the edges of large loaded area such as embankments.

3.9.4.3 Immediate settlement/short term settlement

This vertical compression occurs immediately after the application of loading either on account of elastic behaviour that produces distortion at constant volume and on account of compression of air void. This is sometimes designated as δ_i for sandy soil, even the consolidation component is immediate.

3.9.4.4 Primary consolidation settlement

Primary consolidation settlement or simply the consolidation settlement δ_c of foundation is due to consolidation of the underlying saturated or nearly saturated soil especially cohesive silt or clay. The full deal load and 50% of total live load shall be considered when computing the consolidation settlement of foundations on clay soils.

3.9.4.5 Secondary consolidation settlement

Secondary consolidation settlement δ_s of the foundation is due to secondary compression or consolidation of the underlying saturated or nearly saturated cohesive silt or clay. This is primarily due to particle re-orientation, creep, and decomposition of organic materials. Secondary compression is always time-dependent and can be significant in highly plastic clays, organic soils, and sanitary landfills, but it is negligible in sands and gravels.

3.9.4.6 Differential settlement

Differential settlement is the difference in total settlement between two foundations or two points in the same foundation. It occurs as a result of relative movement between two parts of a building. The related terms describing the effects of differential settlement on the structural as a whole or on parts of it are tilt, rotation and angular distortion/relative rotation which are defined below. Due consideration shall be given to estimate the differential settlement that may occur under the building structure under the following circumstances:

(i) Non-uniformity in subsoil formation within the area covered by the building due to geologic or man-made causes, or anomalies in type, structure, thickness and density of the formation.

(ii) Non-uniform pressure distribution due to non-uniform and incomplete loading.

(iii) Ground water condition during and after construction.

(iv) Loading influence of adjacent structures.

(v) Uneven expansion and contraction due to moisture migration, uneven drying, wetting or softening.

3.9.4.7 Rotation and tilt of shallow foundation

(a) Rotation

Rotation is the angle between the horizontal line and an imaginary straight line connecting any two foundations or two points in a single foundation.

(b) Tilt

Tilt is rotation of the entire superstructure or a well-defined part of it as a result of non-uniform or differential settlement of foundation as a result of which one side of the building settles more than the other thus affecting the verticality of the building.

(c) Angular Distortion/Relative Rotation

Angular distortion or relative rotation is the angle between imaginary straight line indicating the overall tilt of a structure and the imaginary connecting line indicating the inclination of a specific part of it. It is measured as the ratio of differential settlement to the distance between the two points.

(d) Tolerable Settlement, Tilt and Rotation

Allowable or limiting settlement of a building structure will depend on the nature of the structure, the foundation and the soil. Different types of structures have varying degrees of tolerance to settlements and distortions. These variations depend on the type of construction, use of the structure, rigidity of the structure and the presence of sensitive finishes. As a general rule, a total settlement of 25 mm and a differential settlement of 20 mm between columns in most buildings shall be considered safe for buildings on isolated pad footings on sand for working load (un-factored). A total settlement of 40 mm and a differential settlement of 20 mm between columns shall be considered safe for buildings on isolated pad footings on clay soil for working load. Buildings on raft can usually tolerate greater total settlements. Limiting tolerance for distortion and deflections introduced in a structure is necessarily a subjective process, depending on the status of the building and any specific requirements for serviceability. The limiting values, given in Table 6.3.8 may be followed as guidelines.

Table 6.3.8: Permissible Total Settlement, Differential Settlement and Angular Distortion (Tilt) for Shallow Foundations in Soils (in mm) (Adapted from NBCI, 2005)

Type of Structure	Isolated Foundations						Raft Foundation					
	Sand and Hard Clay			Plastic Clay			Sand and Hard Clay			Plastic Clay		
	Maximum Settlement	Differential Settlement	Angular Distortion	Maximum Settlement	Differential Settlement	Angular Distortion	Maximum Settlement	Differential Settlement	Angular Distortion	Maximum Settlement	Differential Settlement	Angular Distortion
Steel Structure	50	0.0033 L	1/300	50	0.0033 L	1/300	75	0.0033 L	1/300	100	0.0033 L	1/300
RCC Structures	50	0.0015 L	1/666	75	0.0015 L	1/666	75	0.0021 L	1/500	100	0.002 L	1/500
Multistoried Building												
(a) RCC or steel framed building with panel walls	60	0.002 L	1/500	75	0.002 L	1/500	75	0.0025 L	1/400	125	0.0033 L	1/300
(b) Load bearing walls												
(i) L/H = 2 *	60	0.0002 L	1/5000	60	0.0002 L	1/5000	Not likely to be encountered					
(ii) L/H = 7 *	60	0.0004 L	1/2500	60	0.0004 L	1/2500	Not likely to be encountered					
Silos	50	0.0015 L	1/666	75	0.0015 L	1/666	100	0.0025 L	1/400	125	0.0025 L	1/400
Water Tank	50	0.0015 L	1/666	75	0.0015 L	1/666	100	0.0025 L	1/400	125	0.0025 L	1/400

Notes: The values given in the Table may be taken only as a guide and the permissible total settlement, differential settlement and tilt (angular distortion) in each case should be decided as per requirements of the 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.

3.9.5 Dynamic Ground Stability or Liquefaction Potential for Foundation Soils

Soil liquefaction is a phenomenon in which a saturated soil deposit loses most, if not all, of its strength and stiffness due to the generation of excess pore water pressure during earthquake-induced ground shaking. It has been a major cause for damage of structures during past earthquakes (e.g., 1964 Niigata Earthquake). Current knowledge of liquefaction is significantly advanced and several evaluation methods are available. Hazards due to liquefaction are routinely evaluated and mitigated in seismically active developed parts of the world.

Liquefaction can be analyzed by a simple comparison of the seismically induced shear stress with the similarly expressed shear stress required to cause initial liquefaction or whatever level of shear strain amplitude is deemed intolerable in design. Usually, the occurrence of 5% double amplitude (DA) axial strain is adopted to define the cyclic strength consistent with 100% porewater pressure build-up. The corresponding strength (CRR) can be obtained by several procedures. Thus, the liquefaction potential of a sand deposit is evaluated in terms of factor of safety F_L, defined as in Eq. 6.3.6. The externally applied cyclic stress ratio (CSR) can be evaluated using Equations 6.3.7a, 6.3.7b and 6.3.8.

$$F_L = \frac{CRR}{CSR} \tag{6.3.6}$$

If the factor of safety F_L is $\leq$ 1, liquefaction is said to take place. Otherwise, liquefaction does not occur. The factor of safety obtained in this way is generally used to identify the depth to which liquefaction is expected to occur in a future earthquake. This information is necessary if countermeasure is to be taken in an in situ deposit of sands.

The cyclic shear stress induced at any point in level ground during an earthquake due to the upward propagation of shear waves can be assessed by means of a simple procedure proposed. If a soil column to a depth z is assumed to move horizontally and if the peak horizontal acceleration on the ground surface is a_{max} , the maximum shear stress τ_{max} acting at the bottom of the soil column is given by

$$\tau_{max} = a_{max}\, r_d(\gamma_t)(z/g) \tag{6.3.7a}$$

$$r_d = 1 - 0.015z \tag{6.3.7b}$$

Where, γ_t is unit weight of the soil, g is the gravitational acceleration, z is the depth and r_d is a stress reduction coefficient to allow for the deformability of the soil column ($r_d < 1$). It is recommended to use the empirical formula given in Eq. 6.3.7b to compute stress reduction coefficient r_d, where z is in meters. Division of both sides of Eq. 6.3.7a by the effective vertical stress $\sigma_v^{'}$ gives

$$CSR = \frac{\tau_{max}}{\sigma_v^{'}} = \frac{a_{max}}{g}\, r_d \frac{\sigma_v}{\sigma_v^{'}} \tag{6.3.8}$$

Where, $\sigma_v = \gamma_t z$ is the total vertical stress. Eq. 6.3.8 has been used widely to assess the magnitude of shear stress induced in a soil element during an earthquake. The peak ground acceleration, a_{max} should be taken from seismic zoning map. One of the advantages of Eq. 6.3.8 is that all the vast amount of information on the horizontal accelerations that has ever been recorded on the ground surface can be used directly to assess the shear stress induced by seismic shaking in the horizontal plane within the ground.

The second step is to determine the cyclic resistance ratio (CRR) of the in situ soil. The cyclic resistance ratio represents the liquefaction resistance of the in situ soil. The most commonly used method for determining the liquefaction resistance is to use the data obtained from the standard penetration test. A cyclic triaxial test may also be used to estimate CRR more accurately. Site response analysis of a site may be carried out to estimate the site amplification factor. For this purpose, dynamic parameters such as shear modulus and damping factors need to be estimated. The site amplification factor is required to estimate a_{max} for a given site properly. The following points are to be noted as regards to soil liquefaction:

- Sandy and silty soils tend to liquefy; clay soils do not undergo liquefaction except the sensitive clays.
- Resistance to liquefaction of sandy soil depends on fine content. Higher the fine content lower is the liquefaction potential.
- As a rule of thumb, any soil that has a SPT value higher than 30 will not liquefy.

Fine grained soils (silty clays/ clayey silt) are susceptible to liquefaction if (Finn et. al., 1994):

- Fraction finer than 0.005 mm $\leq$ 10%
- Liquid limit (LL) $\leq$ 36%
- Natural water content $\leq 0.9 \times$ LL
- Liquidity index $\leq$ 0.75

3.9.6 Structural Design of Shallow Foundations

The foundation members should have enough strength to withstand the stresses induced from soil-foundation interaction. The following important factors should be considered in the structural design of foundations.

3.9.6.1 Loads and reactions

Footings shall be considered as under the action of downward forces, due to the superimposed loads, resisted by an upward pressure exerted by the foundation materials and distributed over the area of the footings as determined by the eccentricity of the resultant of the downward forces. Where piles are used under footings, the upward reaction of the foundation shall be considered as a series of concentrated loads applied at the pile centers, each pile being assumed to carry the computed portion of the total footing load.

3.9.6.2 Isolated and multiple footing reactions

When a single isolated footing supports a column, pier or wall, the footing shall be assumed to act as a cantilever element. When footings support more than one column, pier, or wall, the footing slab shall be designed for the actual conditions of continuity and restraint.

3.9.6.3 Raft foundation reactions

For determining the distribution of contact pressure below a raft it is analyzed either as a rigid or flexible foundation considering the rigidity of the raft, and the rigidity of the superstructure and the supporting soil. Consideration shall be given to the increased contact pressure developed along the edges of raft on cohesive soils and the decrease in contact pressure along the edges on granular soils. Any appropriate analytical method reasonably valid for the condition may be used. Choice of a particular method shall be governed by the validity assumptions used. Numerical analysis of rafts using appropriate software may also be used for determination of reactions, shears and moments.

Both analytical (based on beams on elastic foundation, Eq. 6.3.9) and numerical methods require values of the modulus of subgrade reaction of the soil. For use in preliminary design, indicative values of the modulus of subgrade reaction (*k*) for cohesionless soils and cohesive soils are shown in Tables 6.3.9a and 6.3.9b, respectively.

$$k = 0.65.\left(\frac{E_S B^4}{EI}\right)^{1/12}.\frac{E_S}{(1-\mu^2)}.\frac{1}{B} \qquad (6.3.9)$$

Where, $\boldsymbol{E_s}$= Modulus of elasticity of soil; $\boldsymbol{EI}$ = Flexural rigidity of foundation; $\boldsymbol{B}$ = Width of foundation; $\boldsymbol{\mu}$ = Poisson's ratio of soil.

Table 6.3.9a: Modulus of Subgrade Reaction *(k)* for Cohesionless Soils

Soil Characteristic		***Modulus of Sub-grade Reaction *(k)* of Soil (kN/m³)**	
Relative Density	**Standard Penetration Test Value *(N)* (Blows per 300 mm)**	**For Dry or Moist State**	**For Submerged State**
Loose	<10	15000	9000
Medium	10 to 30	15000 to 47000	9000 to 29000
Dense	30 and over	47000 to 180000	29000 to 108000

*The above values apply to a square plate 300 mm x 300 mm or beams 300 mm wide.

Table 6.3.9b: Modulus of Subgrade Reaction *(k)* for Cohesive Soils

Soil Characteristic		Modulus of Subgrade Reaction, *k* (kN/m³)
Consistency	**Unconfined Compressive Strength (kN/m²)**	
Stiff	100 to 200	27000
Very Stiff	200 to 400	27000 to 54000
Hard	400 and over	54000 to 108000

* The values apply to a square plate 300 mm x 300 mm. The above values are based on the assumption that the average loading intensity does not exceed half the ultimate bearing capacity.

3.9.6.4 Critical section for moment

External moment on any section of a footing shall be determined by passing a vertical plane through the footing and computing the moment of the forces acting over the entire area of the footing on one side of that vertical plane. The critical section for bending shall be taken at the face of the column, pier, or wall. In the case of columns that are not square or rectangular, the section shall be taken at the side of the concentric square of equivalent area. For footings under masonry walls, the critical section shall be taken halfway between the middle and edge of the wall. For footings under metallic column bases, the critical section shall be taken halfway between the column face and the edge of the metallic base. For mat foundations and combined footings critical section should be determined on the basis of maximum positive and negative moments obtained from soil-foundation interaction.

3.9.6.5 Critical section for shear

Computation of shear in footings, and location of critical section shall be in accordance with relevant sections of the structural design part of the Code. Location of critical section shall be measured from the face of column, pier or wall, for footings supporting a column, pier, or wall. For footings supporting a column or pier with metallic base plates, the critical section shall be measured from the location defined in the critical section for moments for footings.

3.9.6.6 Critical section for footings on driven piles/bored piles/drilled piers

Shear on the critical section shall be in accordance with the following. Entire reaction from any driven pile or bored piles, and drilled pier whose center is located $d_p/2$ (d_p = diameter of the pile) or more outside the critical section shall be considered as producing shear on that section. Reaction from any driven pile or drilled shaft whose center is located $d_p/2$ or more inside the critical section shall be considered as producing no shear on that section. For the intermediate position of driven pile or drilled shaft centers, the portion of the driven pile or shaft reaction to be considered as producing shear on the critical section shall be based on linear interpolation between full value at $d_p/2$ outside the section and zero value at $d_p/2$ inside the section.

3.9.6.7 Transfer of Forces at the Base of Column.

All forces and moments applied at base of column or pier shall be transferred to top of footing. If the strength of concrete of footing is less than that of column, then bearing stress of footing concrete and reinforcement should be checked against imposed loading.

Lateral forces shall be transferred to supporting footing in accordance with shear transfer provisions of the relevant sections of the structural design part of the Code.

Bearing on concrete at contact surface between supporting and supported member shall not exceed concrete bearing strength for either surface.

3.9.6.8 Reinforcement

Reinforcement shall be provided across interface between supporting and supported member either by extending main longitudinal reinforcement into footings or by dowels. Reinforcement across interface shall be sufficient to satisfy all of the following:

(i) Reinforcement shall be provided to transfer all force that exceeds concrete bearing strength in supporting and supported member.

(ii) If it is required that loading conditions include uplift, total tensile force shall be resisted by reinforcement only.

(iii) Area of reinforcement shall not be less than 0.005 times gross area of supported member (column) with a minimum of 4 bars.

(iv) Minimum reinforcement of footing and raft shall be governed by temperature and shrinkage reinforcement as per Sec 8.1.11 Chapter 8 of this Part.

Reinforcement of square footings shall be distributed uniformly across the entire width of footing. Reinforcement of rectangular footings shall be distributed uniformly across the entire width of footing in the long direction. In the short direction, the portion of the total reinforcement given by the following equation shall be distributed uniformly over a band width (centered on center line of column or pier) equal to the length of the short side of the footing.

$$\frac{Reinforcement\ in\ band\ width}{Total\ reinforcement\ in\ short\ direction} = \frac{2}{(\beta+1)} \quad (6.3.10)$$

Here, β is the ratio of the footing length to width. The remainder of reinforcement required in the short direction shall be distributed uniformly outside the center band width of footing.

3.9.6.9 Development length and splicing

Computation of development length of reinforcement in footings shall be in accordance with the relevant sections of the structural design part of the Code.

For transfer of force by reinforcement, development length of reinforcement in supporting and supported member required splicing shall be in accordance with the relevant sections (Part. 6, Chapters 6 and 8) of the structural design part of the Code. Critical sections for development length of reinforcement shall be assumed at the same locations as defined above as the critical section for moments and at all other vertical planes where changes in section or reinforcement occur.

3.9.6.10 Dowel size

Diameter of dowels, if used, shall not exceed the diameter of longitudinal reinforcements.

3.10 Geotechnical Design of Deep Foundations

3.10.1 Driven Precast Piles

The provisions of this article shall apply to the design of axially and laterally loaded driven piles in soil. Driven pile foundation shall be designed and installed on the basis of a site investigation report that will include subsurface exploration at locations and depths sufficient to determine the position and adequacy of the bearing soil unless adequate data is available upon which the design and installation of the piles can be based. The report shall include:

(i) Recommended pile type and capacities

(ii) Driving and installation procedure

(iii) Field inspection procedure

(iv) Requirement of pile load test

(v) Durability and quality of pile material

(vi) Designation of bearing stratum or strata

A plan showing clearly the designation of all piles by an identifying system shall be filed prior to installation of such piles. All detailed records for individual piles shall bear an identification corresponding to that shown on the plan. A copy of such plan shall be available at the site for inspection at all times during the construction.

The design and installation of driven pile foundations shall be under the direct supervision of a competent geotechnical/foundation engineer who shall certify that the piles as installed satisfy the design criteria.

3.10.1.1 Application

Pile driving may be considered when footings cannot be founded on granular or stiff cohesive soils within a reasonable depth. At locations where soil conditions would normally permit the use of spread footings but the potential for scour exists, piles may be driven as a protection against scour. Piles may also be driven where an unacceptable amount of settlement of spread footings may occur.

3.10.1.2 Materials

Driven piles may be cast-in-place concrete, pre-cast concrete, pre-stressed concrete, timber, structural steel sections, steel pipe, or a combination of materials.

3.10.1.3 Penetration

Pile penetration shall be determined based on vertical and lateral load capacities of both the pile and subsurface materials. In general, the design penetration for any pile shall be not less than 3D into a hard cohesive or a dense granular material, and not less than 6D into a soft cohesive or loose a granular material.

3.10.1.4 Estimated pile length

Estimated pile lengths of driven piles shall be shown on the drawing and shall be based upon careful evaluation of available subsurface information, axial and lateral capacity calculations, and/or past experience. The maximum length/diameter ratio should not exceed 50 for a single segmental pile.

3.10.1.5 Types of driven piles

Driven piles shall be classified as "friction" or "end bearing" or a combination of both according to the manner in which load transfer is developed. The ultimate load capacity of a pile consists of two parts. One part is due to friction called skin friction or shaft friction or side shear, and the other is due to end bearing at the base or tip of the pile. If the skin friction is greater than about 80% of the end bearing load capacity, the pile is deemed a friction pile and, if the reverse, an end bearing pile. If the end bearing is neglected, the pile is called a “floating pile”.

3.10.1.6 Batter piles

When the lateral resistance of the soil surrounding the piles is inadequate to counteract the horizontal forces transmitted to the foundation, or when increased rigidity of the entire structure is required, batter piles should be used in the foundation. Where negative skin friction loads are expected, batter piles should be avoided, and an alternate method of providing lateral restraint should be used.

Free standing batter piles are subject to bending moments due to their own weight, or external forces from other sources. Batter piles in loose fill or consolidating deposits may become laterally loaded due to settlement of the surrounding soil. In consolidating clay, special precautions, like provision of permanent casing, shall be taken.

3.10.1.7 Selection of soil and rock properties

Soil and rock properties defining the strength and compressibility characteristics of the foundation materials, are required for driven pile design.

3.10.1.8 Pile driving equipment

The pile driving process needs to fulfil assumptions and goals of the design engineer just as much as the design process has to foresee the conception and installation of the pile at the site. This is only possible through the selection of the right driving equipment especially hammer with proper assembly mounted on the most suitable leader, operated according to the specified practices of installation that consists of a series of principle and subsidiary procedures.

There are three principal methods of installing precast displacement piles: jacking, vibratory driving and driving. Jacking is comparatively new method and vibratory driving is suitable to limited soil and pile types (e.g. loose saturated sand, sheet piles). The most common method of installing displacement piles is by driving the piles into the ground by blows of an impact hammer. Because of this, piles installed in this manner are referred to as driven piles. An efficient method of installation requires proper use of the equipment for driving.

The pile driving equipment mainly consists of the components like pile hammer, pile driving leader and driving system components like anvil, cap block, driving head, follower, pile cushion etc. The key to efficient pile driving is a good match of the pile with the hammer and the other system components. Mismatches, often result either inability to drive the pile as specified or in pile damage. A brief account of pile driving equipment especially related to driving by impact hammers is provided in Appendix-F.

3.10.1.9 Design capacity of driven precast pile

The design pile capacity is the maximum load that the driven pile shall support with tolerable movement. In determining the design pile capacity the following items shall be considered:

(i) Ultimate geotechnical capacity (axial and lateral).

(ii) Structural capacity of pile section (axial and lateral).

(iii) The allowable axial load on a pile shall be the least value of the above two capacities.

In determining the design axial capacity, consideration shall be given to the following:

(i) The influence of fluctuations in the elevation of ground water table on capacity.

(ii) The effects of driving piles on adjacent structure and slopes.

(iii) The effects of negative skin friction or down loads from consolidating soil and the effects of lift loads from expansive or swelling soils.

(iv) The influence of construction techniques such as augering or jetting on pile capacity.

(v) The difference between the supporting capacity single pile and that of a group of piles.

(vi) The capacity of an underlying strata to support load of the pile group.

(vii) The possibility of scour and its effect on axial lateral capacity.

3.10.1.10 Ultimate Geotechnical Capacity of Driven Precast Pile for Axial Load

The ultimate load capacity, Q_{ult}, of a pile consists of two parts. One part is due to friction called skin friction or shaft friction or side shear, Q_s and the other is due to end bearing at the base or tip of the pile, Q_b.The ultimate axial capacity (Q_{ult}) of driven piles shall be determined in accordance with the following for compression loading.

$$Q_{ult} = Q_s + Q_b - W \tag{6.3.11}$$

For uplift loading;

$$Q_{ult} \leq 0.7Q_s + W \tag{6.3.12}$$

The allowable or working axial load shall be determined as:

$$Q_{allow} = Q_{ult}/FS \tag{6.3.13}$$

Where, W is the weight of the pile and FS is a gross factor of safety as suggested in Tables 6.3.10a and 6.3.10b. Often, for compression loading, the weight term is neglected if the weight, W, is considered in estimating imposed loading. The ultimate bearing capacity (skin friction and/or end bearing) of a single vertical pile may be determined by any of the following methods.

(i) By the use of static bearing capacity equations

(ii) By the use of SPT and CPT

(iii) By load tests

(iv) By dynamic methods

3.10.1.11 Static bearing capacity equations for driven precast pile capacity

The skin friction, Q_s and end bearing Q_b can be calculated as:

$$Q_s = A_s f_s \tag{6.3.14a}$$

$$Q_b = A_b f_b \tag{6.3.14b}$$

Where, A_s = skin friction area (perimeter area) of the pile=Perimeter × Length

f_s = skin frictional resistance on unit surface area of pile that depends on soil properties and loading conditions (drained or undrained)

A_b = end bearing area of the pile = Cross-sectional area of pile tip (bottom)

f_b = end bearing resistance on unit tip area of pile, that depends on soil properties to a depth of 2B (B is the diameter for a circular pile section or length of sides for a square pile section) from the pile tip and loading conditions (drained or undrained)

For a layered soil system containing n number of layers, end bearing resistance can be calculated considering soil properties of the layer at which the pile rests, and the skin friction resistance considers all the penetrating layers calculated as:

$$Q_S = \sum_{i=1}^{n} \Delta Z_i \times (Perimeter)_i \times (f_s)_i \qquad (6.3.15)$$

Where, ΔZ_i represents the thickness of any i^{th} layer and $(Perimeter)_i$ is the perimeter of the pile in that layer. The manner in which skin friction is transferred to the adjacent soil depends on the soil type. In fine-grained soils, the load transfer is nonlinear and decreases with depth. As a result, elastic compression of the pile is not uniform; more compression occurs on the top part than on the bottom part of the pile. For coarse-grained soils, the load transfer is approximately linear with depth (higher loads at the top and lower at the bottom).

In order to mobilize skin friction and end bearing, some movement of the pile is necessary. Field tests revealed that to mobilize the full skin friction a vertical displacement of 5 to 10 mm is required. The actual vertical displacement depends on the strength of soil and is independent of the pile length and diameter. The full end bearing resistance is mobilized in driven piles when the vertical displacement is about 10% of the pile tip diameter. For bored piles or drilled shafts, a vertical displacement of about 30% of the pile tip diameter is required. The full end bearing resistance is mobilized when slip or failure zones similar to shallow foundations are formed. The end bearing resistance can then be calculated by analogy with shallow foundations. The important bearing capacity factor is N_q.

The full skin friction and full end bearing are not mobilized at the same displacement. The skin friction is mobilized at about one-tenth of the displacement required to mobilize the end bearing resistance. This is important in deciding on the factor of safety to be applied to the ultimate load. Depending on the tolerable settlement, different factors of safety can be applied to skin friction and to end bearing.

Generally, piles driven into loose, coarse-grained soils tend to density the adjacent soil. When piles are driven into dense, coarse-grained soils, the soil adjacent to the pile becomes loose. Pile driving usually remolds fine-grained soils near the pile shaft. The implication of pile installation is that the intact shear strength of the soil is changed and one must account for this change in estimations of the load capacity.

3.10.1.12 Axial capacity of driven precast pile in cohesive soil using static bearing capacity equations

The ultimate axial capacity of driven piles in cohesive may be calculated from static formula, given by Equations 6.3.14a, 6.3.14b and 6.3.15, using a total stress method for undrained loading conditions, or an effective stress method for drained loading conditions. Appropriate values of adhesion factor (α) and coefficient of horizontal soil stress (k_s) for cohesive soils that are consistent with soil condition and pile installation procedure may be used. There are basically two approaches for calculating skin friction:

(i) The α-method that is based on total stress analysis and is normally used to estimate the short term load capacity of piles embedded in fine grained soils. In this method, a coefficient α is used to relate the undrained shear strength c_u or s_u to the adhesive stress (f_s) along the pile shaft. As such,

$$Q_s = \alpha c_u A_s \tag{6.3.16}$$

$\alpha = 1.0$	for clays with $c_u \leq 25$ kN/m^2
$\alpha = 0.5$	for clays with $c_u \geq 70$ kN/m^2
$\alpha = 1 - \left(\frac{c_u - 25}{70}\right)$	for clays with 25 kN/m^2 < c_u < 70 kN/m^2

The end bearing in such a case is found by analogy with shallow foundations and is expressed as:

$$Q_b = (c_u)_b (N_c)_b A_b \tag{6.3.17}$$

N_c is a bearing capacity factor and for deep foundation the value is usually 9. c_u is the undrained shear strength of soil at the base of the pile. The suffix 'b's are indicatives of base of pile. The general equation for N_c is, however, as follows.

$$N_c = 6\left[1 + 0.2\left(\frac{L}{D_b}\right)\right] \quad \leq 9 \tag{6.3.18}$$

D_b represents the diameter of the pile at base and L is the total length of pile. The skin friction value, $f_b = (c_u)_b (N_c)_b$ should not exceed 4.0 MPa.

(ii) The β -method is based on an effective stress analysis and is used to determine both the short term and long term pile load capacities. The friction along the pile shaft is found using Coulomb's friction law, where the friction stress is given by $f_s = \mu\sigma_x' = \sigma_x' tan\phi'$. The lateral effective stress, σ_x' is proportional to vertical effective stress, σ_z' by a co-efficient, K. As such,

$$f_s = K\sigma_z' tan\phi' = \beta\sigma_z' \tag{6.3.19a}$$

Where,

$$\beta = Ktan\phi' = K_o tan\phi' = (1 - sin\phi')\sqrt{OCR} \qquad (6.3.19b)$$

ϕ' is the effective angle of internal friction of soil and OCR is the over-consolidation ratio. For normally consolidated clay, β varies from 0.25 to 0.29. The value of β decreases for a very long pile, as such a correction factor is used.

$$Correction\ factor\ for\ \beta = log\left(\frac{180}{L}\right) \geq 0.5 \qquad (6.3.19c)$$

The end bearing capacity is calculated by analogy with the bearing capacity of shallow footings and is determined from:

$$f_b = (\sigma_v')_b (N_q)_b \qquad (6.3.20)$$

Where, N_q is a bearing capacity factor that depends on angle of internal friction ϕ' of the soil at the base of the pile, as presented in Figure 6.3.2. Subscript "b" designates the parameters at the base soil.

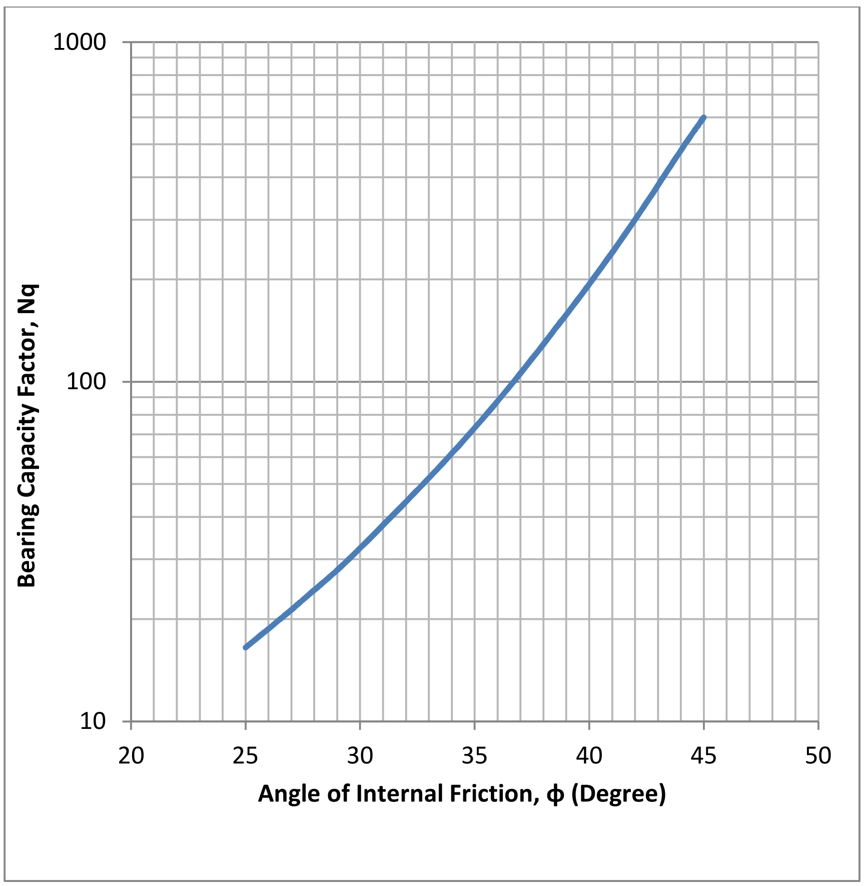


Figure 6.3.2 Bearing capacity factor N_q for deep foundation (After Berezantzev et. al. 1961)

3.10.1.13 Axial Capacity of driven precast pile in cohesive soil using SPT values

Standard Penetration Test N-value is a measure of consistency of clay soil and indirectly the measure of cohesion. The skin friction of pile can thus be estimated from N-value. The following relation may be used for preliminary design of ultimate capacity of concrete piles in clay soil.

For skin friction the relationship is as under.

$$f_s = 1.8\overline{N}_{60} \quad (in\ k\mathrm{Pa}) \quad \leq 70\ k\mathrm{Pa} \qquad (6.3.21)$$

For end bearing, the relationship is as under.

$$f_b = 45N_{60} \quad (\text{in kPa}) \quad \leq 4000\ k\mathrm{Pa} \qquad (6.3.22)$$

Where, $\overline{N}_{60}$ is the average N-value over the pile shaft length and N_{60} is the N-value in the vicinity of pile tip. A factor of safety of 3.5 shall be used to estimate allowable capacity.

3.10.1.14 Axial capacity of driven precast pile in cohesionless soil using static bearing capacity equations

Piles in cohesionless soils shall be designed by effective stress methods of analysis for drained loading conditions. The ultimate axial capacity of piles in cohesionless soils may also be calculated using empirical effective stress method or from in-situ methods and analysis such as the cone penetration or pressure meter tests. Dynamic formula may be used for driven piles in cohesionless soils such as gravels, coarse sand and deposits where pore pressure developed due to driving is quickly dissipated. For piles in cohesionless soil, the ultimate side resistance may be estimated using the following formula:

$$f_s = \beta\sigma_z^{'} \qquad (6.3.23)$$

Where, $\sigma_z^{'}$ is the effective vertical stress at the level under consideration. The values for β are as under.

$\beta = 0.10$ for $\phi = 33^o$

$\beta = 0.20$ for $\phi = 35^o$

$\beta = 0.35$ for $\phi = 37^o$

For uncemented calcareous sand the value of β varies from 0.05 to 0.10.

The following equation, as used for cohesive soil, may be used to compute the ultimate end bearing capacity of piles in sandy soil in which, the maximum effective stress, σ_z' allowed for the computation is 240 kPa. Figure 6.3.2 may also be used to estimate the value of N_q.

$$f_b = (\sigma_v')_b \left(N_q\right)_b \quad (6.3.24)$$

N_q = 8 to 12 for loose sand

N_q = 12 to 40 for medium sand

N_q = 40 for dense sand

3.10.1.15 Critical depth for end bearing and skin friction

The vertical effective stress (σ_v' or σ_z') increases with depth. Hence the skin friction should increase with depth indefinitely. In reality skin friction does not increase indefinitely. It is believed that skin friction would become a constant at a certain depth. This depth is named critical depth. Pile end bearing in sandy soils is also related to effective stress. Experimental data indicates that end bearing capacity does not also increase with depth indefinitely. Due to lack of a valid theory, Engineers use the same critical depth concept adopted for skin friction for end bearing capacity as well. Both the skin friction and the end bearing capacity are assumed to increase till the critical depth, D_c and then maintain a constant value. Following approximations may be used for the critical depth in relation to diameter of pile, D.

$D_c = 10D$ for loose sand

$D_c = 15D$ for medium dense sand

$D_c = 20D$ for dense sand

3.10.1.16 Axial Capacity of Driven Precast Pile in Cohesionless Soil using SPT Values

Standard Penetration Test N-value is a measure of relative density hence angle of internal friction of cohesionless soil. The skin friction of pile can thus be estimated from N-value. The following relation may be used for ultimate capacity of concrete piles in cohesionless soil and non-plastic silt.

For skin friction the relationship is as under.

For sand:

$$f_s = 2\bar{N}_{60} \quad \text{(in kPa)} \quad \leq 60 \text{ kPa} \qquad (6.3.25)$$

For non-plastic silt:

$$f_s = 1.7\bar{N}_{60} \quad \text{(in kPa)} \quad \leq 60 \text{ kPa} \qquad (6.3.26)$$

For end bearing, the relationship is as under.

For sand:

$$f_b = 40N_{60}\left(\frac{L}{D}\right) \text{ (in kPa) } \leq 400N_{60} \text{ and } \leq 11000 \text{ kPa} \qquad (6.3.27)$$

For non-plastic silt:

$$f_b = 30N_{60}\left(\frac{L}{D}\right) \text{ (in kPa) } \leq 300N_{60} \text{ and } \leq 11000 \text{ kPa} \qquad (6.3.28)$$

Where, $\bar{N}_{60}$ is the average N-value over the pile shaft length and N_{60} is the N-value in the vicinity of pile tip. A higher factor of safety of 3.5 should be used to estimate allowable capacity.

3.10.1.17 Axial capacity of driven precast pile using pile load Test

Generally, the load on test pile to determine ultimate capacity is twice the design load. The test load on service/working pile is 1.5 times the design load. The following criteria should be met in deciding the allowable/safe pile capacity.

Safe Load for Single Pile

(a) Two thirds of the final load at which the load displacement attains a value of 12 mm unless otherwise required in a given case on the basis of nature and type of structure in which case, the safe load should be corresponding to the stated total displacement permissible.

(b) Fifty (50) percent of the final load at which the total displacement equals to 10 percent of pile diameter case of uniform diameter piles and 7.5 percent of bulb diameter in case of under-reamed piles.

Safe Load for Pile Group

(a) Final load at which the load displacement attains a value of 25 mm unless otherwise required in a given case on the basis of nature and type of structure, and

(b) Two thirds of the final load at which the total displacement attains a value of 40 mm.

3.10.1.18 Selection of factor of safety for driven precast pile

Driven pile in soil shall be designed for a minimum overall factor of safety of 2.0 against bearing capacity failure (end bearing, side resistance or combined) when the design is based on the results of a load test conducted at the site, with good quality control. Otherwise, it shall be designed for a minimum factor of safety 3.0. The minimum recommended overall factor of safety is based on an assumed normal level of field quality control during construction. If a normal level of field quality control cannot be assured, higher minimum factors of safety shall be used. The recommended values of overall factor of safety on ultimate axial load capacity based on specified construction control is given in Tables 6.3.10a and 6.3.10b.

Partial factor of safety may be used independently for skin friction and end bearing. The values of partial factor of safety may be taken as 1.5 and 3.0 respectively for skin friction and end bearing. The design/allowable load may be taken as the minimum of the values considering overall and partial factor of safety.

Table 6.3.10a: Factor of Safety for Deep Foundation for Downward and Upward Load

Structure	Design Life (yrs.)	Probability of Failure	Design Factor of Safety			
			Good Control	Normal Control	Poor Control	V. Poor Control
Monument	> 100	10^{-5}	2.30	3.00	3.50	4.00
Permanent	25 -100	10^{-4}	2.00	2.50	2.80	3.00
Temporary	< 25	10^{-3}	1.40	2.00	2.30	2.80

Table 6.3.10b: Guidelines for Investigation, Analysis and Construction Control

Item	Good Control	Normal Control	Poor Control	V. Poor Control
Proper Subsoil Investigation	Yes	Yes	Yes	Yes
Proper Review of Subsoil Report	Yes	Yes	Yes	Yes
Supervision by Competent Geotechnical/Foundation Engineer	Yes	Yes	Yes	No

Item	**Good Control**	**Normal Control**	**Poor Control**	**V. Poor Control**
Load Test Data	Yes	Yes	Yes	No
Qualification of Contractor	Yes	Yes	No	No
Proper Construction Equipment's	Yes	No	No	No
Maintaining Proper Construction Log	Yes	No	No	No

3.10.1.19 Group piles and group capacity of driven precast piles

All piles shall be braced to provide lateral stability in all directions. Three or more piles connected by a rigid cap shall be considered as being braced (stable), provided that the piles are located in a radial direction from the centroid of the group, not less than 60^{o} apart circumferentially. A two pile group in a rigid cap shall be considered to be braced along the axis connecting the two piles. Piles supporting walls shall be driven alternately in lines at least 300 mm apart and located symmetrically under the centre of gravity of the wall load, unless effective measures are taken to cater for eccentricity and lateral forces, or the wall piles are adequately braced to provide lateral stability. Individual piles are considered stable if the pile tops are laterally braced in two directions by construction, such as a structural floor slab, grade beams, struts, or walls.

Group pile capacity of driven piles should be determined as the product of the group efficiency, number of piles in the group and the capacity of a single pile. In general, a group efficiency value of 1.0 should be used except for friction piles driven in cohesive soils. The minimum center-to-center pile spacing of 2.5B is recommended. The nominal dimensions and length of all the piles in a group should be similar.

3.10.1.20 Pile caps

Pile caps shall be of reinforced concrete. The soil immediately below the pile cap shall not be considered as carrying any vertical load. The tops of all piles shall be embedded not less than 75 mm into pile caps and the cap shall extend at least 100 mm beyond the edge of all piles. The tops of all piles shall be cut back to sound material before capping. The pile cap shall be rigid enough, so that the imposed load can be distributed on the piles in a group equitably. The cap shall generally be cast over a 75 mm thick levelling course of concrete. The clear cover for the main reinforcement in the cap slab under such condition shall not be less than 50 mm.

3.10.1.21 Lateral load capacity on driven precast piles

Lateral capacity of vertical single piles shall be the least of the values calculated on the basis of soil failure, structural capacity of the pile and deflection of the pile head. In the analysis, pile head conditions (fixed-head or free-head) should be considered. For estimating the depth of fixity, established method of analysis shall be used. The main reinforcement of pile foundation is usually governed by the lateral load capacity and vice versa. Deflection calculations require horizontal subgrade modulus of the surrounding soil. When considering lateral load on piles, the effect of other coexistent loads, including axial load on the pile, shall be taken into consideration for checking structural capacity of the shaft.

To determine lateral load capacity, lateral load tests shall be performed with at least two times the proposed design working load. Allowable lateral load capacity will be the least from the following criteria.

(i) Half of the lateral load at which lateral movement of the pile head is 12 mm or lateral load corresponding to any other specified displacement as per performance requirements.

(ii) Final load at which the total displacement corresponds to 5 mm or lateral load corresponding to any other specified displacement as per performance requirements.

All piles standing unbraced in air, water or soils not capable of providing lateral support shall be designed as columns in accordance with the provisions of this Code.

3.10.1.22 Vertical ground movement and negative skin friction in driven precast piles

The potential for external loading on a pile by vertical ground movements shall be considered as part of the design. Vertical ground movements may result in negative skin friction or downdrag loads due to settlement of compressible soils or may result in uplift loads due to heave of expansive soils. For design purposes, the full magnitude of maximum vertical ground movement shall be assumed.

Driven piles installed in compressible fill or soft soil subject to compression shall be designed against downward load due to downdrag. The potential for external loading on a pile by negative skin friction/downdrag due to settlement of compressible soil shall be considered as a part of the design load. Evaluation of negative skin friction shall include a load-transfer method of analysis to determine the neutral point (i.e., point of zero relative displacement) and load distribution along shaft. Due to the possible time dependence associated with vertical ground movement, the analysis shall consider the effect of time on load transfer between the ground and shaft and the analysis shall be performed for the time period relating to the maximum axial load transfer to the pile. Negative skin friction loads may be reduced by application of bitumen or other viscous coatings to the pile surfaces. In estimating negative skin friction the following factors shall be considered :

(i) Relative movement between soil and pile shaft.

(ii) Relative movement between any underlying compressible soil and pile shaft.

(iii) Elastic compression of the pile under the working load.

(iv) The rate of consolidation of the compressible layer.

(v) Negative skin friction is mobilized only when tendency for relative movement between pile shaft and surrounding soil exists.

3.10.1.23 Driven precast pile in expansive soils (upward movement)

Piles driven in swelling soils may be subjected to uplift forces in the zone of seasonal moisture change. Piles shall extend a sufficient distance into moisture-stable soils to provide adequate resistance to swelling uplift forces. In addition, sufficient clearance shall be provided between the ground surface and the underside of pile caps or grade beams to preclude the application of uplift loads at the pile cap. Uplift loads may be reduced by application of bitumen or other viscous coatings to the pile surface in the swelling zone.

3.10.1.24 Dynamic/Seismic Design of Driven Precast Pile

In case of submerged loose sands, vibration caused by earthquake may cause liquefaction or excessive total and differential settlements. This aspect of the problem shall be investigated and appropriate methods of improvements should be adopted to achieve suitable values of N. Alternatively, large diameter drilled pier foundation shall be provided and taken to depths well into the layers which are not likely to liquefy.

3.10.1.25 Protection against corrosion and abrasion in driven precast pile

Where conditions of exposure warrant a concrete encasement or other corrosion protections shall be used on steel piles and steel shells. Exposed steel piles or steel shells shall not he used in salt or brackish water, and only with caution in fresh water. Details are given in Sec 3.6.2.

3.10.1.26 Dynamic monitoring of driven precast pile

Dynamic monitoring may be specified for piles installed in difficult subsurface conditions such as soils with obstructions and boulders to evaluate compliance with structural pile capacity. Dynamic monitoring may also be considered for geotechnical capacity verification, where the size of the project or other limitations deters static load testing.

3.10.1.27 Maximum allowable driving stresses in driven precast pile

Maximum allowable driving stresses in pile material for top driven piles shall not exceed $0.9f_y$ (compression), $0.9f_y$ (tension) for steel piles, $0.85f_c^{'}$ concrete (compression) and $0.7f_y$ steel reinforcement (tension) for concrete piles and $0.85f_c^{'} - f_{pc}$ (compression) for prestressed concrete piles.

3.10.1.28 Effect of buoyancy in driven precast pile

The effects of hydrostatic pressure shall be considered in the design of driven piles, where used with foundation subjected to buoyancy forces.

3.10.1.29 Protection against Deterioration of Driven Precast Piles

(a) Steel Pile

A steel pile design shall consider that steel piles may be subject to corrosion, particularly in fill soils (low pH soils, acidic, pH value <5.5) and marine environments. In fact, extremely acid soils (below pH 4.5) and very strongly alkaline soils (above pH 9.1) have significantly high corrosion loss rates when compared to other soils. For structural elements, the Code considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: Chloride concentration is 500 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is less than 6. A field electric resistivity survey or resistivity testing and pH testing of soil and ground water samples should be used to evaluate the corrosion potential.

Methods of protecting steel piling in corrosive environments include use of protective coatings, cathodic protection, and increased steel area. The corrosion guidelines are provided in Tables 6.3.6a and 6.3.6b.

(b) Concrete Pile

A concrete pile foundation design shall consider that deterioration of concrete piles can occur due to sulfates in soil, ground water, or sea water; chlorides in soils and chemical wastes; acidic ground water an organic acids. Laboratory testing of soil and ground water samples for sulfates and pH is usually sufficient to assess pile deterioration potential. A full chemical analysis of soil and water samples is recommended when chemical wastes are suspected. Methods of protecting concrete piling include dense impermeable concrete, sulfate resisting Portland cement, minimum cover requirements for reinforcement and use of epoxies, resins, or other protective coatings.

(c) Timber Pile

A timber pile foundation (used for temporary structures) design shall consider that deterioration of timber piles can occur due to decay from wetting and drying cycles or from insects or marine borers Methods of protecting timber piling include pressure treating with creosote or other wood preservers.

3.10.1.30 Pile spacing, clearance and embedment in driven precast pile

End bearing driven piles shall be proportioned such that the minimum center-to-center pile spacing shall exceed the greater of 750 mm or 2.5 pile diameters/widths. The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 100 mm. The spacing of piles shall be that the average load on the supporting strata will not exceed the safe bearing value of those strata as determined by test boring or other established methods.

Piles deriving their capacity from frictional resistance 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, in such cases, the spacing shall not be less than 3.0 times the diameter of the shaft. The tops of piles shall project not less than 75 mm into concrete after all damaged pile material has been removed.

3.10.1.31 Structural capacity of driven precast pile section

The cross-section of driven piles shall be of sufficient size and pile material shall have the necessary structural strength to resist all handling stresses during driving or installation and the necessary strength to transmit the load imposed on them to the underlying and surrounding soil. Pile diameter/cross-section of a pile shaft at any level shall not be less than the designated nominal diameter/cross-section. The structural design of piles must consider each of the following loading conditions.

(i) Handling loads are those imposed on the pile between the time it is fabricated and the time it is in the pile driver leads and ready to be driven. They are generated by cranes, forklifts, and other construction equipment.

(ii) Driving loads are produced by the pile hammer during driving.

(iii) Service loads are the design loads from the completed structures.

The maximum allowable stress on a pile shall not exceed $0.33f_c^{'}$ for precast concrete piles and $33f_c^{'} - f_{pc}$ for prestressed concrete piles and $0.25f_y$ for steel H-piles. The axial carrying capacity of a pile fully embedded in soil with undrained shear strength greater than 10 kN/m^2 shall not be limited by its strength as long column. For driven piles in weaker soils (undrained shear strength less than 10 kN/m^2), due consideration shall be given to determine whether the shaft behaves as a long column or not. If necessary, suitable reductions shall be made in its structural strength considering buckling. The effective length of a pile not secured against buckling by adequate bracing shall be governed by fixity conditions imposed on it by the structure it supports and by the nature of the soil in which it is installed.

Minimum Reinforcement in Driven Concrete Pile

The longitudinal and transverse steel provided in piles should enable the pile to :

- Withstand handling stresses
- Endure driving stresses
- Provide the necessary structural capacity

The maximum bending stress is produced while handling if the pile is pitched at the head. To prevent whipping during handling, length/diameter ratio of the pile should never exceed 50. Otherwise, segmental pile should be used. Considering all of these, the recommended area of main reinforcement for precast concrete piles, designed mainly for vertical load with small lateral capacity, should not be less than the following percentages of the cross sectional area of the piles. In all cases, its adequacy for handling stresses shall be checked. The following reinforcement provisions may not be valid for laterally loaded piles or piles for uplift resistance.

(i) Pile length < 30 times the least width : 1.00%

(ii) Pile length 30 to 40 times the least width : 1.5%

(iii) Pile length > 40 times the least width : 2%

The lateral reinforcement resists the driving stresses induced in the piles and should be in the form hoops or links of diameter not less than 6 mm. The volume of lateral reinforcement shall not be less than the following :

(i) At each end of the pile for a distance of about three times the least width/diameter – not less than 0.4% of the gross volume of the pile.

(ii) In the body of the pile – not less than 0.2% of the gross volume of the pile.

(iii) The transition between closer spacing and the maximum should be gradual over a length of 3 times the least width/diameter.

Minimum Grades of Concrete

The minimum 28 days cylinder strength of concrete for driven piles is 21 MPa. Depending on driving stresses, the following grades of concrete should be used.

(i) For hard driving (driving stress > 1000 kN/m^2) – 28 MPa

(ii) For easy driving (driving stress ≤ 1000 kN/m^2) – 21 MPa

3.10.2 Driven Cast-in-Place Concrete Piles

Driven cast-in-place concrete piles shall be in general cast in metal shells driven into the soil that will remain permanently in place. However, other types of cast-in-place piles, plain or reinforced, cased or uncased, may be used if the soil conditions permit their use and if their design and method of placing are satisfactory.

3.10.2.1 Shape

Cast-in-place concrete piles may have a uniform cross-section or may be tapered over any portion.

3.10.2.2 Minimum area

The minimum area at the butt of the pile shall be 650 cm^2 and the minimum diameter at the tip of the pile shall be 200 mm.

3.10.2.3 General reinforcement requirements

Depending on the driving and installation conditions and the loading condition, the amount of reinforcement and its arrangement shall vary. Cast-in-place piles, carrying axial loads only, where the possibility of lateral forces being applied to the piles is insignificant, need not be reinforced where the soil provides adequate lateral support. Those portions of cast-in-place concrete piles that are not supported laterally shall be designed as reinforced concrete columns and the reinforcing steel shall extend 3000 mm below the plane where the soil provides adequate lateral restraint. Where the shell is smooth pipe and more than 3 mm in thickness, it may be considered as load carrying in the absence of corrosion. Where the shell is corrugated and is at least 2 mm in thickness, it may be considered as providing confinement in the absence of corrosion.

3.10.2.4 Reinforcement in superstructure

Sufficient reinforcement shall be provided at the junction of the pile with the superstructure to make a suitable connection. The embedment of the reinforcement into the cap shall be as specified for precast piles.

3.10.2.5 Shell requirements

The shell shall be of sufficient thickness and strength, so as to hold its original form and show no harmful distortion after it and adjacent shells had driven and the driving core, if any, has been withdrawn. The plans shall stipulate that alternative designs of the shell must be approved by the Engineer before driving is done.

3.10.2.6 Splices

Piles may be spliced provided the splice develops the full strength of the pile. Splices should be detailed on the contract plans. Any alternative method of splicing providing equal results may be considered for approval.

3.10.2.7 Reinforcement cover

The reinforcement shall be placed a clear distance of not less than 50 mm from the cased or uncased sides. When piles are in corrosive or marine environments, or when concrete is placed by the water or slurry displacement methods, the clear distance shall not be less than 75 mm for uncased piles and piles with shells not sufficiently corrosion resistant. Reinforcements shall extend to within 100 mm of the edge of the pile cap.

3.10.2.8 Installation

Steel cased piles shall have the steel shell mandrel driven their full length in contact with surrounding soil, left permanently in place and filled with concrete. No pile shall be driven within 4.5 times the average pile diameter of a pile filled with concrete less than 24 hours old. Concrete shall not be placed in steel shells within the heave range of driving.

3.10.2.9 Concreting

For bored or driven cast-in-situ piles, concrete shall be deposited in such a way as to preclude segregation. Concrete shall be deposited continuously until it is brought to the required level. The top surface shall be maintained as level as possible and the formation of seams shall be avoided.

For under-reamed piles, the slump of concrete shall range between 100 mm and 150 mm for concreting in water free holes. For large diameter holes concrete may be placed by tremie or by drop bottom bucket; for small diameter boreholes a tremie shall be utilized.

A slump of 125 mm to 200 mm shall be maintained for concreting by tremie. In case of tremie concreting for piles of smaller diameter and length up to 10 m, the minimum cement content shall be 350 kg/m^3 of concrete. For larger diameter and/or deeper piles, the minimum cement content shall be 400 kg/m^3 of concrete.

For concreting under water, the concrete shall contain at least 10 percent more cement than that required for the same mix placed in the dry. The amount of coarse aggregate shall be not less than one and a half times, nor more than two times, that of the fine aggregate. The materials shall be so proportioned as to produce a concrete having a slump of not less than 125 mm, nor more than 200 mm.

3.10.2.10 Structural integrity

Bored piles shall be installed in such a manner and sequence as to prevent distortion or damage to piles being installed or already in place, to the extent that such distortion or damage affects the structural integrity of pile.

3.10.3 Prestressed Concrete Piles

3.10.3.1 Shape and size

Prestressed concrete piles that are generally octagonal, square or circular shall be of approved size and shape. Concrete in prestressed piles shall have a minimum compressive strength (cylinder), f_c' of 35 MPa at 28 days. Prestressed concrete piles may be solid or hollow. For hollow piles, precautionary measures should be taken to prevent breakage due to internal water pressure during driving.

3.10.3.2 Reinforcement

Within the context of this Code, longitudinal prestressing is not considered as load-bearing reinforcement. Sufficient prestressing steel in the form of high-tensile wire, strand, or bar should be used so that the effective prestress after losses is sufficient to resist the handling, driving, and service-load stresses. Post-tensioned piles are cast with sufficient mild steel reinforcement to resist handling stresses before stressing. For pretensioned piles, the longitudinal prestressing steel should be enclosed in a steel spiral with the minimum wire size ranging from ACI 318 W3.5 (nominal area 0.035 in^2, nominal dia=0.211 inch) to W5 (nominal area 0.05 in^2, nominal dia=0.252 inch) depending on the pile size. The wire spiral should have a maximum 6 in. (150 mm) pitch with closer spacing at each end of the pile and several close turns at the tip and pile head. The close spacing should extend over at least twice the diameter or thickness of the pile, and the few turns near the ends are often at 1 in. (25 mm) spacing. Occasionally, prestressed piles are designed and constructed with conventional reinforcement in addition to the prestressing steel to increase the structural capacity and ductility of the pile. This reinforcement reduces the stresses in the concrete and should be taken into account.

For prestressed concrete piles, the effective prestress after all losses should not be less than 700 lb/in^2 (4.8 MPa). Significantly higher effective prestress values are commonly used and may be necessary to control driving stresses in some situations. Bending stresses shall be investigated for all conditions of handling, taking into account the weight of the pile plus 50 percent allowance for impact, with tensile stresses limited to $5\sqrt{f_c'}$.

3.10.3.3 Vertical and spiral reinforcement

The full length of vertical reinforcement shall be enclosed within spiral reinforcement. For piles up to 600 mm in diameter, spiral wire shall be No.5 (U.S. Steel Wire Gage). Spiral reinforcement at the ends of these piles shall have a pitch of 75 mm for approximately 16 turns.

In addition, the top 150 mm of pile shall have five turns of spiral winding at 25 mm pitch. For the remainder of the pile, the vertical steel shall be enclosed with spiral reinforcement with not more than 150 mm pitch. For piles having diameters greater than 600 mm. spiral wire shall be No.4 (U.S. Steel Wire Gauge). Spiral reinforcement at the end of these piles shall have a pitch of 50 mm for approximately 16 turns. In addition, the top 150 mm of pile shall have four turns of spiral winding at 38 mm pitch. For the remainder of the pile, the vertical steel shall be enclosed with spiral reinforcement with not more than 100 mm pitch. The reinforcement shall be placed at a clear distance from the face of the prestressed pile of not less than 50 mm.

3.10.3.4 Driving and handling stresses

A prestressed pile shall not be driven before the concrete has attained a compressive strength of at least 28 MPa, but not less than such strength sufficient to withstand handling and driving forces.

3.10.4 Bored Piles

In bored cast in place piles, the holes are first bored with a permanent or temporary casing or by using bentonite slurry to stabilize the sides of the bore. A prefabricated steel cage is then lowered into the hole and concreting is carried by tremie method.

3.10.4.1 Shape and size

Bored cast-in-situ concrete piles that are generally circular in section shall be of approved size and shape. Concrete in bored cast-in-situ concrete piles shall have a minimum compressive strength (cylinder), f_c' of 21 MPa at 28 days.

3.10.4.2 Dimension

All shafts should be sized in 50 mm increments with a minimum shaft diameter of 400 mm.

3.10.4.3 Ultimate geotechnical capacity of bored pile for axial load

The basic concept of ultimate bearing capacity and useful equations for axial load capacity are identical to that of driven pile as described in Art. 3.10.1.10.

3.10.4.4 Axial capacity of bored piles in cohesive soil using static bearing capacity equations

The ultimate axial capacity of bored piles in cohesive may be calculated from the same static formula as used for driven piles, given by Equations 6.3.14a, 6.3.14b and 6.3.15, using a total stress method for undrained loading conditions, or an effective stress method for drained loading conditions. The skin friction f_s may be taken as $2/3^{rd}$ the value of driven piles and the end bearing f_b may be taken as $1/3^{rd}$ of that of driven pile.

3.10.4.5 Axial capacity of bored piles in cohesive soil using SPT values

The following relations may be used for preliminary design of ultimate capacity of concrete bored piles in clay soils.

For skin friction the relationship is as under.

$$f_s = 1.2\overline{N}_{60} \quad \text{(in kPa)} \quad \leq 70 \text{ kPa} \qquad (6.3.29)$$

For end bearing, the relationship is as under.

$$f_b = 25N_{60} \quad \text{(in kPa)} \quad \leq 4000 \text{ kPa} \qquad (6.3.30)$$

Where, $\overline{N}_{60}$ is the average N-value over the pile shaft length and N_{60} is the N-value in the vicinity of pile tip. A higher factor of safety of 3.5 should be used to estimate allowable capacity.

3.10.4.6 Axial capacity of bored piles in cohesionless soil using static bearing

The ultimate axial capacity of bored piles in cohesive soil may be calculated from the same static formula as used for driven piles described in Sec 3.10.1.10. The skin friction f_s may be taken as $2/3^{rd}$ the value of driven pile and the end bearing f_b may be taken as $1/3^{rd}$ of driven pile.

<u>Critical Depth for End Bearing and Skin Friction</u>

Similar to driven piles, following approximations may be used for the critical depth in relation to pile diameter, D.

$D_c = 10D$ for loose sand

$D_c = 15D$ for medium dense sand

$D_c = 20D$ for dense sand

3.10.4.7 Axial capacity of bored piles in cohesionless soil using SPT values

The following relations may be used for preliminary design of ultimate capacity of concrete bored piles in sand and non-plastic silty soils.

For skin friction the relationship is as under :

For sand

$$f_s = 1.0\bar{N}_{60} \quad \text{(in kPa)} \quad \leq 60 \text{ kPa} \qquad (6.3.31)$$

For non-plastic silt:

$$f_s = 0.9\bar{N}_{60} \quad \text{(in kPa)} \quad \leq 60 \text{ kPa} \qquad (6.3.32)$$

For end bearing, the relationship is as under.

For sand

$$f_b = 15N_{60}\left(\frac{L}{D}\right) \quad \text{(in kPa)} \quad \leq 150N_{60} \text{ and} \leq 4000 \text{ kPa} \qquad (6.3.33)$$

For non-plastic silt:

$$f_b = 10N_{60}\left(\frac{L}{D}\right) \quad \text{(in kPa)} \quad \leq 100N_{60} \text{ and} \leq 4000 \text{ kPa} \qquad (6.3.34)$$

Where, $\overline{N}_{60}$ is the average N-value over the pile shaft length and N_{60} is the N-value in the vicinity of pile tip (down to a depth of 3D). A higher factor of safety of 3.5 should be used to estimate allowable capacity.

3.10.4.8 Axial capacity of bored pile using pile load test

The procedures and principles of pile load test for ultimate capacity are similar to that of driven piles.

3.10.4.9 Structural capacity of bored concrete pile/drilled shaft

Minimum Reinforcement in Bored Concrete Pile

For piles loaded in compression alone, it is generally only necessary to reinforce the shaft to a depth of 2 m greater than the depth of temporary casing to prevent any tendency for concrete lifting when pulling the casing. Piles subject to tension or lateral forces and eccentric loading (possibly being out of position or out of plumb) do however require reinforcement suitable to cope with these forces. The following criteria for typical nominal reinforcement for piles in compression shall be considered. Table 6.3.11 may be used as guidelines. The restrictions that apply to the use of this Table have to be carefully considered in any particular application.

Table 6.3.11: Guidance on the Minimum Reinforcing Steel for Bored Cast-in-place Piles

Pile Diameter (mm)	Main Reinforcement		Lateral (Hoop) Reinforcement	
	Bar Size (mm)	No. of Bars	Bar Size (mm)	Pitch (mm)
400	16	6	8	200
450	16	6	8	200
500	16	8	8	250
600	16	8	8	250
750	16	10	10	300
900	16	10	10	300
1050	16	12	10	300
1200	16	12	10	300
1500	20	12	10	400
1800	20	12	10	400
2100	20	16	10	400
2400	25	16	12	500

Notes:

(a) Yield strength of steel = 420 MN/m^2

(b) The above guidelines are for "build-ability" only: They are not appropriate Where:

(i) Piles are required to resist any applied tensile or bending forces- the reinforcement has to be designed for the specific loading conditions.

(ii) Piles are required to accommodate positional and verticality tolerances, or where they are constructed through very soft alluvial deposits (c_u < 10 kN/m^2). Specific reinforcement design is then necessary.

(c) Minimum depth of reinforcement is taken as 3 m below cutoff for simple bearing only. Any lateral loads or moments taken by the pile will require reinforcement to extend to some depth below the zone subjected to bending forces. This zone may be determined from a plot of the bending moment with depth. Furthermore the reinforcement would normally extend at least 1 m below the depth of any temporary casing.

(d) Even with the appropriate reinforcement care will still be required to prevent damage to piles by construction activities especially during cutting-down or in the presence of site traffic.

The longitudinal reinforcement shall be of high yield steel bars (min f_y = 420 Mpa) and shall not be less than:

- 0.5% of A_c for $A_c \leq 0.5$ m^2;
- 0.375% of A_c for 0.5 m^2 < $A_c \leq 1$ m^2;
- 0.25% of A_c for $A_c > 1.0$ m^2;

Where, A_c is the gross cross-sectional area of the pile. The minimum diameter for the longitudinal bars should not be less than 16 mm for large diameter (diameter ≥ 600 mm) piles. Piles should have at least 6 longitudinal bars.

The assembled reinforcement cage should be sufficiently strong to sustain lifting and lowering into the pile bore without permanent distortion or displacement of bars or in addition bars should not be so densely packed that concrete aggregate cannot pass freely between them. Hoop reinforcement (for shear) is not recommended closer than 100 mm centres. Minimum Concrete cover to the reinforcement periphery shall be 75 mm. This guidance is only applicable for piles with vertical load.

Minimum Grades of Concrete

The integrity of pile shaft is of paramount importance, and the concreting mixes and methods that have been evolved for bored piles are directed towards this as opposed to the high strength concrete necessary for precast piles or structural work above ground. This prerequisite has led to the adoption of highly workable mixes, and the “total collapse” mix for tremie piles has been mentioned. In order to ensure that the concrete flows between the reinforcing bars with ease, and into the interstices of the soil, a high slump, self-compacting mix is called for. A minimum cement content of 350 kg/m^3 is generally employed under dry placement condition, increasing to 400 kg/m^3 under submerged condition at slumps greater than 125 mm, with a corresponding increase in fine aggregate content to maintain the cohesion of the mix. The water cement ratio in all cases is recommended as 0.45. Three mixes as recommended are given in Table 6.3.12.

Table 6.3.12: Recommended Concrete Slumps for Cast-in-place Bored Piles

Mix	Slump (mm)	Conditions of use
A	125	Poured into water-free unlined bore. Widely spaced reinforcement leaving ample room for free movement of the concrete between bars

Mix	Slump (mm)	Conditions of use
B	150	Where reinforcement is not placed widely enough to give free movement of concrete between bars. Where cutoff level of concrete is within casing. Where pile diameter is < 600 mm.
C	200	Where concrete is to be placed by tremie under water or bentonite in slurry.

3.10.4.10 Selection of factor of safety for bored pile

Selection of factor of safety for axial capacity of bored pile is similar to that used for driven piles.

3.10.4.11 Group capacity of bored pile

The behavior of group bored piles is almost similar to that of driven piles. For the pile cap, lateral load capacity, vertical ground movement, negative skin friction, piles in expansive soil, dynamic and seismic design, corrosion protection, dynamic monitoring and buoyancy. Sec 3.10.1.18 should be consulted as they are similar for both driven and bored piles. However, Individual bored piles are considered stable if the pile tops are laterally braced in two directions by construction, such as a structural floor slab, grade beams, struts, or walls. Generally, the use of a single pile as foundation is not recommended unless the diameter is 600 mm or more.

3.10.5 Settlement of Driven and Bored Piles

The settlement of axially loaded piles and pile groups at the allowable loads shall be estimated. Elastic analysis, load transfer and/or finite element techniques may be used. The settlement of the pile or pile group shall not exceed the tolerable movement limits as recommended for shallow foundations (Table 6.3.7). When a pile is loaded, two things would happen involving settlement.

- The pile would settle into the soil
- The pile material would compress due to load

The settlement of a single pile can be broken down into three distinct parts.

- Settlement due to axial deformation, S_{ax}
- Settlement at the pile tip, S_{pt}
- Settlement due to skin friction, S_{sf}

$$S_{t(Single)} = S_{ax} + S_{pt} + S_{sf} \tag{6.3.35a}$$

Moreover, piles acting in a group could undergo long term consolidation settlement.

Settlement due to axial deformation of a single pile can be estimated as :

$$S_{ax} = \frac{(Q_p + aQ_s)L}{AE_P} \quad (6.3.35b)$$

Where, Q_p = Load transferred to the soil at tip level

Q_s = Total skin friction load

L = Length of the pile

A = Cross section area of the pile

E_P = Young's modulus of pile material

a = 0.5 for clay and silt soils

= 0.67 for sandy soil

Pile tip settlement, S_{pt} can be estimated as :

$$S_{pt} = \frac{C_p Q_p}{Dq_o} \quad (6.3.35c)$$

Where, Q_p = Load transferred to the soil at tip level

D = Diameter of the pile

q_o = Ultimate end bearing capacity

C_p = Empirical coefficient as given in Table 6.3.13

Table 6.3.13: Typical Values of C_p for Settlement Calculation of Single Pile

Soil Type	Values of C_p	
	Driven Pile	**Bored Pile**
Dense Sand	0.02	0.09
Loose Sand	0.04	0.18
Stiff Clay	0.02	0.03
Soft Clay	0.03	0.06
Dense Silt	0.03	0.09
Loose Silt	0.05	0.12

Skin friction acting along the shaft would stress the surrounding soil. Skin friction acts upward direction along the pile. The force due to pile on surrounding soil would be in downward direction. When the pile is loaded, the pile would slightly move down. The pile would drag the surrounding soil with it. Hence, the pile settlement would occur due to skin friction as given by :

$$S_{sf} = \frac{C_s Q_s}{D q_o} \qquad (6.3.36)$$

Where, C_s = Empirical coefficient = $\left(0.93 + 0.16\frac{L}{D}\right) C_p$

C_p = Empirical coefficient as given in Table 6.3.9

Q_s = Total skin friction load

D = Diameter of the pile

q_o = Ultimate end bearing capacity

<u>Short Term Pile Group Settlement</u>

Short term or elastic pile group settlement can be estimated using the following relation.

$$S_g = S_{t(single)} \left(\frac{B}{D}\right)^{0.5} \qquad (6.3.37)$$

Where,

S_g = Settlement of the pile group

$S_{t(single)}$ = Total settlement of a single pile

B = Smallest dimension of the pile group

D = Diameter of the pile

Interestingly, geometry of the group does not have much of an influence on the settlement. As such, Group Settlement Ratio, R_s of a pile group consisting of n number of piles can be approximated as follows :

$$R_s = \frac{S_g}{S_{t(single)}} = (n)^{0.5} \qquad (6.3.38)$$

The settlement of the group can be estimated as the highest value as obtained from Equations 6.3.37 and 6.3.38.

Long Term Settlement for Pile Group

For pile groups, settlement due to consolidation is more important than for single piles. Consolidation settlement of pile group in clay soil is computed using the following simplified assumptions.

- The pile group is assumed to be a solid foundation with a depth 2/3rd the length of the piles
- Effective stress at mid-point of the clay layer is used to compute settlement

If soil properties are available, the consolidation settlement (S) may be obtained from the following equation. The depth of significant stress increase (10%) or the depth of bed rock whichever is less should be taken for computation of settlement. Stress distribution may be considered as 2 vertical to 1 horizontal.

$$S = \frac{C_c H}{1+e_o} log \frac{\sigma_o' + \sigma_p'}{\sigma_o'} \qquad (6.3.39)$$

Where,

C_c = Compression index of soil

e_o = initial void ratio

H = Thickness of the clay layer

σ_o' = Initial effective stress at mid-point of the clay layer

σ_p'= Increase in effective stress at mid-point of the clay layer due to pile load.

In absence of soil properties the following empirical equations may be used to estimate the long term consolidation settlement of clay soils.

For clay:

$$S = \frac{H}{M} \mathrm{Ln} \left(\frac{\sigma_1'}{\sigma_o'}\right)^j \qquad (6.3.40)$$

For sand:

$$S = \frac{2H}{M} \left[\left(\frac{\sigma_1'}{\sigma_r'}\right)^j - \left(\frac{\sigma_o'}{\sigma_r'}\right)^j\right] \qquad (6.3.41)$$

Where,

H = Thickness of the clay layer

σ'_o = Initial effective stress at mid-point of the clay layer

σ'_1 = New effective stress at mid-point of the clay layer after pile load.

σ'_r = Reference stress (100 kPa)

M = Dimensionless modulus number as obtained from Table 6.3.14

j = Stress exponent as obtained from Table 6.3.14.

Table 6.3.14: Settlement Parameters

Soil	Density	Modulus Number, M	Stress Exponent, j
Till	V. Dense to Dense	1000 - 300	1.0
Gravel	-	400 - 40	0.5
Sand	Dense	400 - 250	0.5
Sand	Medium Dense	250 - 150	0.5
Sand	Loose	150 - 100	0.5
Silt	Dense	200 - 80	0.5
Silt	Medium Dense	80 - 60	0.5
Silt	Loose	60 - 40	0.5
Silty Clay	Stiff	60 - 40	0.5
Silty Clay	Medium Stiff	20 - 10	0.5
Silty Clay	Soft	10 - 5	0.5
Marine Clay	Soft	20 - 5	0.0
Organic Clay	Soft	20 - 5	0.0
Peat	-	5 - 1	0.0

3.10.6 Drilled Shafts/ Drilled Piers

Large diameter (more than 600 mm) bored piles are sometimes classified as drilled shaft or drilled piers. They are usually provided with enlarged base called bell. The provisions of this article shall apply to the design of axially and laterally loaded drilled shafts/ drilled piers in soil or extending through soil to or into rock.

3.10.6.1 Application of drilled shaft

Drilled shafts may be considered when spread footings cannot be founded on suitable soil within a reasonable depth and when piles are not economically viable due to high loads or obstructions to driving. Drilled shafts may be used in lieu of spread footings as a protection against scour. Drilled shafts may also be considered to resist high lateral or uplift loads when deformation tolerances are small.

3.10.6.2 Materials for drilled shaft

Shafts shall be cast-in-place concrete and may include deformed bar steel reinforcement, structural steel sections, and/or permanent steel casing as required by design.

3.10.6.3 Embedment for Drilled Shaft

Shaft embedment shall be determined based on vertical and lateral load capacities of both the shaft and sub-surface materials.

3.10.6.4 Batter drilled shaft

The use of battered shafts to increase the lateral capacity of foundations is not recommended due to their difficulty of construction and high cost. Instead, consideration should first be given to increasing the shaft diameter to obtain the required lateral capacity.

3.10.6.5 Selection of soil properties for drilled shaft

Soil and rock properties defining the strength and compressibility characteristics of the foundation materials are required for drilled shaft design.

3.10.6.6 Geotechnical design of drilled shafts

Drilled shafts shall be designed to support the design loads with adequate bearing and structural capacity, and with tolerable settlements. The response of drilled shafts subjected to seismic and dynamic loads shall also be evaluated. Shaft design shall be based on working stress principles using maximum un-factored loads derived from calculations of dead and live loads from superstructures, substructures, earth (i.e., sloping ground), wind and traffic. Allowable axial and lateral loads may be determined by separate methods of analysis.

The design methods presented herein for determining axial load capacity assume drilled shafts of uniform cross section, with vertical alignment, concentric axial loading, and a relatively horizontal ground surface. The effects of an enlarged base, group action, and sloping ground are treated separately.

3.10.6.7 Bearing capacity equations for drilled shaft

The ultimate axial capacity Q_{ult} of drilled shafts shall be determined in accordance with the principles laid for bored piles.

Cohesive Soil

Skin friction resistance in cohesive soil may be determined using either the α-method or the β-method as described in the relevant section of driven piles. However, for clay soil, α-method has wide been used by the engineers. This method gives:

$$f_s = \alpha s_u \qquad (6.3.42)$$

Where,

f_s = Skin friction

s_u = undrained shear strength of soil along the shaft

α = adhesion factor =0.55 for undrained shear strength ≤ 190 kPa (4000 psf)

For higher values of s_u the value of α may be taken from Figure 6.3.3 as obtained from test data of previous investigators.

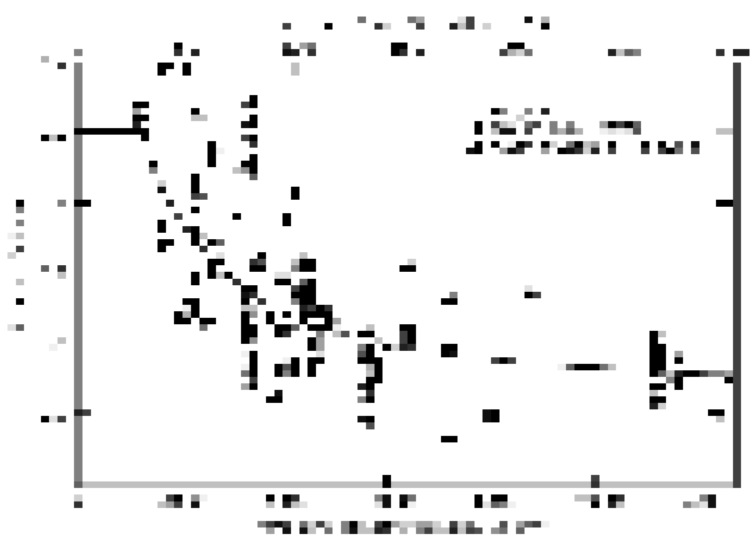


Figure 6.3.3 Adhesion factor α for drilled shaft (after Kulhawy and Jackson, 1989)

The skin friction resistance should be ignored in the upper 1.5 m of the shaft and along the bottom one diameter of straight shafts because of interaction with the end bearing. If end bearing is ignored for some reasons, the skin friction along the bottom one diameter may be considered. For belled shaft, skin friction along the surface of the bell and along the shaft for a distance of one shaft diameter above the top of bell should be ignored. For end bearing of cohesive soil, the following relations given by Equations 6.3.43 and 6.3.44 are recommended.

$$f_b = N_c S_u \leq 4000 \text{ kPa} \tag{6.3.43}$$

Where, $N_c = 6\left[1 + 0.2\left(\frac{L}{D_b}\right)\right] \leq 9$

Where,

f_b = End bearing stress

S_u = undrained shear strength of soil along the shaft

N_c = Bearing capacity factor

L = Length of the pile (Depth to the bottom of the shaft)

D_b = Diameter of the shaft base

If the base diameter is more than 1900 mm, the value of f_b from Eq. 6.3.43 could produce settlements greater than 25 mm, which would be unacceptable for most buildings. To keep settlement within tolerable limits, the value of f_b should be reduced to $f_b^{'}$ by multiplying a factor F_r such that:

$$f_b^{'} = F_r f_b \tag{6.3.44a}$$

$$F_r = \frac{2.5}{120\, \omega_1\, D_b/B_r + \omega_2} \leq 1.0 \tag{6.3.44b}$$

$$\omega_1 = 0.0071 + 0.0021\left(\frac{L}{D_b}\right) \leq 0.0015 \tag{6.3.44c}$$

$$\omega_2 = 1.59\sqrt{\frac{S_u}{\sigma_r}} \quad 0.5 \leq \omega 2 \leq 1.5 \tag{6.3.44d}$$

Where,

B_r = Reference width=1 ft = 0.3 m = 12 inch = 300 mm

σ_r = Reference stress = 100 kPa = 2000 psf

Cohesionless Soil

Skin friction resistance in cohesionless soil is usually determined using the β-method. The relevant equation is reproduced again:

$$f_s = \beta\sigma_z' \qquad (6.3.45)$$

$$\beta = Ktan\phi_s \qquad (6.3.46)$$

Where,

f_s = Skin friction

σ_z' = Effective vertical stress at mid-point of soil layer

K = Coefficient of lateral earth pressure

ϕ_s = Soil shaft interface friction angle

The values of K and ϕ_s can be obtained from the chart of Tables 6.3.15, from the soil friction angle, ϕ and preconstruction coefficient of lateral earth pressure K_o. However, K_o is very difficult to determine. An alternative is to compute β directly using the following empirical relation.

$$\beta = 1.5 - 0.135\sqrt{\frac{z}{B_r}} \qquad (6.3.47)$$

Where,

B_r = Reference width=1 ft = 0.3 m = 12 inch = 300 mm

z = Depth from the ground surface to the mid-point of the strata

Table 6.3.15: Typical ϕ_s/ϕ *and* K/K_o Values for the Design of Drilled Shaft

Construction Method	ϕ_s/ϕ	**Construction Method**	K/K_o
Open hole or temporary casing	1.0	Dry construction with minimal side wall disturbance and prompt concreting	1
Slurry method – minimal slurry cake	1.0	Slurry construction – good workmanship	1
Slurry method – heavy slurry cake	0.8	Slurry construction – poor workmanship	2/3
Permanent casing	0.7	Casing under water	5/6

The unit end bearing capacity for drilled shaft in cohesionless soils will be less than that for driven piles because of various reasons like soil disturbance during augering, temporary stress relief while the hole is open, larger diameter and depth of influence etc. The reasons are not well defined, as such the following empirical formula developed by Reese and O' Nell (1989) may be suggested to use to estimate end bearing stress.

$$f_b = 0.60\sigma_r N \leq 4500 \text{ kPa} \qquad (6.3.48)$$

Where,

f_b = Unit bearing resistance

σ_r' = Reference stress = 100 kPa = 2000 psf

N = Mean SPT value for the soil between the base of the shaft and a depth equal to two times the base diameter below the base. No overburden correction is required (N= N_{60})

If the base diameter is more than 1200 mm, the value of f_b from Eq. 6.3.48 could produce settlements greater than 25 mm, which would be unacceptable for most buildings. To keep settlement within tolerable limits, the value of f_b should be reduced to f_b' by multiplying a factor F_r such that:

$$f_b' = F_r f_b \qquad (6.3.49a)$$

$$F_r = 4.17\frac{B_r}{D_b} \leq 1.0 \qquad (6.3.49b)$$

Where,

B_r = Reference width=1 ft = 0.3 m = 12 inch = 300 mm

D_b =Base diameter of drilled shaft

3.10.6.8 Other methods of evaluating axial load capacity of drilled shaft

A number of other methods are available to estimate the ultimate axial load capacity of drilled shafts. These methods are based on N-values obtained from Standard Penetration Test (SPT) and on angle of internal friction of sand. These methods may also be used to estimate the ultimate load carrying capacity of drilled shafts. Three of these methods are as follows and they are summarized in Appendix G.

- Method based on the Standard Penetration Test (CGS, 1985)
- Method based on Theory of Plasticity (CGS, 1985)
- Tomlinson (1995) Method

3.10.6.9 Factor of safety for drilled shaft

Similar to bored and driven piles, drilled shafts shall be designed for a minimum overall factor of safety of 2.0 against bearing capacity failure (end bearing, side resistance or combined) when the design is based on the results of a load test conducted at the site. Otherwise, it shall be designed for a minimum overall factor of safety 3.0. The minimum recommended overall factor of safety is based on an assumed normal level of field quality control during construction. If a normal level of field quality control cannot be assured, higher minimum factors of safety shall be used. The recommended values of overall factor of safety on ultimate axial load capacity based on specified construction control is presented in Tables 6.3.10a and 6.3.10b.

3.10.6.10 Deformation and settlement of axially loaded drilled shaft

Similar to driven and bored piles, settlement of axially loaded shafts at working or allowable loads shall be estimated using elastic or load transfer analysis methods. For most cases, elastic analysis will be applicable for design provided the stress levels in the shaft are moderate relative to Q_{ult}. Analytical methods are similar to that provided in Sec 3.10.1.10 for driven and bored piles. The charts provided in Appendix G may also be used to estimate the settlement of drilled shaft.

3.10.6.11 Drilled shaft in layered soil profile

The short-term settlement of shafts in a layered soil profile may be estimated by summing the proportional settlement components from layers of cohesive and cohesionless soil comprising the subsurface profile.

3.10.6.12 Tolerable movement of drilled shaft

Tolerable axial displacement criteria for drilled shaft foundations shall be developed by the structural designer consistent with the function and type of structure, fixity of bearings, anticipated service life, and consequences of unacceptable displacements on the structure performance. Drilled shaft displacement analyses shall be based on the results of in-situ/laboratory testing to characterize the load-deformation behavior of the foundation materials.

3.10.6.13 Group loading of drilled shaft

Cohesive Soil

Evaluation of group capacity of shafts in cohesive soil shall consider the presence and contact of a cap with the ground surface and the spacing between adjacent shafts.

For a shaft group with a cap in firm contact with the ground, Q_{ult} may be computed as the lesser of (1) the sum of the individual capacities of each shaft in the group or (2) the capacity of an equivalent pier defined in the perimeter area of the group. For the equivalent pier, the shear strength of soil shall not be reduced by any factor (e.g., α_1) to determine the Q_s component of Q_{ult}, the total base area of the equivalent pier shall be used to determine the Q_T component of Q_{ult} and the additional capacity of the cap shall be ignored. If the cap is not in firm contact with the ground, or if the soil at the surface is loose or soft, the individual capacity of each shaft should be reduced to ζ times Q_T for an isolated shaft, where $\zeta = 0.67$ for a center-to-center (CTC) spacing of 3B (where B is the shaft diameter) and $\zeta = 1.0$ for a CTC spacing of 6B. For intermediate spacings, the value of ζ may be determined by linear interpolation. The group capacity may then be computed as the lesser of (1) the sum of the modified individual capacities of each shaft in group, or (2) the capacity of an equivalent pier as stated above.

Cohesionless Soil

Evaluation of group capacity of shafts in cohesion soil shall consider the spacing between adjacent shafts. Regardless of cap contact with the ground, the individual capacity of each shaft should be reduced to times Q_T for an isolated shaft, where $\zeta = 0.67$ for a center-lo-center (CTC) spacing of 3B and $\zeta = 1.0$ for a CTC spacing of 8B. For intermediate spacings, the value of ζ may be determined by linear interpolation. The group capacity may be computed as the lesser of (I) sum of the modified individual capacities of each shaft in the group or (2) capacity of an equivalent pier circumscribing the group including resistance over the entire perimeter and base areas.

3.10.6.14 Drilled shaft in strong soil overlying weak soil

If a group of shafts is embedded in a strong soil deposit which overlies a weaker deposit (cohesionless and cohesive soil), consideration shall be given to the potential for a punching failure of the lip into the weaker soil strata. For this case, the unit tip capacity q_E of the equivalent shaft may be determined using the following:

$$q_E = \frac{HB_r}{10}(q_{UP} - q_{Lo}) \leq q_{UP} \qquad (6.3.50)$$

In the above equation q_{UP} is the ultimate unit capacity of an equivalent shaft bearing in the stronger upper layer and q_{Lo} is the ultimate unit capacity of an equivalent shaft bearing in the weaker underlying soil layer. If the underlying soil unit is a weaker cohesive soil strata, careful consideration shall be given to the potential for large settlements in the weaker layer.

3.10.6.15 Lateral loads on drilled shaft

Soil Layering

The design of laterally loaded drilled shafts in layered soils shall be based on evaluation of the soil parameters characteristic of the respective layers

Ground Water

The highest anticipated water level shall be used for design

Scour

The potential for loss of lateral capacity due to scour shall be considered in the design. If heavy scour is expected, consideration shall be given to designing the portion of the shaft that would be exposed as a column. In all cases, the shaft length shall be determined such that the design structural load can be safely supported entirely below the probable scour depth.

Group action

There is no reliable rational method for evaluating the group action for closely spaced, laterally loaded shafts. Therefore, as a general guide, drilled shaft with diameter B in a group may be considered to act individually when the center-to-center (CTC) spacing is greater than 2.5B in the direction normal to loading, and CTC > 8B in the direction parallel to loading. For shaft layout not conforming to these criteria, the effects of shaft interaction shall be considered in the design. As a general guide, the effects of group action for in-line CTC <8B may be considered using the ratios (CGS, 1985) appearing as below, Table 6.3.16:

Table 6.3.16: Ratio of Group and Single Plie Shaft Resistance

Centre to Centre Shaft Spacing for In-line Loading	Ratio of Lateral Resistance of Shaft in Group to Single Shaft
8B	1.00
6B	0.70
4B	0.40
3B	0.25

Cyclic Loading

The effects of traffic, wind, and other non-seismic cyclic loading on the load-deformation behavior of laterally loaded drilled shafts shall be considered during design. Analysis of drilled shafts subjected to cyclic loading may he considered in the COM624 analysis (Reese et. al., 1984).

Combined Axial and Lateral Loading

The effects of lateral loading in combination with axial loading shall be considered in the design. Analysis of drilled shafts subjected to combined loading may be considered in the COM624 analysis (Reese et. al., 1984).

Sloping Ground

For drilled shafts which extend through or below sloping ground. The potential for additional lateral loading shall be considered in the design. The general method of analysis developed by Borden and Gabr (1987) may be used for the analysis of shafts instable slopes. For shafts in marginally stable slopes. Additional consideration should be given for smaller factors of safety against slope failure or slopes showing ground creep, or when shafts extend through fills overlying soft foundation soils and bear into more competent underlying soil or rock formations. For unstable ground, detailed explorations, testing and analysis are required to evaluate potential additional lateral loads due to slope movements

Tolerable Lateral Movements

Tolerable lateral displacement criteria for drilled shaft foundations shall be developed by the structural designer consistent with the function and type of structure, fixity, anticipated service life, and consequences of unacceptable displacements on the structure performance. Drilled shaft lateral displacement analysis shall be based on the results of in-situ and/or laboratory testing to characterize the load-deformation behavior of the foundation materials.

3.10.6.16 Uplift loads on drilled shaft

Uplift capacity shall rely only on side resistance in conformance with related articles for driven piles. If the shaft has an enlarged base, Q_S shall be determined in conformance with related articles for driven piles.

3.10.6.17 Consideration of vertical ground movement

The potential for external loading on a shaft by vertical ground movement (i.e., negative skin friction down-drag due to settlement of compressible soil or uplift due to heave of expansive soil) shall be considered as a part of design. For design purposes, it shall be assumed that the full magnitude of maximum potential vertical ground movement occurs.

3.10.6.18 Negative skin friction

Evaluation of negative skin friction shall include a load-transfer method of analysis to determine the neutral point (i.e., point of zero relative displacement) and load distribution along shaft (e.g., Reese and O'Neill, 1988). Due to the possible time dependence associated with vertical ground movement, the analysis shall consider the effect of time on load transfer between the ground and shaft and the analysis shall be performed for the time period relating to the maximum axial load transfer to the shaft. Evaluation of negative skin friction shall include a load-transfer method of analysis to determine the neutral point (i.e., point of zero relative displacement) and load distribution along shaft (e.g., Reese and O'Neill, 1988). Due to the possible time dependence associated with vertical ground movement, the analysis shall consider the effect of time on load transfer between the ground and shaft and the analysis shall be performed for the time period relating to maximum axial load transfer to the shaft.

3.10.6.19 Expansive soils

Shafts designed for and constructed in expansive soil shall extend to a sufficient depth into moisture-stable soils to provide adequate anchorage to resist uplift movement in addition; sufficient clearance shall be provided between the ground surface and underside of caps or beams connecting shafts to preclude the application of uplift loads at the shaft/cap connection from swelling ground conditions.

3.10.6.20 Dynamic/seismic design of drilled shaft

Refer to Seismic Design section of this Code and Lam and Martin (1986a; 1986b) for guidance regarding the design of drilled shafts subjected to dynamic and seismic loads.

3.10.6.21 Structural shaft design, shaft dimensions and shaft spacing

Drilled shafts shall be designed to resist failure loads to insure that the shaft will not collapse or suffer loss of serviceability due to excessive stress and/or deformation.

Dimensions

All shafts should be sized in 50 mm increments with a minimum shaft diameter of 600 mm. The diameter of columns supported by shafts shall be less than or equal to the shaft diameter B.

Center to Center Spacing

The center-to-center spacing of drilled shafts of diameter B should be 3B or greater to avoid interference between adjacent shafts during construction. If closer spacing is required, the sequence of construction shall be specified and the interaction effects between adjacent shafts shall be evaluated by the designer.

Reinforcement

Where the potential for lateral loading is insignificant, drilled shafts need to be reinforced for axial loads only. Those portions of drilled shafts that are not supported laterally shall be designed as reinforced concrete columns in accordance with relevant sections in structural design part of the Code and the reinforcing steel shall extend a minimum of 5 m below the plane where the soil provides adequate lateral restraint. Where permanent steel casing is used and the shell is smooth pipe and more than 3 mm in thickness, it may be considered as load carrying in the absence of corrosion.

The design of longitudinal and spiral reinforcement shall be in conformance with the requirements of the relevant sections of the structural design part of the Code. Development of length of deformed reinforcement shall be in conformance with the relevant sections of the structural design part of the Code.

Longitudinal Bar Spacing

The minimum clear distance between longitudinal reinforcement shall not be less than 3 times the bar diameter nor 3 times the maximum aggregate size. If bars are bundled in forming the reinforcing cage, the minimum clear distance between longitudinal reinforcement shall not be less than 3 times the diameter of the bundled bars. Where heavy reinforcement is required, consideration may be given to an inner and outer reinforcing cage.

Splices

Splices shall develop the full capacity of the bar in tension and compression. The location of splices shall be staggered around the perimeter of the reinforcing cage so as not to occur at the same horizontal plane. Splices may be developed by lapping, welding, and special approved connectors. Splices shall be in conformance with the relevant sections of the structural design part of the Code.

Transverse Reinforcement

Transverse reinforcement shall be designed to resist stresses caused by fresh concrete flowing from inside the cage to the side of the excavated hole. Transverse reinforcement may be constructed of hoops or spiral steel.

Handling Stresses

Reinforcement cages shall be designed to resist handling and placement stresses.

Reinforcement Cover

The reinforcement shall be placed a clear distance of not less than 50 mm from the permanently cased or 75 mm from the uncased sides. When shafts are constructed in corrosive or marine environments, or when concrete is placed by the water or slurry displacement methods, the clear distance shall not be less than 100 mm for uncased shafts and shafts with permanent casings not sufficiently corrosion resistant.

The reinforcement cage shall be centered in the hole using centering devices. All steel centering devices shall be epoxy coated.

Reinforcement into Superstructure

Sufficient reinforcement shall be provided at tit junction of the shaft with the superstructure to make a suitable connection. The embedment of the reinforcement into the cap shall be in conformance with relevant articles of the structural design part of the Code.

3.10.6.22 Enlarged base of drilled shaft

Enlarged bases shall be designed to insure that plain concrete is not overstressed. The enlarged base shall slope at a side angle not less than 30 degrees from the vertical and have a bottom diameter not greater than 3 times diameter of the shaft. The thickness of the bottom edge of enlarged base shall not be less than 150 mm.

3.10.6.23 Construction of drilled shaft

Drilled shafts may be constructed using the dry, casing, or wet method of construction, or a combination of methods. In every case, excavation of hole, placement of concrete, and all other aspects of shaft construction shall be performed in conformance with the provisions of this Code.

The load capacity and deformation behavior of drilled shafts can be greatly affected by the quality and methods of construction. The effects of construction methods are incorporated in design by application of factor of safety consistent with the expected construction methods and level of field quality control measures undertaken as described in the relevant sections for driven piles.

Where the spacing between shafts in a group is restricted, consideration shall be given to the sequence of construction to minimize the effect of adjacent shaft construction operations on recently constructed shafts. The following construction procedure shall be followed:

(i) Place permanent/temporary steel casing in position and embed casing toe into firm strata.

(ii) Bore and excavate inside the steel casing down to casing toe level, or to a level approved, and continue excavation to final pile tip level using drilling mud. The fluid level inside casings shall at all times be at least 2 metres higher than outside the casings.

(iii) Carefully clean up all mud or sedimentation from the bottom of borehole.

(iv) Place reinforcement cage, inspection pipes etc.

(v) Concrete continuously under water, or drilling fluid, by use of the tremie method.

(vi) After hardening, break out the top section of the concrete pile to reach sound concrete.

In drilling of holes for all piles, bentonite and any other material shall be mixed thoroughly with clean water to make a suspension which shall maintain the stability of the pile excavation for the period necessary to place concrete and complete construction. The control tests shall cover the determination of' density, viscosity, gel strength and pH values. Bentonite slurry shall meet the Specifications as shown in Table 6.3.17.

Table 6.3.17: Specifications of Bentonite Slurry

Item to be Measured	Range of Results at 20° C	Test Method
Density during drilling to support excavation	greater than 1.05 g/ml	Mud density Balance (ASTM D4380)
Density prior to concreting	less than 1.25 g/ml	Mud density Balance (ASTM D4380)
Viscosity	30 - 90 seconds	Marsh Cone Method (ASTM D6910)
pH	9.5 to 12	pH indicator paper strips or electrical pH meter (ASTM D4972)
Liquid limit	> 450%	Casagrande apparatus (ASTM D4318)

Temporary casing of approved quality or an approved alternative method shall be used to maintain the stability of pile excavations, which might otherwise collapse. Temporary casings shall be free from significant distortion.

Where a borehole is formed using drilling fluid for maintaining the stability of a boring, the level of the water or fluid in the excavation shall be maintained so that the water or fluid pressure always exceeds the pressure exerted by the soils and external ground water. The water or fluid level shall be maintained at a level not less than 2 m above the level of ground water.

The reinforcement shall be placed as indicated on the Drawings. Reinforcement in the form of a cage shall be assembled with additional support, such as Spreader forks and lacings, necessary to form a rigid cage. Hoops, links or helical reinforcement shall fit closely around the main longitudinal bars and be bound to them by approved wire, the ends of which shall be turned into the interior of the pile or pour. Reinforcement shall be placed and maintained in position. The cover to all reinforcement for pile cap and bored cast in place pile shall be not less than 75 mm.

Joints in longitudinal steel bars shall be permitted unless otherwise specified. Joints in reinforcement shall be such that the full strength of the bar is effective across the joint and shall be made so that there is no relative displacement of the reinforcement during the construction of the pile. Joints in longitudinal bars in piles with tension (for instance for test loading) shall be carried out by welding or other approved method.

Concrete to be placed under water or drilling fluid shall be placed by tremie equipment and shall not be discharged freely into the water or drilling fluid. The tremie equipment shall be designed to minimize the occurrence of entrapped air and other voids, so that it causes minimal surface disturbance, which is particularly important when a concrete-water interface exists. It shall be so designed that external projections are minimised, allowing the tremie to pass through reinforcing cages without causing damage. The internal face of the pipe of the tremie shall be free from projections. The tremie pipes shall meet the following requirements:

(i) The tremie pipes shall be fabricated of heavy gage steel pipe to withstand all anticipated handling stress. Aluminium pipe shall not be used for placing concrete.

(ii) Tremie pipes should have a diameter large enough to ensure that aggregates-caused blockage will not occur. The diameter of the tremie pipe shall be 200 mm to 300 mm.

(iii) The tremie pipes shall be smooth internally.

(iv) Since deep placement of concrete will be carried out, the tremie shall be made in sections/lengths with detachable joints that allow the upper sections/lengths to be removed as the placement progresses.

(v) Sections may be joined by flanged, bolted connections (with gaskets) or may be screwed together. Whatever joint technique is selected, joints between tremie sections must be watertight. The joint system selected shall be tested for water tightness before beginning of concrete placement.

(vi) The joint system to be used shall need approval of the Engineer.

(vii) The tremie pipe should be marked to allow quick determination of the distance from the surface of the water to the mouth of the tremie.

(viii) The tremie should be provided with adequately sized funnel or hopper to facilitate transfer of sufficient concrete from the delivery device to the tremie.

Before placing concrete, it shall be ensured that there is no accumulation of silt, other material, or heavily contaminated bentonite suspension at the base of the boring, which could impair the free flow of concrete from the pipe of the tremie. Flushing of boreholes before concreting with fresh drilling fluid/mud is preferred. A sample of the bentonite suspension shall be taken from the base of the boring using an approved sampling device. If the specific gravity of the suspension exceeds 1.25, the placing of concrete shall not proceed. In this event the Contractor shall modify the mud quality. During and after concreting, care shall be taken to avoid damage to the concrete from pumping and dewatering operations.

The hopper and pipe of the tremie shall be clean and watertight throughout. The pipe shall extend to the base of the boring and a sliding plug or barrier shall be placed in the pipe to prevent direct contact between the first charge of concrete in the pipe of the tremie and the water or drilling fluid. The pipe shall at all times penetrate the concrete, which has previously been placed and shall not be withdrawn from the concrete until completion of concreting. The bottom of the tremie pipe shall be embedded in the fresh concrete at least 2.0 m and maintained at that depth throughout concreting. At all times a sufficient quantity of concrete shall be maintained within the pipe to ensure that the pressure from it exceeds that from the water or drilling fluid.

To ensure the quality of concrete being free from mud, clay lumps or any other undesirable materials mixed with concrete at the top portion of the pile, fresh concrete shall be overflowed sufficiently at the end of the each pour. The level of concrete poured at the end of concreting operation shall be at least 600 mm higher than the elevation of the pile at cut-off.

3.10.6.24 Concreting of drilled shaft

In drilled shafts/cast-in-situ bored piles, concrete shall be placed only after excavation has been completed, inspected and accepted, and steel reinforcement accurately placed and adequately supported. Concrete shall be placed in one continuous operation in such a manner as to ensure the exclusion of any foreign matter and to secure a full sized shaft. Concrete shall not be placed through water except where tremie methods are approved. When depositing concrete from the top of the pile, the concrete shall not be chuted directly into the pile but shall be poured in a rapid and continuous operation through a funnel hopper centred at the top of the pile.

For large diameter holes concrete may be placed by tremie or by drop bottom bucket; for small diameter boreholes a tremie shall be utilized. In tremie concreting, toe of the tremie shall be set at a maximum of 150 mm above the bottom of the borehole. Maximum permissible siltation in bore hole prior to start of concrete operation shall be 75 mm. A slump of 125 mm to 150 mm shall be maintained for concreting by tremie. In case of tremie concreting for piles of smaller diameter and length up to 10 m, the minimum cement content shall be 350 kg/m^3 of concrete. For larger diameter and/or deeper piles, the minimum cement content shall be 400 kg/m^3 of concrete. See relevant sections of the Code for further specification.

For uncased concrete piles, if pile shafts are formed through unstable soil and concrete is placed in an open drill hole, a steel liner shall be inserted in the hole prior to placing concrete. If the steel liner is withdrawn during concreting, the level of concrete shall be maintained above the bottom of the liner to a sufficient height to offset any hydrostatic or lateral earth pressure.

If concrete is placed by pumping through a hollow stem auger, the auger shall not be permitted to rotate during withdrawal and shall be withdrawn in a steady continuous motion. Concrete pumping pressures shall be measured and shall be maintained high enough at all times to offset hydrostatic and lateral earth pressure. Concrete volumes shall be measured to ensure that the volume of concrete placed in each pile is equal to or greater than the theoretical volume of the hole created by the auger. If the installation process of any pile is interrupted or a loss of concreting pressure occurs, the hole shall be redrilled to original depth and reformed.

Augured cast-in-situ pile shall not be installed within 6 pile diameters centre to centre of a pile filled with concrete less than 24 hours old. If concrete level in any completed pile drops, the pile shall be rejected and replaced. Bored cast-in-situ concrete piles shall not be drilled/bored within a clear distance of 3 m from an adjacent pile with concrete less than 48 hours old. For under-reamed piles, the slump of concrete shall range between 100 mm and 150 mm for concreting in water free holes.

For concreting under water, the concrete shall contain at least 10 percent more cement than that required for the same mix placed in the dry. The amount of coarse aggregate shall be not less than one and a half times, nor more than two times, that of the fine aggregate. The materials shall be so proportioned as to produce a concrete having a slump of not less than 100 mm, nor more than 150 mm, except where plasticizing admixtures is used in which case, the slump may be 175 mm.

Successful placement of concrete under water requires preventing flow of water across or through the placement site. Once flow is controlled, the tremie placement consists of the following three basic steps:

(i) The first concrete placed is physically separated from the water by using a "rabbit" or go-devil in the pipe, or by having the pipe mouth capped or sealed and the pipe dewatered.

(ii) Once filled with concrete, the pipe is raised slightly to allow the "rabbit" to escape or to break the end seal. Concrete will then flow out and develop a mound around the mouth of the pipe. This is termed as "establishing a seal".

(iii) Once the seal is established, fresh concrete is injected into the mass of existing concrete.

Two methods are normally used for the placement of concrete using tremie pipe, namely, the capped tremie pipe approach and the "rabbit" plug approach. In the capped tremie approach the tremie pipe should have a seal, consisting of a bottom plate that seals the bottom of the pipe until the pipe reaches the bottom of excavation. The tremie pipe should be filled with enough concrete before being raised off the bottom. The tremie pipe should then be raised a maximum of 150 mm (6 inch) to initiate flow. The tremie pipe should not be lifted further until a mound is established around the mouth of the tremie pipe. Initial lifting of the tremie should be done slowly to minimize disturbance of material surrounding the mouth of the tremie.

In the “rabbit” plug approach, open tremie pipe should be set on the bottom, the “rabbit” plug inserted at the top and then concrete should be added to the tremie slowly to force the “rabbit” downward separating the concrete from the water. Once the tremie pipe is fully charged and the “rabbit” reaches the mouth of the tremie, the tremie pipe should be lifted a maximum of 150 mm (6 inch) off the bottom to allow the “rabbit” to escape and to start the concrete flowing. After this, a tremie pipe should not be lifted again until a sufficient mound is established around the mouth of the tremie.

Tremies should be embedded in the fresh concrete a minimum of 1.0 to 1.5 m (3 to 5 ft) and maintained at that depth throughout concreting to prevent entry of water into the pipe. Rapid raising or lowering of the tremie pipe should not be allowed. All vertical movements of the tremie pipe must be done slowly and carefully to prevent “loss of seal”. If “loss of seal” occurs in a tremie, placement of concrete through the tremie must be halted immediately. The tremie pipe must be removed and the end plate must be restarted using the capped tremie approach. In order to prevent washing of concrete in place, a “rabbit” plug approach must not be used to restart a tremie after “loss of seal”.

Means of raising or lowering tremie pipes and of removing pipes smoothly without loss of concrete and without disturbing placed concrete or trapping air in the concrete shall be provided. Pipes shall not be moved horizontally while they are embedded in placed concrete or while they have concrete within them.

Underwater concrete shall be placed continuously for the whole of a pour to its full depth approved by the Engineer, without interruption by meal breaks, change of shift, movements of placing positions, and the like. Delays in placement may allow the concrete to stiffen and resist flow once placement resumes. The rate of pour from individual tremie shall be arranged so that concrete does not rise locally to a level greater than 500 mm above the average level of the surrounding concrete.

Tremie blockages which occur during placement should be cleared extremely carefully to prevent loss of seal. If a blockage occurs, the tremie should be quickly raised 150 to 600 mm (6 inch to 2 ft) and then lowered in an attempt to dislodge the blockage. The depth of pipe embedment must be closely monitored during all such attempts. If the blockage cannot be cleared readily, the tremie shall be removed, cleared, resealed, and restarted.

The volume of concrete in place should be monitored throughout the placement. Underruns are indicative of loss of tremie seal since the washed and segregated aggregates will occupy a greater volume. Overruns are indicative of loss of concrete from the inside of the steel pile.

3.11 Field Tests for Driven Piles and Drilled Shafts

3.11.1 Integrity Test

Low strain integrity testing of piles is a tool for quality control of long structural elements that function in a manner similar to foundation piles, regardless of their method of installation, provided that they are receptive to low strain impact testing. The test provides velocity (and optionally force) data, which assists evaluation of pile integrity and pile physical dimensions (i.e., cross-sectional area, length), continuity and consistency of pile material. The test does not give any information regarding the pile bearing capacity or about pile reinforcement. Integrity test principles have been well documented in literature (ASTM 5882; Klingmuller, 1993). There exist two methods of integrity testing, namely, Pulse Echo Method (PEM) and Transient Response Method (TRM). In Pulse Echo Method, the pile head motion is measured as a function of time. The time domain record is then evaluated for pile integrity. In Transient Response Method, the pile head motion and force (measured with an instrumented hammer) are measured as a function of time. The data are then evaluated usually in the frequency domain.

In order to check the structural integrity of the piles Integrity tests shall be performed on the piles in accordance with the procedure outlined in ASTM D5882. The test is carried out by pressing a transducer onto a pile top while striking the pile head with a hand hammer. The Sonic Integrity Testing (SIT)-system registers the impact of the hammer followed by the response of the pile and shows the display. If instructed by the operator, the signal will be stored in the memory of the SIT-system together with other information, such as pile number, date, time, site, amplification factor, filter length etc. The reflectograms are horizontally scaled and vertically amplified to compensate external soil friction, which facilitate the interpretation. Consequently, the reflection of the pile toe matches the length of the pile which will be confirmed by the SIT-system. In case of any defects, the exact location can be determined from the graph on the display.

For any project where pile has been installed, integrity tests shall be performed on 100% of the piles. Integrity testing may not identify all imperfections, but it can be used in identifying major defects within effective length. In literature, there are many examples that highlight success of low strain integrity testing (Klingmuller, 1993).

<u>Factors Influencing Implication of Pile Integrity Test</u>

(a) This sonic echo pile integrity testing or dynamic response method is based on measuring (or observing on an oscilloscope) the time it takes for a reflected compression stress wave to return to the top of the pile.

(b) Some waves will be reflected by a discontinuity in the pile shaft. When the compressive strength is known for the pile material involved, the depth to the discontinuity and the pile length can be determined.

(c) On the other hand, area of pile shaft and hence its diameter, is determined from impedance of wave response, while impedance in any section is a function of elastic modulus of pile material, shaft area and wave velocity propagating through that section. If the concrete material is uniform throughout the pile length, elastic modulus and the wave velocity (provided disturbance from other source of vibration nearby is insignificant) are constant for that pile. In that case, changes in impedance usually indicate changes of pile cross-sectional area.

(d) While evaluating pile integrity (i.e., pile length and shaft diameter), the wave velocity is assumed to be constant throughout pile length. Thus, the reliability of integrity evaluation entirely depends on the pile material and its uniformity throughout shaft length while casting was done. The length and diameter obtained from pile integrity test is an indication of the actual length and diameter of the tested piles.

(e) Besides, this test can only assess shaft integrity and gives no information for pile bearing capacity determination. However, if a large number of piles are tested, it is generally easy to focus the piles having unusual responses. Therefore, whenever an integrity testing is contemplated, consideration must be given to the limitations of the various methods/process of pile installation (i.e. pile driving or casting) and the possible need for further investigation (such as pile load test) to check the results of such testing.

(f) It should be noted here that pile integrity test is an indicative test about the length and quality of concrete in the pile. This test does not give any idea about its actual load capacity. It is usually suggestive to substantiate the findings of integrity test by excavation or pull out of the pile to facilitate decisions about final acceptance or rejection of any pile. Because of the large cost involved in a pile load test, the necessity of integrity test in facilitating the selection of piles for load test is a rational approach for quality and safety assurance of piled foundations.

3.11.2 Axial Load Tests for Compression

Where accurate estimate of axial load carrying capacity of a pile is required tests in accordance with "Standard Test Method for Deep Foundations Under Static Axial Compressive Load", (ASTM D1143) or equivalent shall be performed on individual piles. For a major project, at least 2% of piles (test piles plus service piles) shall be tested in each area of uniform subsoil conditions. Where necessary, additional piles may be load tested to establish the safe design capacity. The ultimate load carrying capacity of a single pile may be determined with reasonable accuracy from load testing. The load test on a pile shall not be carried out earlier than 4 (four) weeks from the date of casting the pile. A minimum of one pile at each project shall be load tested for bored cast-in-situ piles.

Two principal types of test may be used for compression loading on piles - the constant rate of penetration (CRP) test and the maintained load (ML) test. The CRP test was developed by Whitaker (1963). The CRP method is essentially a test to determine the ultimate load on a pile and is therefore applied only to preliminary test piles or research type investigations where fundamental pile behaviour is being studied. In this test the compressive force is progressively increased to cause the pile to penetrate the soil at constant rate until failure occurs. The rate of penetration selected usually corresponds to that of shearing soil samples in unconfined compression tests. However, rate does not affect results significantly. In CRP test the recommended rates of penetration are 0.75 mm/min for friction piles in clay and 1.55 mm/min for piles end bearing in granular soil. The CRP test shall not be used for checking compliance with specification requirements for maximum settlement at given stages of loading.

Maintained load (ML) test is so far the most usual one in practice. In the ML test the load is increased in stages to 1.5 times or twice the working load with time settlement curve recorded at each stage of loading and unloading. The general procedure is to apply static loads in increments of 25% of the anticipated design load. The ML test may also be taken to failure by progressively increasing the load in stages. In the ML test, the load test arrangements as specified in (ASTM D1143) shall be followed. According to ASTM D1143 each load increment is maintained until the rate of settlement is not greater than 0.25 mm/hr or 2 hours is elapsed, whichever occurs first. After that the next load increment is applied. This procedure is followed for all increments of load. After the completion of loading if the test pile has not failed the total test load is removed any time after twelve hours if the butt settlement over one hour period is not greater than 0.25 mm otherwise the total test load is kept on the pile for 24 hours. After the required holding time, the test load is removed in decrement of 25% of the total test load with 1 hour between decrement. If failure occurs, jacking the pile is continued until the settlement equals 15% of the pile diameter or diagonal dimension. Selection of an appropriate load test method shall be based on an evaluation of the anticipated types and duration of loads during service, and shall include consideration of the following:

(i) The immediate goals of the load test (i.e., to proof load the foundation and verify design capacity)

(ii) The loads expected to act on the production foundation (compressive and/or uplift, dead and/or live), and the soil conditions predominant in the region of concern.

(iii) The local practice or traditional method

As a minimum, the written test procedures should include the following:

(i) Apparatus for applying loads including reaction system and loading system.

(ii) Apparatus for measuring movements.

(iii) Apparatus for measuring loads.

(iv) Procedures for loading including rates of load application, load cycling and maximum load.

(v) Procedures for measuring movements.

(vi) Safety requirements.

(vii) Data presentation requirements and methods of data analysis.

(viii) Drawings showing the procedures and materials to be used to construct the load test apparatus.

3.11.2.1 Load test evaluation methods for axial compressive

A number of arbitrary or empirical methods are used to serve as criteria for determining the allowable and ultimate load carrying capacity from pile load test. Some are based on maximum permissible gross or net settlement as measured at the pile butt while the others are based on the performance of the pile during the progress of testing (Chellis, 1961; Whitaker, 1976; Poulos and Davis, 1980; Fuller, 1983). It is recommended to evaluate the load carrying capacity of piles and drilled shaft using any of the following methods along with the arbitrary methods:

(a) Davission Offset Limit

(b) British Standard Institution Criterion

(c) Indian Standard Criteria

(d) Butler-Hoy Criterion

(e) Brinch-Hansen 90% Criterion

(f) Other methods approved by the Geotechnical Engineer

The recommended criteria to be used for evaluating the ultimate and allowable load carrying capacity of piles and drilled shaft are summarized below.

(a) A very useful method of computing the ultimate failure load has been reported by Davisson (1973). This method is based on offset method that defines the failure load. The elastic shortening of the pile, considered as point bearing, free standing column, is computed and plotted on the load-settlement curve, with the elastic shortening line passing through the origin. The slope of the elastic shortening line is 20^{o}. An offset line is drawn parallel to the elastic line. The offset is usually 0.15 inch plus a quake factor, which is a function of pile tip diameter. For normal size piles, this factor is usually taken as 0.1D inch, where D is the diameter of pile in foot. The intersection of offset line with gross load-settlement curve determines the arbitrary ultimate failure load. Davisson method is too restrictive for drilled piles, unless the resistance is primarily friction. This method is recommended for driven precast piles.

(b) Terzaghi (1942) reported that the ultimate load capacity of a pile may be considered as that load which causes a settlement equal to 10% of the pile diameter. However, this criterion is limited to a case where no definite failure point or trend is indicated by the load-settlement curves. This criterion has been incorporated in BS 8004 ”Code of Practice for Foundations” which recommends that the ultimate load capacity of pile should be that which causes the pile to settle a depth of 10% of pile width or diameter.

(c) The allowable load capacity of pile should be 50% of the final load, which causes the pile to settle a depth of 10% of pile width or diameter (BS 8004).

(d) Ultimate load capacity of pile is smaller of the following two (IS: 2911 Part-4):

(i) Load corresponding to a settlement equal to 10% of the pile diameter in the case of normal uniform diameter pile or 7.5% of base diameter in case of under-reamed or large diameter cast in-situ pile.

(ii) Load corresponding to a settlement of 12 mm.

(e) Allowable load capacity of pile is smaller of the following (IS: 2911 Part-4):

(i) Two thirds of the final load at which the total settlement attains a value of 12 mm.

(ii) Half of the final load at which total settlement equal to 10% of the pile diameter in the case of normal uniform diameter pile or 7.5% of base diameter in case of under-reamed pile.

(f) Butler and Hoy (1977) states that the intersection of tangent at initial straight portion of the load-settlement curve and the tangent at a slope point of 1.27 mm/ton determines the arbitrary ultimate failure load.

(g) The Brinch Hansen (1963) proposed a definition for ultimate load capacity as that load for which the settlement is twice the settlement under 90 percent of the full test load.

(h) Where failure occurs, the ultimate load may be taken to calculate the allowable load using a factor of safety of 2.0 to 2.5.

For load test on working pile/shaft, the safe load should be determined using the criteria of Sec 3.10.1.16.

3.11.2.2 Some factors influencing interpretations of load test results for axial compression

The following factors should be taken into account while interpreting the test results from pile load tests:

(a) Potential residual loads (strains) in the pile which could influence the interpreted distribution of load along the pile shaft.

(b) Possible interaction of friction loads from test pile with downward friction transferred to the soil from reaction piles obtaining part or all of their support in soil at levels above the tip level of the test pile.

(c) Changes in pore water pressure in the soil caused by pile driving, construction fill and other construction operations which may influence the test results for frictional support in relatively impervious soils such as clay and silt.

(d) Differences between conditions at time of testing and after final construction such as changes in grade groundwater level.

(e) Potential loss of soil resistance from events such as excavation, or scour, or both of surrounding soil.

(f) Possible difference in the performance of a pile in a group or of a pile group from that of a single pile.

(g) Effect on long term pile performance of factors such as creep, environmental effects on pile material, friction loads from swelling soils and strength losses.

(h) Type of structure to be supported, including sensitivity of structure to movement and relations between live and dead loads.

(i) Special testing procedures which may be required for the application of certain acceptance criteria or methods of interpretation.

(j) Requirement of all conditions for non-tested piles be basically identical to those for test pile including such thing as subsurface conditions, pile type, length, size and stiffness, and pile installation methods and equipment so that application or extrapolation of the test results to such other piles is valid.

3.11.3 Load Test for Uplift Capacity of Driven Pile, Bored Pile and Drilled Shaft

Where required by the design, the uplift capacity of pile and drilled shaft shall be determined by an approved method or analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D3689 (Standard Test Method for Deep Foundations Under Static Axial Tensile Load). The maximum allowable uplift load shall not exceed the ultimate load capacity as determined using the results of load test conducted in accordance with ASTM D3689, divided by a factor of safety of 2.0. Where uplift is due to wind or seismic loading, the minimum factor of safety shall be 2.0 where capacity is determined by an analysis and 1.5 where capacity is determined by load tests.

For group pile subjected to uplift, the allowable working uplift load for the group shall be calculated by an approved method of analysis where the piles in the group are placed at centre-to-centre spacing of at least 2.5 times the least horizontal dimension of the largest pile, the allowable working uplift load for the group is permitted to be calculated as the lesser of the two:

(i) The proposed individual working load times the number of piles in the group.

(ii) Two-thirds of the effective weight of the group and the soil contained within a block defined by the perimeter of the group and the embedded length of the pile.

(iii) One-half the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the embedded pile length plus one-half the total soil shear on the peripheral surface of the group

Uplift or tension test on piles subject to tension/uplift shall be performed by a continuous rate of uplift (CRU) or an incremental loading (i.e. ML) test. Where uplift loads are intermittent or cyclic in character, as in wave loading on a marine structure, it is recommended to adopt repetitive loading on the test pile. The tests shall be performed in accordance with ASTM D3689. Safe load shall be taken as the least of the following:

(a) Two thirds of the load at which the total displacement (pile top) is 12 mm or the load corresponding to a specified permissible uplift, and

(b) Half of the load at which the load displacement curve shows a clear break (downward trend).

The initial load test (on test pile/shaft) shall be carried out up to twice the estimated design load or the load displacement curve shows a clear break. The routine test on working pile shall be done up to one and a half times the design load or 12 mm total displacement whichever occurred earlier.

3.11.4 Load Tests for Lateral Load Capacity

Load test for lateral capacity shall be performed as per the procedure of ASTM D3966. Safe load capacity shall be determined as per criteria mentioned in 3.10.1.20 for driven piles.

Division C: Additional Considerations in Planning, Design and Construction of Building Foundations (Sections 3.12 To 3.22)

3.12 Excavation

Excavation for building foundation or for other purpose shall be done in a safe manner so that no danger to life and property prevails at any stage of the work or after completion. The requirements of this Section shall be satisfied for all such works in addition to those of Sec 3.3 of Part 7.

Permanent excavations shall have retaining walls of sufficient strength made of steel, masonry, or reinforced concrete to retain the embankment, together with any surcharge load.

Excavations for any purpose shall not extend within 300 mm under any footing or foundation, unless such footing or foundation is properly underpinned or protected against settlement, beforehand.

The design and construction of deep excavation work more than 6 m depth or excavation in soft soil or erratic soil must be checked by a competent Geotechnical Engineer.

3.12.1 Notice to Adjoining Property

Prior to any excavation close to an adjoining building in another property, a written notice shall be given to the owner of the adjoining property at least 10 days ahead of the date of excavation. The person undertaking the excavation shall, where necessary, incorporate adequate provisions and precautionary measures to ensure safety of the adjoining property and shall supply the details of such measures in the notice to the owner of the adjoining property. He shall obtain approval of the

Authority regarding the protective provisions, and permission of the owner of the adjoining property regarding the proposed excavation in writing. The protective measures shall incorporate the following:

(i) Where the level of the foundations of the adjoining structure is at or above the level of the bottom of the proposed excavation, the vertical load of the adjoining structure shall be supported by proper foundations, underpinning, or other equivalent means.

(ii) Where the level of the foundations of the adjoining structure is below the level of the bottom of the proposed excavation, provision shall be made to support any increased vertical or lateral load on the existing adjoining structure caused by the new construction.

If on giving the required notice, incorporating or proposing to incorporate the protective provisions which have duly been approved by the Authority, the owner of the adjoining property refuses to permit the proposed excavation or to allow necessary access and other facilities to the person undertaking the excavation for providing the necessary and approved protection to the adjoining property, the responsibility for any damage to the adjoining property due to excavation shall be that of the owner of the adjoining property.

3.12.2 Excavation Work

Every excavation shall be provided with safe means of entry and exit kept available at all times. When an excavation has been completed, or partly completed and discontinued, abandoned or interrupted, or the required permits have expired, the lot shall be filled and graded to eliminate all steep slopes, holes, obstructions or similar sources of hazard. Fill material shall consist of clean, noncombustible substances. The final surface shall be graded in such a manner as to drain the lot, eliminate pockets, prevent accumulation of water, and preclude any threat of damage to the foundations on the premises or on the adjoining property.

3.12.2.1 Methods of protection

<u>Shoring, Bracing and Sheeting</u>

With the exception of rock cuts, the sides of all excavations, including related or resulting embankments, 1.5 m or greater in depth or height measured from the level of the adjacent ground surface to the deepest point of excavation, shall be protected and maintained by shoring, bracing and sheeting, sheet piling, or other retaining structures. Alternatively, excavated slopes may be inclined not steeper than 1:1, or stepped so that the average slope is not steeper than forty five degrees with no step more than 1.5 m high, provided such slope does not endanger any structure, including subsurface structures. All sides or slopes of excavations or embankments shall be inspected after rainstorms, or any other hazard increasing event, and safe conditions shall be restored. Sheet piling and bracing needed in trench excavations shall have adequate strength to resist possible forces resulting from earth or surcharge pressure. Design of Protection system shall be checked by a qualified Geotechnical Engineer.

Guard Rail

A guard rail or a solid enclosure at least 1 m high shall be provided along the open sides of excavations, except that such guard rail or solid enclosure may be omitted from a side or sides when access to the adjoining area is precluded, or where side slopes are one vertical to three horizontal or flatter.

3.12.2.2 Placing of construction material

Excavated materials and superimposed loads such as equipment, trucks, etc. shall not be placed closer to the edge of the excavation than a distance equal to one and one-half times the depth of such excavation, unless the excavation is in rock or the sides have been sloped or sheet piled (or sheeted) and shored to withstand the lateral force imposed by such superimposed load. When sheet piling is used, it shall extend at least 150 mm above the natural level of the ground. In the case of open excavations with side slopes, the edge of excavation shall be taken as the toe of the slope.

3.12.2.3 Safety regulations

Whenever subsurface operations are conducted that may impose loads or movement on adjoining property, such as driving of piles, compaction of soils, or soil densification, the effects of such operations on adjoining property and structures shall be considered. The owner of the property that may be affected shall be given 48 hours written notice of the intention to perform such operations. Where construction operations will cause changes in the ground water level under adjacent buildings, the effects of such changes on the stability and settlement of the adjacent foundation shall be investigated and provision made to prevent damage to such buildings. When a potential hazard exists, elevations of the adjacent buildings shall be recorded at intervals of twenty four hours or less to ascertain if movement has occurred. If so, necessary remedial action shall be undertaken immediately.

Whenever, an excavation or fill is to be made that will affect safety, stability, or usability of, the adjoining properties or buildings shall be protected as required by the provisions of Sec 3.3 Part 7.

On excavation, the soil material directly underlying footings, piers, and walls shall be inspected by an engineer/architect prior to construction of the footing. If such inspection indicates that the soil conditions do not conform to those assumed for the purposes of design and described on the plans, or are unsatisfactory due to disturbance, then additional excavation, reduction in allowable bearing pressure, or other remedial measures shall be adopted.

Except in cases where a proposed excavation will extend less than 1.5 m below grade, all underpinning operations and the construction and excavation of temporary or permanent cofferdams, caissons, braced excavation surfaces, or other constructions or excavations required for or affecting the support of adjacent properties or buildings shall be subject to controlled inspection. The details of underpinning, and construction of cofferdams, caissons, bracing or other constructions required for the support of adjacent properties or buildings shall be shown on the plans or prepared in the form of shop or detail drawings and shall be approved by the engineer who prepared the plans.

3.13 Dewatering

All excavations shall be drained and the drainage maintained as long as the excavation continues or remains. Where necessary, pumping shall be used. No condition shall be created as a result of construction operations that will interfere with natural surface drainage. Water courses, drainage ditches, etc. shall not be obstructed by refuse, waste building materials, earth, stones, tree stumps, branches, or other debris that may interfere with surface drainage or cause the impoundment of surface water.

3.14 Slope Stability of Adjoining Buildings

The possibility of overturning and sliding of the building shall be considered. The minimum factor of safety against overturning of the structure as a whole shall be 1.5. Stability against overturning shall be provided by the dead load of the building, the allowable uplift capacity of piling, anchors, weight of the soil directly overlying footings provided that such soil cannot be excavated without recourse to major modification of the building, or by any combination of these factors.

The minimum factor of safety against sliding of the structure under lateral load shall be 1.5. Resistance to lateral loads shall be provided by friction between the foundation and the underlying soil, passive earth pressure, batter piles or by plumb piles, subject to the following:

(i) The resistance to lateral loads due to passive earth pressure shall not be taken into consideration where the abutting soil could be removed inadvertently by excavation.

(ii) In case of pile supported structures, frictional resistance between the foundation and the underlying soil shall be discounted.

(iii) The available resistance to friction between the foundation and the underlying soil shall be predicted on an assumed friction factor of 0.5. A greater value of the coefficient of friction may be used subject to verification by analysis and test.

The faces of cut and fill slopes shall be prepared and maintained to control erosion. The control may consist of effective planting. The protection for slopes shall be installed as soon as practicable. Where cut slopes are not subject to erosion due to erosion resistant character of the materials, such protection may be omitted. Where necessary, check dams, cribbing, riprap or other devices or methods shall be employed to control erosion.

3.15 Fills

3.15.1 Quality of Fill

The excavation outside the foundation shall be backfilled with soil that is free of organic material, construction debris and large rocks. The backfill shall be placed in lifts and compacted in a manner which does not damage foundation, the waterproofing or damp-proofing material.

3.15.2 Placement of Fill

Fills to be used to support the foundation of any building or structure shall be placed in accordance with established engineering principle. Before placement of the fill, the existing ground surface shall be stripped off all organic growth, timber, rubbish and debris. After stripping, the ground surface shall be compacted. Materials for fill shall consist of sand, gravel, crushed stone, crushed earth, or a mixture of these. The fill material shall contain no particles exceeding 100 mm in the largest dimension. A soil investigation report and a report of satisfactory placement of fill, both acceptable to the Building Official shall be submitted. In an uncontrolled fill, the soil within the building area shall be explored using test pits. At least one test pit penetrating at least 2 m below the level of the bottom of the proposed foundation shall be provided for every 200 m^2 of building area. Wherever such test pits consistently indicate that the fill is composed of material that is free of voids and free of extensive inclusion of mud, organic materials such as paper, garbage, cans, metallic objects, or debris, the fill material shall be acceptable. Where the fill shows voids or inclusions as described above, either the fill shall be treated as having no presumptive bearing capacity, or the building shall incorporate adequate strength and stiffness to bridge such voids or inclusions or shall be articulated to prevent damage due to differential or localized settlement of the fill.

3.15.3 Specifications

Where foundations are to be placed on controlled fill materials, the fill must be compacted in layers not exceeding 300 mm. Clear specifications shall be provided for the range of water content, the degree of compaction to be achieved and the method of compaction that shall be followed. Such specifications shall be based on the shear strength requirement for the fill soil and allowable settlement estimate. The minimum density of controlled fill shall be 95% of the optimum density obtained from "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort ", (ASTM D1557).

The degree of compaction achieved in a fill shall be obtained from in-situ density measurements. No new layer shall be placed unless a satisfactory density is attained in each layer.

3.16 Protective Retaining Structures for Foundations/ Shore Piles

A retaining wall is a wall designed to resist lateral earth and/or fluid pressures, including any surcharge, in accordance with accepted engineering practice. Retaining walls for foundations shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift; and that they be designed for a safety factor of 1.5 against lateral sliding and overturning. Generally sheet pile retaining walls are used for construction raft foundations for buildings. Taller sheet piles may need a tie back anchor driven and anchored behind the soil of the sheet pile retaining wall.

3.17 Waterproofing and Damp-Proofing

3.17.1 General

Walls or portions thereof that retain earth and enclose interior spaces, and floors below grade shall be waterproofed and damp-proofed, with the exception of those spaces where such omission is not detrimental to the building or occupancy. The roof is also required to be waterproofed. The owner shall perform a subsurface investigation to determine the possibility of the ground water table rising above the proposed elevation of the floor or floors below grade unless satisfactory data from adjacent areas demonstrate that ground water has not been a problem.

There may arise two situations: (i) where no hydrostatic pressure occurs and (ii) where hydrostatic pressure occurs. Where hydrostatic pressure conditions exist, floors and walls below finished ground level shall be waterproofed in accordance with Sec 3.17.1.1 below. Where hydrostatic pressure conditions do not exist, damp-proofing and perimeter drainage shall be provided in accordance with Sec 3.17.1.2 below. In addition, the damp-proofing and waterproofing shall also meet the requirements of Sec 3.13.3. All damp-proofing and waterproofing materials shall conform to the requirements of Sec 2.16.7 of Part 5.

3.17.1.1 Waterproofing where hydrostatic pressure occurs

Where ground water investigation indicates that a hydrostatic pressure condition exists, or is likely to occur, walls and floors shall be waterproofed in accordance with the provisions stated as under.

3.17.1.2 Floor waterproofing

Floors required to be waterproofed shall be of concrete and shall be designed and constructed to withstand the anticipated hydrostatic pressure. Waterproofing of the floor shall be accomplished by placing under the slab a membrane of rubberized asphalt, or butyl rubber, or polymer modified asphalt, or neoprene, or not less than 0.15 mm polyvinyl chloride or polyethylene, or other approved materials, capable of bridging nonstructural cracks. Joints in the membrane shall be lapped not less than 150 mm and sealed in an approved manner.

3.17.1.3 Wall waterproofing

Walls required to be waterproofed shall be of concrete or masonry designed to withstand the anticipated hydrostatic pressure and other lateral loads. Prior to the application of waterproofing materials on concrete walls, all holes and recesses resulting from the removal of form ties shall be sealed with a bituminous material or other approved methods or materials. Unit masonry walls shall be pargeted on the exterior surface below ground level with not less than 10 mm of Portland cement mortar. The pargeting shall be continued to the foundation. Pargeting of unit masonry walls is not required where a material is approved for direct application to the masonry.

Waterproofing shall be applied from a point 300 mm above the maximum elevation of the ground water table down to the top of the spread portion of the foundation. The remainder of the wall up to a level not less than 150 mm above finished grade shall be damp-proofed.

Wall waterproofing materials shall consist of two-ply hot-mopped felts, not less than 0.15 mm polyvinylchloride, 1.0 mm polymer modified asphalt, 0.15 mm polyethylene or other approved methods or materials capable of bridging nonstructural cracks. Joints in the membrane shall be lapped not less than 150 mm and sealed in an approved manner. Joints in walls and floors, joints between the wall and the floor, and penetrations of the wall and floor shall be made watertight utilizing established methods and materials.

3.17.1.4 Damp-proofing with no hydrostatic pressure

Where hydrostatic pressure will not occur, floors and walls shall be damp-proofed and a subsoil drainage system shall be installed as described below:

3.17.1.5 Floor damp-proofing

For floors, damp-proofing materials shall be installed between the floor and base materials. The base material shall not be less than 100 mm in thickness consisting of gravel or crushed stone containing not more than 10 percent material that passes a 4.75 mm sieve. Where a site is located in well drained gravel or sand/gravel mixture, a floor base is not required. When the finished ground level is below the floor level for more than 25 percent of the perimeter of the building, the base material need not be provided. Where a separate floor is provided above a concrete slab the damp-proofing may be installed on top of the slab.

Damp-proofing materials, where installed beneath the slab, shall consist of not less than 0.15 mm polyethylene with joints lapped not less than 150 mm, or other approved methods or materials. Where permitted to be installed on top of the slab, damp-proofing shall consist of mopped on bitumen, not less than 0.1 mm polyethylene, or other approved methods or materials. Joints in membranes shall be lapped not less than 150 mm and sealed in an approved manner.

3.17.1.6 Wall damp-proofing

For walls, damp-proofing materials shall be installed and shall extend from a point 150 mm above grade, down to the top of the spread portion of the foundation.

Wall damp-proofing material shall consist of a bituminous material, acrylic modified cement base coating, rubberized asphalt, polymer-modified asphalt, butyl rubber, or other approved materials capable of bridging nonstructural cracks.

3.17.1.7 Perimeter drain

A drain shall be placed around the perimeter of a foundation that consists of gravel or crushed stone containing not more than 10 percent material that passes through a 4.76 mm sieve. The drain shall extend a minimum of 300 mm beyond the outside edge of the foundation. The thickness shall be such that the bottom of the drain is not higher than the bottom of the base under the floor, and that the top of the drain is not less than 150 mm above the top of the foundation. The top of the drain shall be covered with an approved filter membrane material. Where a drain tile or perforated pipe is used, the invert of the pipe or tile shall not be higher than the floor elevation. The top of joints or the top of perforations shall be protected with an approved filter membrane material. The pipe or tile shall be placed on not less than 50 mm of gravel or crushed stone complying with this section, and shall be covered with not less than 150 mm of the same material.

The floor base and foundation perimeter drain shall discharge by gravity or mechanical means into an approved drainage system. Where a site is located in well drained gravel or sand/gravel mixture, a dedicated drainage system is not required. When the finished ground level is below the floor level for more than 25 percent of the perimeter of the building, the foundation drain need be provided only around that portion of the building where the ground level is above the floor level.

3.17.2 Other Damp-proofing and Waterproofing Requirements

3.17.2.1 Placement of backfill

The excavation outside the foundation shall be backfilled with soil that is free of organic material, construction debris and large rocks. The backfill shall be placed in lifts and compacted in a manner which does not damage the waterproofing or damp-proofing material or structurally damage the wall.

3.17.2.2 Site grading

The ground immediately adjacent to the foundation shall be sloped away from the building at a slope not less than 1 unit vertical in 12 units horizontal (1:12) for a minimum distance of 2.5 m measured perpendicular to the face of the wall or an alternative method of diverting water away from the foundation shall be used. Consideration shall be given to possible additional settlement of the backfill when establishing the final ground level adjacent to the foundation.

3.17.2.3 Erosion protection

Where water impacts the ground from the edge of the roof, down spout, scupper, valley or other rainwater collection or diversion device, provisions shall be used to prevent soil erosion and direct the water away from the foundation.

3.18 Foundation on Slopes

Where footings are to be founded on a slope, the distance of the sloping surface at the base level of the footing measured from the centre of the footing shall not be less than twice the width of the footing.

When adjacent footings are to be placed at different levels, the distance between the edges of footings shall be such as to prevent undesirable overlapping of structures in soil and disturbance of the soil under the higher footing due to excavation of the lower footing.

On a sloping site, footing shall be on a horizontal bearing and stepped. At all changes of levels, footings shall be lapped for a distance of at least equal to the thickness of foundation or three times the height of step, whichever is greater. Adequate precautions shall be taken to prevent tendency for the upper layers of soil to move downhill.

3.19 Foundations on Fills and Problematic Soils

3.19.1 Footings on Filled up Ground

Footings shall not be constructed on loosely filled up ground with non-uniform density or consistency, unless adequate strengthening of the soil is made by applying ground improvement techniques.

3.19.2 Ground Improvement

In poor and weak subsoil, the design of 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 recent years is to improve the subsoil to an extent that the subsoil would develop an adequate bearing capacity and foundations constructed after subsoil improvement would have resultant settlements within acceptable limits. Selection of ground improvement techniques may be done in accordance with good practice.

3.19.3 Soil Reinforcement

Use of suitable geo-synthetics/geo-textiles may be made in an approved manner for ground improvement where applicable based on good practice.

3.20 Foundation Design for Dynamic Forces

3.20.1 Effect of Dynamic Forces

Where machinery operations or other vibrations are transmitted through foundation, consideration shall be given in the foundation design to prevent detrimental disturbance of the soil. Impact forces shall be neglected in foundation design except for foundations bearing on loose granular soils, foundations supporting cranes, heavy machinery and moving equipment, or where ratio of live load causing the impact to the dead load exceeds 50%.

3.20.2 Machine Foundation

Machine foundations are subjected to the dynamic forces caused by the machine. These dynamic forces are transmitted to the foundation supporting the machine. Although the moving parts of the machine are generally balanced, there is always some unbalance in practice which causes an eccentricity of rotating parts. This produces an oscillating force. The machine foundation must satisfy the criteria for dynamic loading in addition to that for static loading.

3.20.2.1 Types of machine foundations

Basically, there are three types of machine foundation:

(i) Machines which produce a periodic unbalanced force, such as reciprocating engines and compressors. The speed of such machines is generally less than 600 rpm. In these machines, the rotary motion of the crank is converted into the translatory motion. The unbalanced force varies sinusoidal.

(ii) Machines which produce impact loads, such as forge hammers and punch presses. In these machines, the dynamic force attains a peak value in a very short time and then dies out gradually. The response is a pulsating curve. It vanishes before the next pulse. The speed is usually between 60 to 150 blows per minute.

(iii) High speed machines, such as turbines, and rotary compressors. The speed of such machines is very high; sometimes, it is even more than 3000 rpm.

The following four types of machine foundations are commonly used.

(i) Block Type: This type of machine foundation consists of a pedestal resting on a footing (Figure 6.3.4a). The foundation has a large mass and a small natural frequency.

(ii) Box Type: The foundation consists of a hollow concrete block (Figure 6.3.4b). The mass of the foundation is less than that in the block type and the natural frequency is increased.

(iii) Wall Type: A wall type of foundation consists of a pair of walls having a top slab. The machine rests on the top slab (Fig6.3.4c).

(iv) Framed Type: This type of foundation consists of vertical columns having a horizontal frame at their tops. The machine is supported on the frame (Figure 6.3.4d).

Machines which produce periodical and impulsive forces at low speeds are generally provided with a block type foundation. Framed type foundations are generally used for the machines working at high speeds and for those of the rotating types. Some machines which induce very little dynamic forces, such as lathes, need not be provided with a machine foundation. Such machines may be directly bolted to the floor.

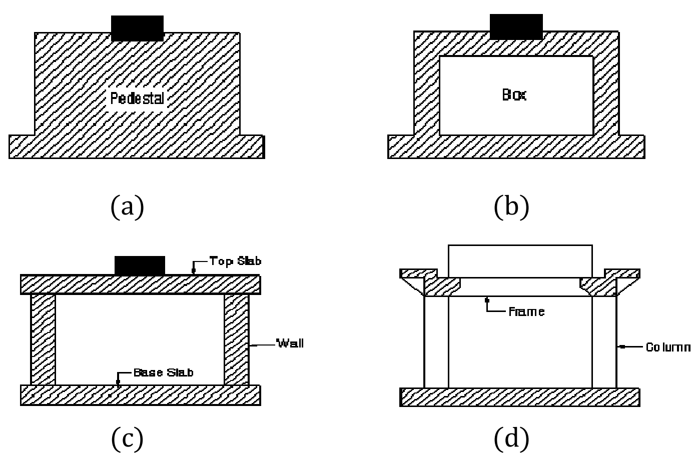


Figure 6.3.4. Types of machine foundations; (a) Block type; (b) Box type; (c) Wall type; (d) Framed type

3.20.2.2 Design considerations

For satisfactory performance, machine foundations should satisfy the following requirements: (i) resonance is avoided, (ii) bearing capacity and settlement are safe, and (iii) there is an adequate vibration and shock isolation. Avoidance of resonance is discussed in this Section.

Resonance:

Based on their operating frequencies, the machines are classified as (i) low speed having frequency less than 300 revolutions per minute (rpm), (ii) medium speed, frequency 300 to 1000 rpm, and (iii) high speed, frequency greater than 1000 rpm. To avoid resonance, the natural frequency (or the resonant frequency) of the machine foundation-soil system must be either very large or very small compared to the operating speed of the machine.

Low speed machines ($f_1 < 300$ rpm):

Provide a foundation with a natural frequency at least twice the operating frequency, i.e., the frequency ratio r $(= f_1/f_n)$ is less than 0.5. Natural frequency can be increased (i) by increasing base area or reducing total static weight of the foundation, (ii) by increasing modulus of shear rigidity of the soil by compaction, grouting or injection, (iii) by using piles to provide the required foundation stiffness.

High speed machines ($f_1 > 1000$ rpm):

Provide a foundation with natural frequency not higher than one-half of the operating value, i.e., frequency ratio ≥ 2 . Natural frequency can be decreased by increasing weight of foundation. During starting and stopping, the machine will operate briefly at resonant frequency f_r of the foundation. Probable amplitude is computed at both f_r and f_1 and compared with allowable values to determine if the foundation arrangement must be altered.

Types of foundations:

Considering their structural forms, the machine foundations, in general, are of the following types: (i) box foundation consisting of a pedestal of concrete, (ii) box foundation consisting of a hollow concrete block, (iii) wall foundation consisting of a pair of walls supporting the machine. (iv) framed foundation consisting of vertical columns and a top horizontal frame work which forms the seat of essential machinery.

Low speed machines (e.g., forge hammers, presses, low speed reciprocating engines and compressors) are generally supported on block foundation having a large contact area with soil.

Medium speed machines (e.g., reciprocating diesel and gas engines) also have, in general, block foundations resting on springs or suitable elastic pads.

High speed and rotating type of machines (e.g., internal combustion engines, electric motors, and turbo generator machines) are generally mounted on framed foundations. Other high speed machines are placed on block foundations.

As far as possible, the centre of gravity of the whole system and the centroid of the base area should be on the same vertical axis. At the most an eccentricity of 5% could be allowed.

Permissible amplitude:

Many times the permissible amplitude at operating speed is specified by the manufactures. If not specified, the following values may be adopted for guidance (i) low speed machines. ($f_1 < 500$ rpm), horizontal and vertical vibrations, $A = 0.25$ mm. (ii) operating speed $f_1 = 500$ to 1500 rpm, $A = 0.4$ mm to 0.6 mm for horizontal, and $A = 0.7$ mm to 0.9 mm for vertical mode of vibration; (iii) operating speed f_1 up to 3000 rpm, $A = 0.2$ mm for horizontal and $A = 0.5$ mm for vertical vibrations (iv) hammer foundations, $A = 10$ mm.

3.20.2.3 Design methods

The various design methods can be grouped as follows: (i) empirical and semi-empirical methods, (ii) methods considering soil as a spring and (iii) methods considering soil as a semi-infinite elastic mass (elastic half-space-approach) and its equivalent lumped parameter method. The lumped parameter method is currently preferred and will be described here. A good machine foundation should satisfy the following criteria.

(i) Like ordinary foundations, it should be safe against shear failure caused by superimposed loads, and also the settlements should be within the safe limits.

(ii) The soil pressure should normally not exceed 80% of the allowable pressure for static loading.

(iii) There should be no possibility of resonance. The natural frequency of the foundation should be either greater than or smaller than the operating frequency of the machine.

(iv) The amplitudes under service condition should be within the permissible limits for the machine.

(v) The combined centre of gravity of the machine and the foundation should be on the vertical line passing through the centre of gravity of the base plane.

(vi) Machine foundation should be taken to a level lower than the level of the foundation of the, adjacent buildings and should be properly separated.

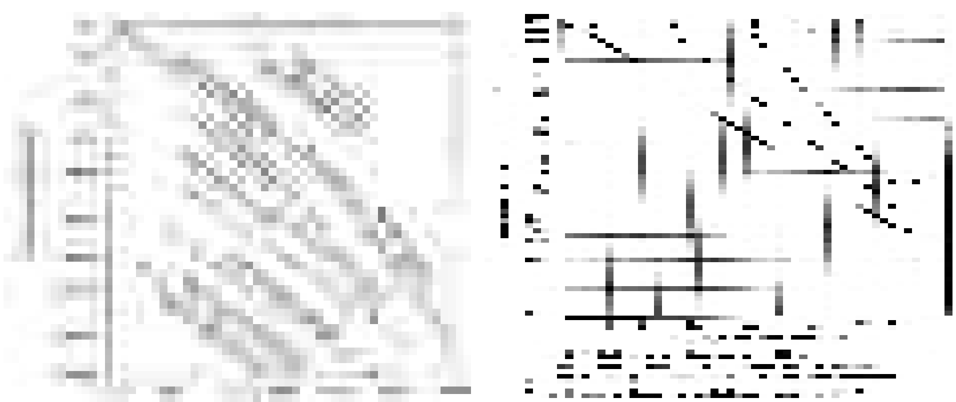
(vii) The vibrations induced should neither be annoying to the persons nor detrimental to other structures.

(viii) Richart (1962) developed a plot for vertical vibrations, which is generally taken as a guide for various limits of frequency and amplitude which has been presented in Figure 6.3.5(a). A modified chart suggested by IS: 2974-Part 1, Figure 6.3.5(b) may also be used.

(ix) The depth of the ground-water table should be at least one fourth of the width of the foundation below the base place.

3.20.2.4 Vibration analysis of a machine foundation:

Although a machine foundation has 6 degree of freedom, it is assumed to have a single degree of freedom for a simplified analysis. Figure 6.3.6 shows a machine foundation supported on a soil mass. In this case, the mass m_f lumps together the mass of the machine and the mass of foundation. The total mass m_f acts at the centre of gravity of the system. The mass is under the supporting action of the soil. The elastic action can be lumped together into a single elastic spring with a stiffness k. Likewise; all the resistance to motion is lumped into the damping coefficient c. Thus the machine foundation reduces to a single mass having one degree of freedom. The analysis of damped, forced vibration is, therefore, applicable to the machine foundation.



(a) (b)

Figure 6.3.5. Limits of frequency and amplitudes of foundation; (a) Richart (1962) chart; (b) IS: 2974-Part 1, 1982

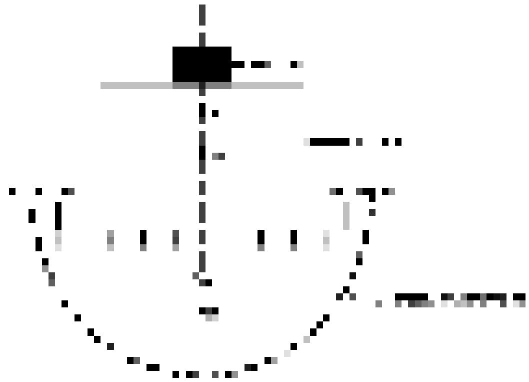


Figure 6.3.6. Machine foundation supported on a soil mass

3.20.2.5 Determination of parameters for vibration analysis

For vibration analysis of a machine foundation, the parameters m, c and k are required. These parameters can be determined as under.

Mass (m):

When a machine vibrates, some portion of the supporting soil mass also vibrates. The vibrating soil is known as the participating mass or in-phase soil mass. Therefore, the total mass of the system is equal to the mass of the foundation block and machine (m_f) and the mass (m_s) of the participating soil. Thus

$$m = m_f + m_s \quad (6.3.51)$$

Unfortunately, there is no rational method to determine the magnitude of m_s. It is usually related to the mass of the soil in the pressure bulb. The value of m_s generally varies between zero and m_f. In other words, the total mass (m) varies between m_f and $2m_f$ in most cases.

Spring Stiffness (k):

The spring stiffness depends upon the type of soil, embedment of the foundation block, the contact area and the contact pressure distribution. The following are the common methods.

Laboratory Test:

A triaxial test with vertical vibrations is conducted to determine Young's modulus (E). Alternatively, the modulus of rigidity (G) is determined conducting the test under torsional vibration, and E is obtained indirectly from the relation, $E = 2G(1 + \mu)$, where μ is Poisson's ratio. The stiffness (k) is determined as

$$k = \frac{A_{sp}E}{L} \quad (6.3.52)$$

Where, A_{sp} = cross-sectional area of the specimen, and L = length of the specimen.

Barkan's Method:

The stiffness can also be obtained from the value of E using the following relation given by Barken.

$$k = \frac{1.13E}{1-\mu}\sqrt{A} \tag{6.3.53}$$

Where, A = base area of the machine, i.e. area of contact.

Plate Load Test:

A repeated plate load test is conducted and the stiffness of the soil k_p is found as the slope of the load-deformation curve. The spring constant k of the foundation is as under.

For cohesive soils:

$$k = k_p\left(\frac{B}{B_p}\right) \tag{6.3.54}$$

For cohesionless soil:

$$k = k_p\left(\frac{B+0.3}{B_p+0.3}\right)^2 \tag{6.3.55}$$

Where, B is the width of foundation (in m), B_p is the width of plate (in m). Alternatively, spring constant can be obtained from the subgrade modulus k_s, as

$$k = k_s A \tag{6.3.56}$$

Where, A = area of foundation.

Resonance Test:

The resonance frequency f_n is obtained using a vibrator of mass m set up on a steel plate supported on the ground. The spring stiffness obtained from the relation

$$f_n = \frac{\omega_n}{2\pi} = \frac{1}{2\pi}\sqrt{k/m} = 4\pi^2 f_n m \tag{6.3.57}$$

Where, ω_n is natural circular frequency.

Damping Constant (c):

Damping is due to dissipation of vibration energy, which occurs mainly because of the following reasons.

(i) Internal friction loss due to hysteresis and viscous effects.

(ii) Radiational loss due to propagation of waves through soil.

The damping factor D for an under-damped system can be determined in the laboratory. Vibration response is plotted and the logarithmic decrement δ is found from the plot, as

$$\delta = \frac{2\pi D}{\sqrt{1-D^2}} \Longrightarrow D = \frac{\delta}{2\pi} \qquad (6.3.58)$$

The damping factor D may also be obtained from the area of hysteresis loop of the load displacement curve, as

$$D = \frac{\Delta W}{W} \qquad (6.3.59)$$

Where, W = total work done; and ΔW = work lost hysteresis. The value of D for most soils generally varies between 0.01 and 0.1.

3.21 Geo-Hazard Analysis for Buildings

Geo-hazard analysis of buildings include design considerations for possible landslides, ground subsidence, earthquakes and other seismic events, erosion and scour, construction in toxic and/or contaminated landfills, groundwater contamination etc. A preliminary review of the selected site should be carried out for existence of any of the above mentioned geo-hazard in the area. A detailed analysis may be carried out only if the preliminary review indicates a significant threat for the building which may exist from any of the above mentioned potential geo-hazard at the selected location for the building. See relevant section for details.

3.22 List of Related Appendices

Appendix D	Methods of Soil Exploration, Sampling and Groundwater Measurements
Appendix E	Recommended Criteria for Identification and Classification of Expansive Soil
Appendix F	Construction of Pile Foundation
Appendix G	Other Methods of Estimating Ultimate Axial Capacity of Piles and Drilled Shafts, and Design Charts for Settlement
Appendix H	References of Chapter 3 Part 6 (Soils and Foundations).

PART VI
Chapter 4
Bamboo Structures

4.1 Scope

This Section relates to the use of bamboo in construction as structural elements, nonstructural elements and also for temporary works 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.

4.2 Terminology

For the purpose of this Section, the following definitions shall apply.

4.2.1 Anatomical Purpose Definitions

BAMBOO	Tall perennial grasses found in tropical and sub-tropical regions. They belong to the family Poaceae and sub-family Bambusoidae.
BAMBOO CULM	A single shoot of bamboo usually hollow except at nodes which are often swollen.
BAMBOO CLUMP	A cluster of bamboo culms emanating from two or more rhizomer in the same place.
CELLULOSE	A carbohydrate, forming the fundamental material of all plants and a main source of the mechanical properties of biological materials.
CELL	A fundamental structural unit of plant and animal life, consisting of cytoplasm and usually enclosing a central nucleus and being surrounded by a membrane (animal) or a rigid cell wall (plant).
CROSS WALL	A wall at the node closing the whole inside circumference and completely separating the hollow cavity below from that above.

HEMI CELLULOSE	The polysaccharides consisting of only 150 to 200 sugar molecules, also much less than the 10000 of cellulose.
LIGNIN	A polymer of phenyl propane units, in its simple form ($C_6H_5CH_3CH_2CH_3$).
SLIVER	Thin strips of bamboo processed from bamboo culm.
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.

4.2.2 Structural Purpose Definitions

BAMBOO MAT BOARD	A board made of two or more bamboo mats bonded with an adhesive.
BEAM	A structural member which supports load primarily by its internal resistance to bending.
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.
BUNDLE-COLUMN	A column consisting of three or more number of culm bound as integrated unit with wire or strap type of fastenings.
CENTRE INTERNODE	A test specimen having its centre between two nodes.
CHARACTERISTIC LOAD	The value of loads which has a 95 percent probability of not exceeding during the life of the structure.
CHARACTERISTIC STRENGTH	The strength of the material below which not more than 5 percent of the test results are expected to fall.
CLEAVABILITY	The ease with which bamboo can be split along the longitudinal axis. The action of splitting is known as cleavage.
COLUMN	A structural member which supports axial load primarily by inducing compressive stress along the fibres.
COMMON RAFTER	A roof member which supports roof battens and roof coverings, such as boarding and sheeting.

CURVATURE	The deviation from the straightness of the culm.
DELAMINATION	Separation of mats through failure of glue.
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.
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.
FULL CULM	The naturally available circular section/shape.
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.
INNER DIAMETER	Diameter of internal cavity of a hollow piece of bamboo.
INSIDE LOCATION	Position in buildings in which bamboo remains continuously dry or protected from weather.
JOINT	A connection between two or more bamboo structural elements.
JOIST	A beam directly supporting floor, ceiling or roof of a structure.
LENGTH OF INTERNODE	Distance between adjacent nodes.
LOADED END OR COMPRESSION END DISTANCE	The distance measured from the centre of the fastener to the end towards which the load induced by the fastener acts.
MATCHET	A light cutting and slashing tool in the form of a large knife.
MAT	A woven sheet made using thin slivers.
MORTISE AND TENON	A joint in which the reduced end (tenon) of one member fits into the corresponding slot (mortise) of the other.
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.

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.
OUTER DIAMETER	Diameter of a cross-section of a piece of bamboo measured from two opposite points on the outer surface.
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,
PERMISSIBLE STRESS	Stress obtained after applying factor of safety to the ultimate or basic stress.
PRINCIPAL RAFTER	A roof member which supports purlins.
PURLINS	A roof member directly supporting roof covering or common rafter and roof battens.
ROOF BATTENS	A roof member directly supporting tiles, corrugated sheets, slates or other roofing materials.
ROOF SKELETON	The skeleton consisting of bamboo truss or rafter over which solid bamboo purlins are laid and lashed to the rafter or top chord of a truss by means of galvanized iron wire, cane, grass, bamboo leaves, etc.
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).
SPLITS	The pieces made from quarters by dividing the quarters radially and cutting longitudinally.
TAPER	The ratio of difference between minimum and maximum outer diameter to length.
UNLOADED END DISTANCE	The end distance opposite to the loaded end
WALL THICKNESS	Half the difference between outer diameter and inner diameter of the piece at any cross-section.
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.

4.2.3 Definitions Relating to Defects

BAMBOO BORE/ GHOON HOLE	The defect caused by bamboo GHOON beetle (Dinoderus spp. Bostychdae), which attacks felled culms.
CROOKEDNESS	A localized deviation from the straightness in a piece of bamboo.
DISCOLORATION	A change from the normal colour of the bamboo which does not impair the strength of bamboo or bamboo composite products.

4.2.4 Definitions Relating to Drying Degrades

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.
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.
SURFACE CRACKING	Fine surface cracks not detrimental to strength, However, the cracking which occurs at the nodes reduces the structural strength.
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.

4.3 Symbols

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), $= \frac{\pi}{4}\left(D^2 - d^2\right)$, mm^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 = $\frac{\pi}{64}\left(D^2 - d^2\right)$, mm^4

l = Unsupported length of column, m or mm

M = Moisture content, %

r = Radius of gyration = $\sqrt{(I/A)}$, mm

R' = Modulus of rupture, N/mm^2

W = Wall thickness, mm

Z = Section modulus, mm^3

δ = Deflection or deformation, mm.

4.4 Materials

4.4.1 Species of Bamboo

In Bangladesh, four species are widely used, hence studied for the mechanical properties as tabulated in Table 6.4.1-6.4.4 for top, bottom and middle positions. Table 6.4.5 further summarize the average mechanical properties of 21 bamboo species.

Table 6.4.1: Moisture content and specific gravity values of bamboo species

Species	Moisture content (%)			Specific Gravity (based on oven dry weight and at different volumes)					
				Green volumes			Oven dry volumes		
	bottom	middle	top	bottom	middle	top	bottom	middle	top
Kali (Oxytenanthera nigrociliata)	129	118	104	0.48	0.49	0.51	0.66	0.69	0.74
Mitinga (Bambusa tulda)	108	92	86	0.54	0.58	0.61	0.75	0.79	0.83
Bethua (Bambusa polymorpha)	104	93	79	0.55	0.57	0.61	0.79	0.81	0.54
Borak (Bambusa balcooa)	100	84	66	0.57	0.64	0.74	0.79	0.84	0.85

Table 6.4.2: Shrinkages of wall thickness and diameter of bamboo species

Species	Shrinkage in wall thickness (%)						Shrinkage in diameter (%)		
	From green to 12% mc			From green to oven dry condition			From green to 12% mc		
	bottom	middle	top	bottom	middle	top	bottom	middle	top
Kali (Oxytenanthera nigrociliata)	9.6	8.1	5.9	13.2	10.7	8.7	4.8	3.0	2.4
Mitinga (Bambusa tulda)	11.9	7.3	4.9	14.9	9.6	7.6	3.9	3.5	2.6
Bethua (Bambusa polymorpha)	10.7	6.5	5.1	12.1	10.1	8.2	7.3	5.5	4.1
Borak (Bambusa balcooa)	11.1	7.6	4.8	13.7	11.1	8.4	4.2	3.4	2.5

Table 6.4.3: Compressive strength of bamboo species

Species	Compression parallel to the grain (kg/cm^2)					
	Green			Air dry		
	bottom	middle	top	bottom	middle	top
Kali (Oxytenanthera nigrociliata)	257	287	301	346	387	417
Mitinga (Bambusa tulda)	403	466	513	529	596	620
Bethua (Bambusa polymorpha)	320	361	419	452	512	534
Borak (Bambusa balcooa)	394	459	506	510	536	573

Table 6.4.4: Modulus of elasticity and modulus of rupture values of bamboo species

Species	Modulus of elasticity (1000 kg/cm^2)						Modulus of rapture (kg/cm^2)					
	Green			Air dry			Green			Air dry		
	bottom	middle	top	bottom	middle	top	bottom	middle	top	bottom	middle	top
Kali (Oxytenanthera nigrociliata)	119	131	169	131	150	224	541	459	415	721	580	530
Mitinga (Bambusa tulda)	105	138	147	114	140	168	710	595	542	883	745	671
Bethua (Bambusa polymorpha)	61	65	82	60	70	96	469	426	373	566	468	414
Borak (Bambusa balcooa)	72	92	103	93	108	127	850	712	624	926	787	696

4.4.2 Grouping

Sixteen species of bamboo are suitable for structural applications and classified into three groups, namely, Group A, Group B and Group C as given in Table 6.4.6.

The characteristics of these groups are as given in Table 6.4.6.

Species of bamboo other than those listed in the Table 6.4.6 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 in Table 6.4.5.

4.4.3 Moisture Content in Bamboo

With decrease of moisture content (M) the strength of bamboo increases exponentially and bamboo has an intersection point (fibre saturation point) at around 25 percent moisture content depending upon the species. Matured culms shall be seasoned to about 20 percent moisture content before use.

Table 6.4.5: Physical and Mechanical Properties of Bamboos (in Round Form)

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
Bambusa auriculata	594	65.1	15.01	36.7	670	89.1	21.41
B. balcooa	740	64.2	7.06	38.6	850	68.3	9.12
B. bambos (Syn.B.atwndinacea)	559	58.3	5.95	35.3	663	80.1	8.96
B. burmanica	570	59.7	11.01	39.9	672	105.0	17.81
B. glancescens (Syn.B.nana)	691	82.8	14.77	53.9	—	—	—
B. nutans	603	52.9	6.62	45.6	673	52.4	10.72
B. pallida	731	55.2	12.90	54.0	—	—	—
B. polymorpha	610	36.6	6.0	31.4	840	40.6	5.89
B. tulda	610	53.2	10.3	39.5	830	65.8	11.18
B. ventricosa	626	34.1	3.38	36.1	—	—	—
B. vulgaris	626	41.5	2.87	38.6	—	—	—
Cephalostachyum pergracile	601	52.6	11.16	36.7	640	71.3	19.22
Dendrocalamus giganteous	597	17.2	0.61	35.2	—	—	—
D. hamiltonii	515	40.0	2.49	43.4	—	—	—
D. longispathus	711	33.1	5.51	42.1	684	47.8	6.06
D. membranacaus	551	26.3	2.44	40.5	664	37.8	3.77
D. strictus	631	73.4	11.98	35.9	728	119.1	15.00
Melocanna baccifera	817	53.2	11.39	53.8	751	57.6	12.93
Oxytenanthera abyssinicia	688	83.6	14.96	46.6	—	—	—
Oxytenanthera nigrociliata	510	40.70	11.7	25.2	830	51.98	12.85
Thyrsostachys oliveri	733	61.9	9.72	46.9	758	90.0	12.15

4.4.3.1 Air seasoning of split or half-round bamboo does not pose much problem but care has to be taken to prevent fungal discoloration and decay. However, rapid drying in open sun can control decay due to fungal and insect attack. Seasoning in round form presents considerable problem as regards mechanical degrade due to drying defects.

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.

4.4.3.2 Accelerated air seasoning method gives good results. In this method, the nodal diaphragms (septa) are punctured to enable thorough passage of hot air from one end of the resulting bamboo tube to the other end.

4.4.4 Grading of Structural Bamboo

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.

Table 6.4.6: Safe Working Stresses of Bamboos for Structural Designing[(1)]

Species	Extreme Fibre Stress in Bending N/mm^2	Modulus of Elasticity $10^3 N/mm^2$	Allowable Compressive Stress N/mm^2
GROUP A			
Barnbusa glancescens (syn. B. nana)	20.7	3.28	15.4
Dendrocalamus strictus	18.4	2.66	10.3
Oxytenanthera abyss inicia	20.9	3.31	13.3

Species	Extreme Fibre Stress in Bending N/mm^2	Modulus of Elasticity $10^3 N/mm^2$	Allowable Compressive Stress N/mm^2
GROUP B			
Bambusa balcooa	16.05	1.62	13.3
B. pallida	13.8	2.87	15.4
B. nutans	13.2	1.47	13.0
B. tulda	13.3	1.77	11.6
B. auriculata	16.3	3.34	10.5
B. burmanica	14.9	2.45	11.4
Cephalostachyum pergraci[e	13.2	2.48	10.5
Melocanna baccifera (Syn. M. bambusoides)	13.3	2.53	15.4
Thyrsotachys oliveri	15.5	2.16	13.4
GROUP C			
Bambusa arundinacea (Syn. B. bambos)	14.6	1.32	10.1
B. polymorpha	9.15	1.71	8.97
B. ventricosa	8.5	0.75	10.3
B. vulgaris	10.4	0.64	11.0
Dendrocalamus longispathus	8.3	1.22	12.0
Oxytenanthera nigrociliata	10.18	2.6	7.2

(1) The values given pertain to testing of bamboo in green condition.

Table 6.4.7: Limiting Strength Values (in Green Condition)

	Modulus of Rupture (R') N/mm^2	Modulus of Elasticity (E) in Bending $10^3 N/mm^2$
Group A	*R'>70*	*E>9*
Group B	*70≥ R'>50*	*9≥E>6*
Group C	*50≥ R'>30*	*6≥E>3*

Table 6.4.8: 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 x 600 = 9 N/mm^2'.

4.4.4.1 Diameter and length

4.4.4.1.1 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 70mm<Diameter <100mm

Grade I 50mm<Diameter <70mm

Grade II 30mm<Diameter <50mm

Grade III Diameter <30mm

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

4.4.4.1.2 The minimum length of culms shall be preferably 6 m for facilitating close fittings at joints.

4.4.5 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.

4.4.5.1 Curvature

The maximum curvature shall not be more than 75 mm in a length of 6 m of any grade of bamboo.

4.4.5.2 Wall thickness

Preferably minimum wall thickness of 8 mm shall be used for load bearing members.

4.4.5.3 Defects and permissible characteristics

4.4.5.3.1 Dead and immature bamboos, bore/GHOON holes, decay, collapse, checks more than 3 mm in depth, shall be avoided.

4.4.5.3.2 Protruded portion of the nodes shall be flushed smooth. Bamboo shall be used after at least six weeks of felling.

4.4.5.3.3 Broken, damaged and discolored bamboo shall be rejected.

4.4.5.3.4 Matured bamboo of at least 4 years of age shall be used.

4.4.6 Durability and Treatability

4.4.6.1 Durability

The natural durability of bamboo is low and varies between 12 months and 36 months depending on the species and climatic conditions. In tropical countries the bio-deterioration is very severe, Bamboos are generally destroyed in about one to two years' time when used in the open and in contact with ground while a service life of two to five years can be expected from bamboo when used under cover and out of contact with ground. The mechanical strength of bamboo deteriorates rapidly with the onset of fungal decay in the sclerenchymatous fibres. Split bamboo is more rapidly destroyed than round bamboo. For making bamboo durable, suitable treatment shall be given.

4.4.6.2 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.

4.4.6.3 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.

4.4.6.3.1 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.

4.4.6.3.2 For provisions on safety of bamboo structures against fire, see Part 7.

4.5 Permissible Stresses

4.5.1 Factor of Safety

The safety factor for deriving 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

4.5.2 Coefficient of Variation

The coefficient of variation (in percent) shall be as under:

Property	Mean	Range	Maximum Expected Value
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 culms will be at least 53 percent of the average reported value of modulus of rupture in Table 6.4.5.

4.5.3 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.

4.5.4 The safe working stresses for 18 species of bamboos are given in Table 4.5.6

4.5.5 For change in duration of load other than continuous (long-term), the permissible stresses given in Table 4.5.6 shall be multiplied by the modification factors given below:

For imposed or medium term loading	1.25
For short-term loading	1.50

4.6 Design Considerations

4.6.1 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 Chapter 11 Part 6).

4.6.2 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.

4.6.3 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.

4.6.4 Loads

The loads shall be in accordance with Chapter 2 Part 6.

4.6.5 Structural Forms

4.6.5.1 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 skeletons are much better applications of bamboo.

4.6.5.2 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.

4.6.6 Flexural Members

4.6.6.1 All flexural members maybe designed using the principles of beam theory.

4.6.6.2 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.

4.6.6.3 The moment of inertia, I shall be determined as follows:

(a) The outside diameter and the wall thickness should 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.

4.6.6.4 The maximum bending stress shall be calculated and compared with the allowable stress.

4.6.6.5 For shear checks, conventional design procedure in accordance with Chapter 11 Part 6 shall be followed. The basic shear stress values (N/mm^2) for five species of bamboo in split form in green condition can be assumed as under:

Bambusa pallida	9.77
B. Vulgaris	9.44
Dedroculumus giganteous	8.86
D. humiltonii	7.77
Oxytenanthera abyssinicia	11.2

4.6.6.6 Forces acting on a beam, being loads or reaction forces at supports, shall act in nodes or as near to nodes as by any means possible.

4.6.7 Bamboo Column (Predominantly Loaded in Axial Direction)

4.6.7.1 Columns and struts are essential components sustaining compressive forces in a structure. They transfer load to the supporting media.

4.6.7.2 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:

(i) The moment of inertia shall be as per Sec 4.6.6.3.

(ii) 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.

(iii) 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.

4.6.7.3 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 the reduced diameter is not less than 0.6 percent.

4.6.7.4 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.

4.6.8 Assemblies, Roof Trusses

4.6.8.1 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 Figure 6.4.1.(a)]. Bamboo trusses may also be formed using bamboo mat board or bamboo mat-veneer composite or plywood gusset [see Figure 6.4.1(b)].

4.6.8.2 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.

4.6.8.3 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.

4.6.8.4 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.

4.6.8.5 For strength verification of members in compression and connections, the calculated axial forces should be increased by 10 percent.

4.6.8.6 The spacing of trusses shall be consistent with use of bamboo purlins (2 m to 3 m).

4.6.8.7 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.

4.7 Design and Techniques of Joints

4.7.1 Bamboo Joints

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.

4.7.1.1 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.

4.7.1.1.1 Lengthening joints (End Joints)

(a) 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 (Figure 6.4.2).

(b) Butt Joint

Culms of similar diameter are butted end to end, interconnected 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 (Figure 6.4.3).

(c) Sleeves and Inserts

Short length of bamboo of appropriate diameter may be used either externally or internally to join two culms together (Figure 6.4.4).

(d) Scarf Joint

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 (Figure 6.4.5).

4.7.1.1.2 Bearing joints

Bearing joints are formed when members which bear against one another or cross each other and transfer the loads at an angle other than parallel to the axis.

(a) Butt Joint

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 (Figure 6.4.6).

(b) 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 (Figure 6.4.7).

(c) 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 maybe load bearing (see Figure 6.4.8). Such joints are also used to transmit angle thrust.

(d) Angled Joint

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 Figure 6.4.9).

4.7.1.2 Modern practices

Following are some of the modern practices for bamboo jointing (Figure 6.4.10):

(a) Plywood or solid timber gusset plates maybe 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.

4.7.1.3 Fixing methods and fastening devices

In case of butt joints the tie maybe passed through a pre-drilled hole or around hardwood or bamboo pegs or dowels inserted into prefomed 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.

4.7.1.3.1 Normally 1.60 mm diameter galvanized iron wire may be used for tight lashing.

4.7.1.3.2 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.

4.7.1.3.3 Pin And Wire Bound Joints

Generally 12 mm diameter bamboo pins are fastened to culms and bound by 2.00 mm diameter galvanized iron wire.

4.7.1.3.4 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.

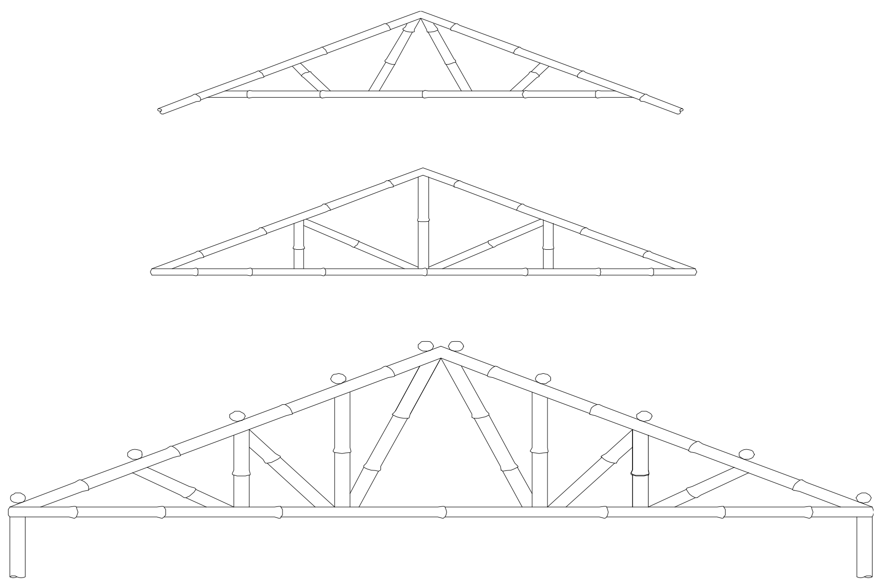
4.7.1.3.5 Horned Joints

Two tongues made at one end of culm may be fastened with across member with its mortise grooves to receive horns, the assembly being wire bound.

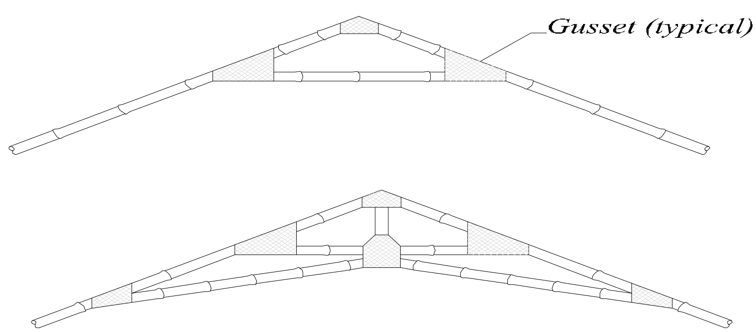
4.7.1.4 For any complete joint alternative for a given load and geometry, description of all fastening elements, their sizes and location shall be indicated. Data shall be based on full scale tests.

4.7.1.5 Tests on full scale joints or on components shall be carried out in a recognized laboratory.

4.7.1.6 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.



6.4.1(a)- Pin ended joint trusses



6.4.1(b)- Gusset joint trusses

Figure 6.4.1 Some typical configurations for small and large trusses in Bamboo

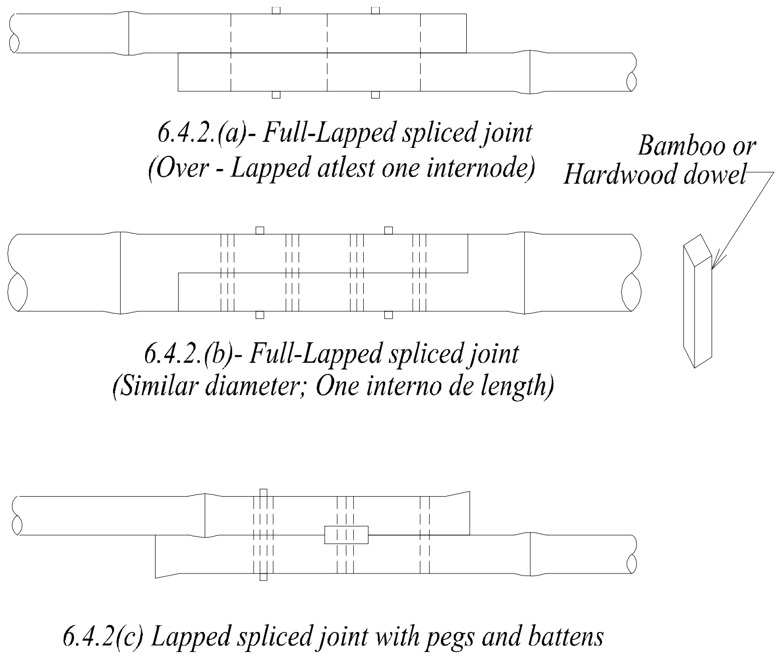


Figure 6.4.2 Lap joint in Bamboo

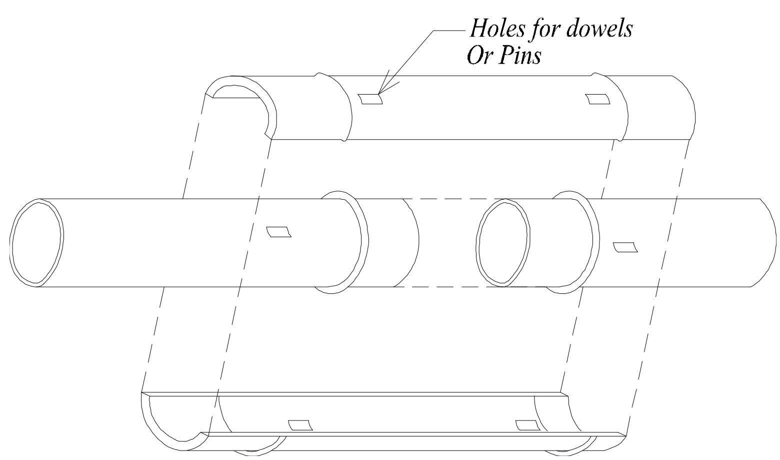


Figure 6.4.3 Butt joint with side plates in Bamboo

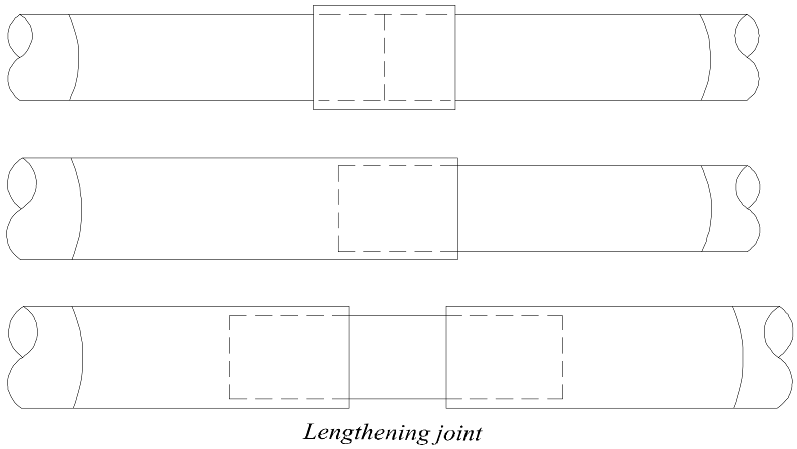


Figure 6.4.4 Sleeves and inserts for Bamboo joint

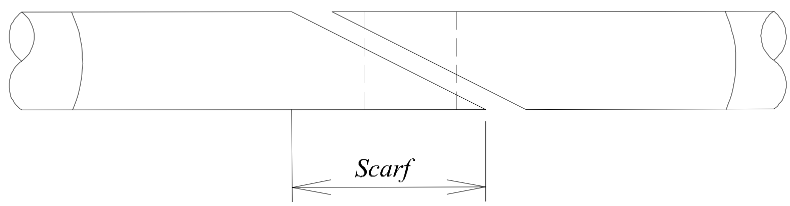


Figure 6.4.5 Scarf joint

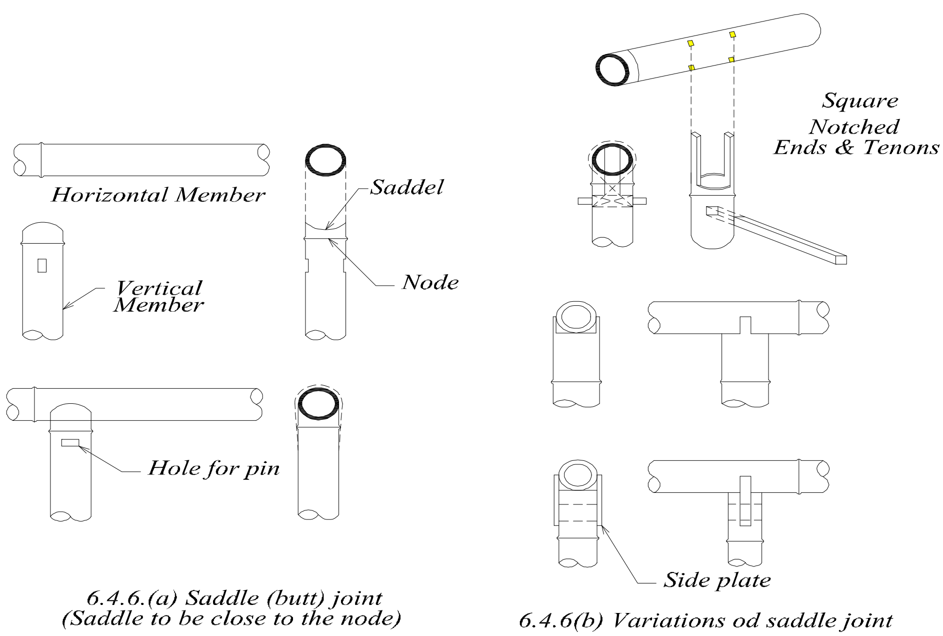
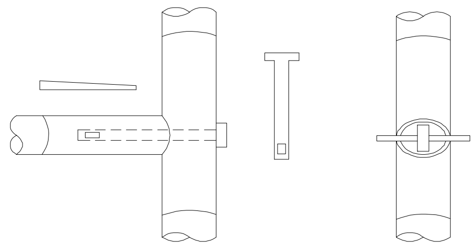
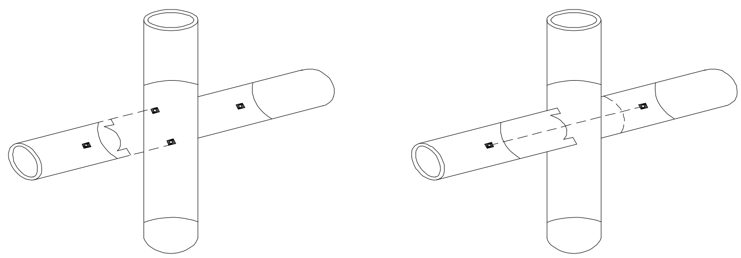


Figure 6.4.6 Butt joints in Bamboo

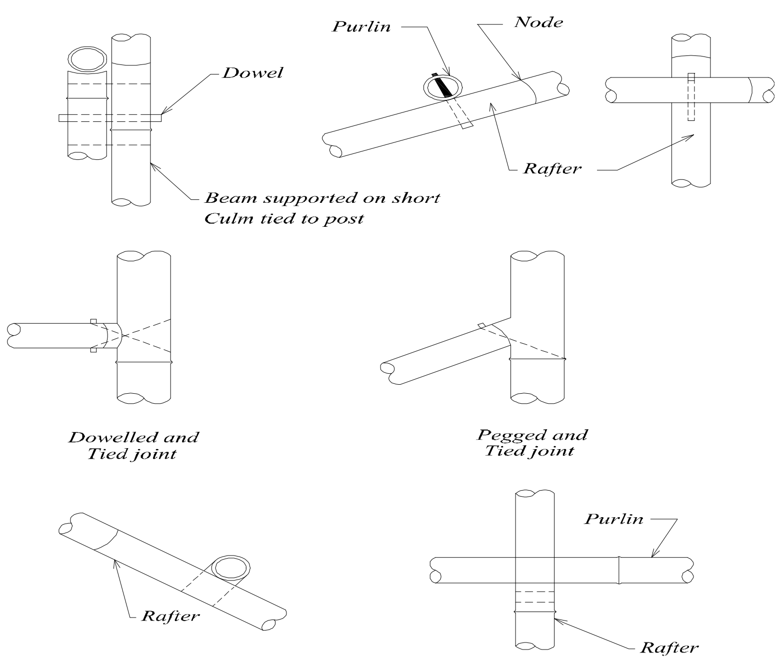


Tenon and key joint



Integral tenon (horned) joint

Figure 6.4.7 Tenon joint



Alternative purlin-Rafter connection

Figure 6.4.8 Cross over joints (Bearing joints)

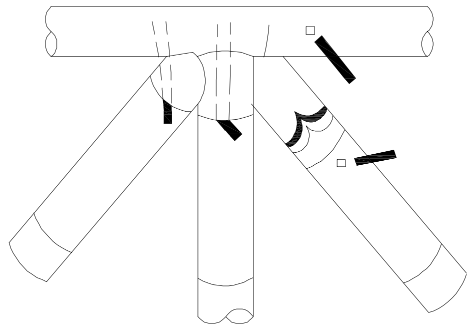


Figure 6.4.9 Angled joints with integral tenons

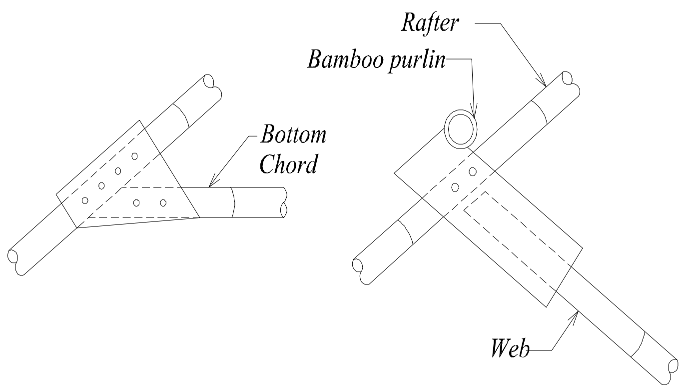


Figure 6.4.10 Gusset plated joint

4.8 Storage of Bamboo

Procurement and storage of bamboo stocks are essential for any project work and shall be done in accordance with Part 7 of this Code.

4.9 Related References

(1) IS 6874: 1973, "Method of Test for Round Bamboo", Bureau of Indian Standards, India, 1974.

(2) IS 9096: 1979, "Code of Practice for Preservation of Bamboo for Structural Purposes", Bureau of Indian Standards, India, 1974.

(3) Salehuddin, A. B. M., "Unnoto Poddhotite Bash Shongrokkhon o Babohar", Bangladesh Agriculture Research Institute, 2004.

PART VI
Chapter 5
Concrete Material

5.1 General

5.1.1 Scope

The provisions of this Chapter shall apply to the design of reinforced and prestressed concrete structures specified in Chapters 6, 8, 9 shall be applicable for normal weight aggregate only unless otherwise specified.

5.1.2 Notation

C_c = Creep coefficient
E_c = Modulus of elasticity of concrete
E_s = Modulus of elasticity of reinforcement
E_t = Modulus of elasticity of concrete at the age of loading t
$f_c^{'}$ = Specified compressive strength of concrete
$f_{cr}^{'}$ = Required average compressive strength of concrete used as the basis for selection of concrete proportions
f_y = Specified yield strength of reinforcement
K = Coefficient of shrinkage
s = Standard deviation
W_c = Unit weight of concrete
ε_{cc} = Creep strain in concrete
ε_{sh} = Shrinkage of plain concrete
ρ = Area of steel relative to that of the concrete.

5.2 Constituents Of Concrete

5.2.1 Cement

5.2.1.1 Cement shall conform to one of the following specifications:

(a) "Composition, Specification and Conformity Criteria for Common Cements" (BDS EN 197-1:2003)

(b) "Standard Specification for Portland Cement" (ASTM C150/C150M)

(c) "Standard Specification for Blended Hydraulic Cements" (ASTM C595/C595M)

(d) "Standard Performance Specification for Hydraulic Cement" (ASTM C1157/C1157M)

5.2.1.2 Cement used in the construction shall be the same as that used in the concrete mix design.

5.2.2 Aggregates

5.2.2.1 Concrete aggregates shall conform to the standards “Coarse and Fine Aggregates from Natural Sources for Concrete” (BDS 243: 1963); “Standard Specification for Concrete Aggregates” (ASTM C33/C33M).

5.2.2.2 Maximum nominal size of coarse aggregate shall be the minimum of the following:

(a) One fifth (1/5) the narrowest dimension between sides of forms,

(b) One third (1/3) the depth of slabs,

(c) Three fourth (3/4) the minimum clear spacing between individual reinforcing bars, or bundles of bars, or prestressing tendons or ducts.

The above limitations may be relaxed if, in the judgment of the engineer, workability and methods of consolidation are such that concrete can be placed without honeycomb or voids.

5.2.2.3 Coarse aggregate made from Grade A brick as specified in BDS 208 "Specification for Common Building Clay Bricks" may be used in different types slab and non-structural elements, except in applications where the ambient environmental conditions may impair the performance of concrete made of such aggregates.

5.2.3 Water

5.2.3.1 Water used in mixing concrete shall be clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials, or other substances that may be harmful to concrete or reinforcement.

5.2.3.2 For concrete wherein aluminium members will be embedded, mixing water shall not contain harmful amounts of chloride ion as indicated in Sec 5.5.3.

5.2.3.3 Nonpotable water shall not be used in concrete except the following conditions:

(a) Selection of concrete proportions shall be based on concrete mixes using water from the same source.

(b) Nonpotable water is permitted only if specified comparative mortar test cubes made with nonpotable water produce at least 90 percent of the strength achieved with potable water.

5.2.4 Admixtures

5.2.4.1 Prior approval of the engineer shall be required for the use of admixtures in concrete. All admixtures shall conform to the requirements of this Section and Sec 2.4.5 Chapter 2 Part 5.

5.2.4.2 Admixture used in the work shall be the same as that used in the concrete mix design.

5.2.4.3 Admixtures containing chloride other than impurities from admixture ingredients shall not be used in concrete containing embedded aluminium, or in concrete cast against permanent galvanized metal forms (see Sections 5.5.1.2 and 5.5.2.1).

5.2.4.4 Air entraining admixtures, if used in concrete, shall conform to "Specification for Air entraining Admixtures for Concrete" (ASTM C260).

5.2.4.5 Water reducing admixtures, retarding admixtures, accelerating admixtures, water reducing and retarding admixtures, and water reducing and accelerating admixtures, if used in concrete, shall conform to "Standard Specification for Chemical Admixtures for Concrete" (ASTM C494/C494M) or "Standard Specification for Chemical Admixtures for use in Producing Flowing Concrete" (ASTM C1017/C1017M).

5.2.4.6 Fly ash or other pozzolans used as admixtures shall conform to "Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for use as a Mineral Admixture in Portland Cement Concrete " (ASTM C618).

5.2.4.7 Ground granulated blast-furnace slag used as an admixture shall conform to "Standard Specification for Ground Iron Blast Furnace Slag for use in Concrete and Mortar" (ASTM C989).

5.3 Steel Reinforcement

5.3.1 General

5.3.1.1 Steel reinforcement for concrete shall conform to the provisions of this Section and those of Sec 2.4.6 Chapter 2 Part 5.

5.3.1.2 Modulus of elasticity E_s for reinforcement shall be taken as 200 kN/mm^2.

5.3.1.3 Reinforcing bars to be welded shall be indicated on the drawings and welding procedure to be used shall be specified. Reinforcing bars otherwise conforming to BDS ISO 6935-2, shall also possess material properties necessary to conform to welding procedures specified in "Structural Welding Code - Reinforcing Steel" (AWS D1.4) of the American Welding Society.

5.3.2 Deformed Reinforcement

5.3.2.1 Deformed reinforcing bars shall conform to one of the following specifications:

(a) "Bangladesh Standard Steel for the reinforcement of concrete Part-1; Plain bars" (BDS ISO 6935-1 and "Bangladesh Standard Steel for the reinforcement of concrete Part-2; Ribbed bars" (BDS ISO 6935-2)

(b) "Standard Specification for Deformed and Plain Billet Steel Bars for Concrete Reinforcement" (ASTM A615/A615M),

(c) "Standard Specification for Rail Steel Deformed and Plain Bars for Concrete Reinforcement" Including Supplementary Requirements S1 (ASTM A996/A996M),

(d) "Standard Specification for Axle Steel Deformed and Plain Bars for Concrete Reinforcement" (ASTM A996/A996M),

(e) "Standard Specification for Low Alloy Steel Deformed Bars for Concrete Reinforcement" (ASTM A706/A706M),

(f) "Specification for Cold Worked Steel Bars for the Reinforcement of Concrete" (BS 4461).

5.3.2.2 Deformed reinforcing bars with a specified yield strength f_y exceeding 420 N/mm^2 shall be permitted, provided f_y shall be the stress corresponding to a strain of 0.35 percent and the bars otherwise conform to one of the ASTM specifications listed in Sec 5.3.2.1 (Also see Sec 6.1.2.5).

5.3.2.3 Galvanized reinforcing bars shall comply with "Standard Specification for Zinc Coated (Galvanized) Steel Bars for Concrete Reinforcement" (ASTM A767/ A767M). Epoxy coated reinforcing bars shall comply with "Standard Specifications for Epoxy Coated Reinforcing Steel Bars" (ASTM A775/A775M). Galvanized or epoxy coated reinforcement shall also conform to one of the standards listed in Sec 5.3.2.1 above.

5.3.3 Plain Reinforcement

5.3.3.1 Plain bars shall conform to one of the specifications listed in Section 5.3.2.1 (a), (b), (c) or (d).

5.3.3.2 Plain wire shall conform to "Standard Specification for Steel Wire, Plain, for Concrete Reinforcement" (ASTM A82/A82M) except that for wire with a specified yield strength f_y exceeding 420 N/mm^2, f_y shall be the stress corresponding to a strain of 0.0035.

5.3.3.3 Plain bars and wire may be used as ties, stirrups and spirals for all structural members and for all reinforcement in structures up to 4-storey high.

5.3.4 Structural Steel, Steel Pipe or Tubing

5.3.4.1 Structural steel used with reinforcing bars in composite compression members meeting the requirements of Sec 6.3.10.8 or Sec 6.3.10.9 of Chapter 6 of this Part shall conform to one of the following specifications:

(a) "Standard Specification for Structural Steel" (ASTM A36/A36M),

(b) "Standard Specification for High Strength Low Alloy Structural Steel"(ASTM A242/A242M),

(c) "Standard Specification for High Strength Low Alloy Structural Manganese Vanadium Steel" (ASTM A572/A572M),

(d) "Standard Specification for High Strength Low Alloy Columbium-Vanadium Steels of Structural Quality" (ASTM A572/A572M),

(e) "Standard Specification of High Strength Low Alloy Structural Steel with 50 ksi (345 Mpa) Minimum Yield Point to 4 in (100 mm) Thick" (ASTM A588/A588M).

5.3.4.2 Steel pipe or tubing for composite compression members composed of a steel encased concrete core meeting the requirements of Sec 6.3.10.7 Chapter 6 of this Part shall conform to one of the following specifications:

(a) Grade B of "Standard Specification for Pipe, Steel, Black and Hot Dipped, Zinc Coated Welded and Seamless" (ASTM A53/A53M).

(b) "Standard Specification for Cold Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes" (ASTM A500/A500M).

(c) "Standard Specification for Hot Formed Welded and Seamless Carbon Steel Structural Tubing" (ASTM A501).

5.4 Workability Of Concrete

Concrete mix proportions shall be such that the concrete is of adequate workability and can properly be compacted. Suggested ranges of values of workability of concrete for some placing conditions, are given in Table 6.5.1.

5.5 Durability Of Concrete

5.5.1 Special Exposures

5.5.1.1 For concrete intended to have low permeability when exposed to water, the water cement ratio shall not exceed 0.50.

5.5.1.2 For corrosion protection of reinforced concrete exposed to brackish water, sea water or spray from these sources, the water cement ratio shall not exceed 0.4.

If minimum concrete cover required by Sec 8.1.8 Chapter 8 of this Part is increased by 12 mm, water cement ratio may be increased to 0.45.

5.5.1.3 The water cement ratio required in Sections 5.5.1.1 and 5.5.1.2 above and Table 6.5.2 shall be calculated using the weight of cement meeting the requirements of BDS EN-197-1 or ASTM C595/C595M or C1157/C1157M, plus the weight of fly ash or pozzolan satisfying ASTM C618 and/or slag satisfying ASTM C989, if any.

5.5.2 Sulphate Exposures

5.5.2.1 Concrete to be exposed to sulphate containing solutions or soils shall conform to the requirements of Table 6.5.2 or be made with a cement that provides sulphate resistance with the maximum water cement ratio provided in Table 6.5.2.

5.5.2.2 Calcium chloride shall not be used as an admixture in concrete exposed to severe or very severe sulphate containing solutions, as defined in Table 6.5.2.

Table 6.5.1: Suggested Workability of Concrete for Various Placing Conditions

Placing Conditions	Degree of Workability	Values of Workability
Concreting of thin sections with vibration	Very low	20-10 seconds Vee-Bee time, or 0.75-0.80 compacting factor
Concreting of lightly reinforced sections with vibration	Low	10-5 seconds Vee-Bee time, or 0.80-0.85 compacting factor
Concreting of lightly reinforced sections without vibration or heavily reinforced section with vibration	Medium	5-2 seconds Vee-Bee time, or 0.85-0.92 compacting factor, or 25-75 mm slump for 20 mm aggregate*
Concreting of heavily rein-forced sections without vibration	High	Above 0.92 compacting factor, or 75-125 mm slump for 20 mm aggregate*

* Slump test shall be performed as per ASTM C143. For smaller aggregates the values will be lower.

Table 6.5.2: Requirements for Normal Weight Aggregate Concrete Exposed to Sulphate Containing Solutions

Sulphate Exposure	Water Soluble Sulphate (SO_4) in Soil, percent by Weight	Sulphate (SO_4) in Water, (ppm)	Cement Type[1]	Maximum Water Cement Ratio, by Weight
Negligible	0.00-0.10	0 – 150	-	-
Moderate[2]	0.10-0.20	150 -1500	Other than CEM I and B type	0.50
Severe	0.20-2.00	1500 -10,000	Other than CEM-I and B type	0.45
Very severe	Over 2.00	Over 10,000	Other than CEM-I and B type	0.45

Notes: Pozzolan that has been determined by test or service record to improve sulphate resistance when used in concrete containing Type V cement.

[1] For types of cement see BDS EN 197-1:2003 or ASTM C150 and C595

[2] Sea water

Table 6.5.3: Maximum Chloride-ion Content for Corrosion Protection

Type of Member	Maximum Water Soluble Chloride Ion (Cl^-) in Concrete, Percent by Weight of Cement
Prestressed concrete	0.06
Reinforced concrete exposed to chloride in service	0.15
Reinforced concrete that will be dry or protected from moisture in service	1.00
Other reinforced concrete construction	0.30

5.5.3 Corrosion of Reinforcement

5.5.3.1 For corrosion protection, maximum water soluble chloride ion concentrations in hardened concrete at ages from 28 to 42 days contributed from the ingredients including water, aggregates, cementitious materials, and admixtures, shall not exceed the limits of Table 6.5.3. When testing is performed to determine water soluble chloride ion content, test procedure shall conform to AASHTO T260, "Methods of Sampling and Testing for Total Chloride Ion in Concrete and Concrete Raw Materials".

5.5.3.2 When reinforced concrete will be exposed to brackish water, sea water, or spray from these sources, requirements of Sections 5.5.1.1 and 5.5.1.2 for water cement ratio, or concrete strength and minimum cover requirements of Sec 8.1.8 Chapter 8 of this Part shall be satisfied.

5.5.4 Minimum Concrete Strength

Minimum concrete strength for structural use of reinforced concrete shall be 20 N/mm^2. However, for buildings up to 4 storey, the minimum concrete strength may be relaxed to 17 N/mm^2.

5.6 Concrete Mix Proportion

5.6.1 General

5.6.1.1 Proportions of materials for concrete shall be such that :

(a) Workability and consistency are achieved for proper placement into forms and around reinforcement, without segregation or excessive bleeding;

(b) Resistance to special exposures to meet the durability requirements of Sec 5.5 are provided; and

(c) Conformance with strength test requirements of Sec 5.12 is ensured.

5.6.1.2 Where different materials are to be used for different portions of the proposed work, each combination shall be evaluated.

5.6.1.3 Concrete proportions, including water cement ratio, shall be established on the basis of field experience and/or trial mixtures with materials to be employed (Sec 5.6.2) except as permitted in Sec 5.6.3 or required by Sec 5.5.

5.6.2 Proportioning Concrete Mix on the Basis of Field Experience and/or Trial Mixtures

5.6.2.1 Standard deviation

(a) A standard deviation shall be established where test records are available in a concrete production facility. Test records from which a standard deviation is calculated shall meet the following requirements :

(i) These shall represent materials, quality control procedures, and conditions similar to those expected for the proposed work. Deviations in materials and proportions for the proposed work shall be more restricted than those within the test records.

(ii) Test records shall represent concrete produced to meet a specified strength f_c' within 7 N/mm^2 of that specified for the proposed work.

(iii) The record shall consist of at least 30 consecutive tests or two groups of consecutive tests totaling at least 30 tests as defined in Sec 5.12.2.4 except as provided in (b) below.

(b) Where a concrete production facility does not have test records meeting the requirements of (a) above but does have a record based on 15 to 29 consecutive tests, a standard deviation shall be established as the product of the calculated standard deviation and the modification factor specified in Table 6.5.4. However, the test records shall meet the requirements (i) and (ii) of (a) above and represent only a single record of consecutive tests that span a period of not less than 45 calendar days.

Table 6.5.4: Modification Factor for Standard Deviation when Less Than 30 Tests are Available

No. of Tests*	Modification Factor for Standard Deviation**
Less than 15	See Section 5.6.2.2(b)
15	1.16
20	1.08
25	1.03
30 or more	1.00

* Interpolate for intermediate numbers of tests

** Modified standard deviation to be used to determine the required average strength f'_{cr} from Sec 5.6.2.2(a).

5.6.2.2 Required average strength

(a) Required average compressive strength f_c' used as the basis for selection of concrete proportions shall be the larger of the values given by Eq (6.5.1) and (6.5.2) using a standard deviation calculated in accordance with Sec 5.6.2.1(a) or Sec 5.6.2.1(b) above.

$$f_{cr}' = f_c' + 1.34s \qquad (6.5.1)$$

$$f_{cr}' = f_c' + 2.33s - 3.5 \qquad (6.5.2)$$

(b) When a concrete production facility does not have field strength test records for calculation of standard deviation meeting the requirements of Sec 5.6.2.1(a) or Sec 5.6.2.1(b), the required average strength shall be determined from Table 6.5.5 and documentation of the average strength shall be in accordance with the requirements of Sec 5.6.2.3 below.

Table 6.5.5: Required Average Compressive Strength when Data are not available
to establish a Standard Deviation

Specified Compressive Strength f'_c N/mm^2	**Required Average Compressive Strength, f'_{cr} N/mm^2**
Less than 20	$f'_c + 7.0$
20 to 35	$f'_c + 8.5$
Over 35	$f'_c + 10.0$

5.6.2.3 Documentation of average strength

Documentation shall be prepared to demonstrate that the proposed concrete proportions will produce an average compressive strength equal to or greater than the required average compressive strength (Sec 5.6.2.2). Such documentation shall consist of one or more field strength test records or trial mixtures.

(a) When test records are used to demonstrate that proposed concrete proportions will produce the required average strength f'_{cr} (Sec 5.6.2.2) such records shall represent materials and conditions similar to those expected. Deviations in materials, conditions and proportions within the test records shall not have been more restricted than those for proposed work. For the purpose of documenting average strength potential, test records consisting of less than 30 but not less than 10 consecutive tests are acceptable provided the test records encompass a period of time not less than 45 days. Required concrete proportions shall be permitted to be established by interpolation between the strengths and proportions of two or more test records each of which meets other requirements of this Section.

(b) When an acceptable record of field test results is not available, concrete proportions may be established based on trial mixtures meeting the following restrictions :

  (i) Combination of materials shall be those for the proposed work.

  (ii) Trial mixtures having proportions and consistencies required for the proposed work shall be made using at least three different water cement ratios or cement contents that will produce a range of strengths encompassing the required average strength.

(iii) Trial mixtures shall be designed to produce a slump within ±20 mm of the maximum permitted, and for air entrained concrete the air content shall be within ±0.5 percent of the maximum allowable.

(iv) For each water cement ratio or cement content, at least three test cylinders for each test age shall be made and cured in accordance with "Method of Making and Curing Concrete Test Specimens in the Laboratory" (ASTM C192/C192M). Cylinders shall be tested at 28 days or at test age designated for the determination of f_c' .

(v) From the results of cylinder tests, a curve shall be plotted showing the relationship between the water cement ratio or cement content and the compressive strength at designated test age.

(vi) Maximum water cement ratio or minimum cement content for concrete to be used in the proposed work shall be that shown by the above curve to produce the average strength required by Sec 5.6.2.2 unless a lower water cement ratio or higher strength is required by Sec 5.5.

5.6.3 Proportioning by Water Cement Ratio

5.6.3.1 If the data required in Sec 5.6.2 are not available, concrete proportions shall be based on water cement ratio limits specified in Table 6.5.6 when approved by the engineer.

5.6.3.2 Table 6.5.6 shall be used for concrete to be made with cements meeting strength requirements of “Bangladesh Standard Cement Part-1: Composition, specifications and conformity criteria for common cements” (BDS EN 197-1: 2003), and shall not be applied to concrete containing lightweight aggregates or admixtures other than those for entraining air.

5.6.3.3 Concrete proportioned by water cement ratio limits prescribed in Table 6.5.6 shall also conform to special exposure requirements of Sec 5.5 and to compressive strength test criteria of Sec 5.12.

5.6.4 Average Strength Reduction

As data become available during construction, amount by which value of f_c' must exceed specified value of f_c' may be reduced, provided:

(a) 30 or more test results are available and the average of test results exceeds that required by Sec 5.6.2.2(a) using a standard deviation calculated in accordance with Sec 5.6.2.1(a), or

(b) 15 to 29 test results are available and the average of test results exceeds that required by Sec 5.6.2.2(a) using a standard deviation calculated in accordance with Sec 5.6.2.1(b), and provided further that special exposure requirements of Sec 5.5 are met.

Table 6.5.6: Maximum Permissible Water Cement Ratios for Concrete when Strength Data from Field Experience or Trail Mixers are not Available

Specified Compressive Strength*, f'_c N/mm^2	Absolute Water Cement Ratio by Weight	
	Concrete other than air-entrained	Air-entrained concrete
17	0.66	0.54
20	0.60	0.49
25	0.50	0.39
30	0.40	**
35	**	**

* 28 day strength. With most materials, water cement ratios shown will provide average strengths greater than that required in Sec 5.6.2.2.

** For strengths above 30 N/mm^2 (25 N/mm^2 for air entrained concrete) concrete proportions shall be established by methods of Sec 5.6.2.

5.7 Preparation of Equipment and Place of Deposit

Preparation before concrete placement shall include the following:

(a) All equipment for mixing and transporting concrete shall be clean.

(b) All debris shall be removed from spaces to be occupied by concrete.

(c) Forms shall be properly cleaned and coated.

(d) Masonry filler units that will be in contact with concrete shall be soaked thoroughly.

(e) Reinforcement shall be thoroughly clean of deleterious coatings.

(f) Water shall be removed from place of deposit before concrete is placed unless a tremie is used or unless otherwise permitted by the engineer.

(g) All laitance and other unsound material shall be removed before additional concrete is placed against hardened concrete.

5.8 Mixing

5.8.1 All concrete shall be mixed thoroughly until there is a uniform distribution of materials and shall be discharged completely before the mixer is recharged.

5.8.2 Ready mixed concrete shall be mixed and delivered in accordance with the requirements of "Standard Specification for Ready Mixed Concrete" (ASTM C94) or "Standard Specification for Concrete Made by Volumetric Batching and Continuous Mixing" (ASTM C685).

5.8.3 Job mixed concrete shall be mixed in accordance with the following:

(a) Mixing shall be done in a batch mixer of approved type.

(b) Mixer shall be rotated at a speed recommended by the manufacturer.

(c) Mixing shall be continued for at least 90 seconds after all materials are in the drum, unless a shorter time is shown to be satisfactory by the mixing uniformity tests of "Specification for Ready Mixed Concrete" (ASTM C94).

(d) Materials handling, batching, and mixing shall conform to the applicable provisions of "Specification for Ready Mixed Concrete" (ASTM C94).

(e) A detailed record shall be kept to identify:

(i) number of batches produced;

(ii) proportions of materials used;

(iii) approximate location of final deposit in structure;

(iv) time and date of mixing and placing.

5.9 Conveying

5.9.1 Concrete shall be conveyed from the mixer to the place of final deposit by methods that will prevent segregation or loss of materials.

5.9.2 Conveying equipment shall be capable of providing a supply of concrete to the place of deposit without segregation of ingredients and without interruptions sufficient to permit loss of plasticity between successive increments.

5.10 Depositing

5.10.1 Concrete shall be deposited as near its final position as practical to avoid segregation due to rehandling or flowing.

5.10.2 Concreting shall be carried on at such a rate that concrete is at all times plastic and flows readily into spaces between and around the reinforcement.

5.10.3 Concrete that has partially hardened or been contaminated by foreign materials shall not be deposited in the structure.

5.10.4 Retempered concrete or concrete that has been remixed after initial set shall not be used.

5.10.5 After concreting is started, it shall be carried on as a continuous operation until placing of a panel or section, as defined by its boundaries or predetermined joints, is completed except as permitted or prohibited by Sec 5.16.4.

5.10.6 Top surfaces of vertically formed lifts shall be generally level.

5.10.7 When construction joints are required, joints shall be made in accordance with Sec 5.16.4.

5.10.8 All concrete shall be thoroughly consolidated by suitable means during placement and shall be thoroughly worked around reinforcement and embedded fixtures and into corners of forms.

5.11 Curing

5.11.1 Concrete (other than high early strength) shall be maintained above 10°C and in a moist condition for at least the first 7 days after placement, except when cured in accordance with Sec 5.11.3.

5.11.2 High early strength concrete shall be maintained above 10°C and in a moist condition for at least the first 3 days, except when cured in accordance with Sec 5.11.3.

5.11.3 Accelerated Curing

5.11.3.1 Curing by high pressure steam, steam at atmospheric pressure, heat and moisture or other accepted processes, shall be permitted to accelerate strength gain and reduce time of curing.

5.11.3.2 Accelerated curing shall provide a compressive strength of the concrete at the load stage considered, at least equal to the required design strength at that load stage.

5.11.3.3 Curing process shall be such as to produce concrete with a durability at least equivalent to that obtained for concrete cured by the method of Sec 5.11.1 or 5.11.2.

5.11.4 When required by the engineer, supplementary strength tests in accordance with Sec 5.12.4 shall be performed to assure that curing is satisfactory.

5.12 Evaluation and Acceptance of Concrete

5.12.1 General

5.12.1.1 Concrete shall be proportioned to provide an average compressive strength as prescribed in Sec 5.6.2.2 as well as to satisfy the durability criteria of Sec 5.5. Concrete shall be produced to limit frequency of strengths below f_c' to that prescribed in Sec 5.12.3.3.

5.12.1.2 Requirements of shall be based on tests of cylinders made and tested as prescribed in Sec 5.12.3.

5.12.1.3 Unless otherwise specified, f_c' shall be based on 28 day tests. Test age for f_c' shall be indicated in design drawings or specifications, if it is different from 28 days.

5.12.1.4 Splitting tensile strength tests shall not be used as a basis for field acceptance of concrete.

5.12.2 Frequency of Testing

5.12.2.1 Samples for strength tests of each class of concrete placed each day shall be taken not less than once a day, nor less than once for each 60 m^3 of concrete, nor less than once for each 250 m^2 surface area for slabs or walls.

5.12.2.2 On a given project, if the total volume of concrete is such that frequency of testing required by Sec 5.12.2.1 above would provide less than three strength tests for a given class of concrete, tests shall be made from at least three randomly selected batches or from each batch if three or fewer batches are used.

5.12.2.3 When the total quantity of a given class of concrete is less than 20 m^3, strength tests are not required when evidence of satisfactory strength is submitted to and approved by the Engineer.

5.12.2.4 A strength test shall be the average of the strengths of at least two 150 mm by 300 mm cylinders or at least three 100 mm by 200 mm cylinders made from the same sample of concrete and tested at 28 days or at test age designated for determination of f_c'.

5.12.3 Laboratory Cured Specimens

5.12.3.1 Samples for strength tests shall be taken in accordance with "Method of Sampling Freshly Mixed Concrete" (ASTM C172).

5.12.3.2 Cylinders for strength tests shall be moulded and laboratory cured in accordance with "Practice for Making and Curing Concrete Test Specimens in the Field" (ASTM C31/C31M) and tested in accordance with "Test Method for Compressive Strength of Cylindrical Concrete Specimens" (ASTM C39/C39M).

5.12.3.3 Strength level of an individual class of concrete shall be considered satisfactory if both of the following requirements are met :

(a) Average of three consecutive strength tests (see Sec 5.12.2.4) equals or exceeds f_c'

(b) No individual strength test (average of two cylinders of 150 mm by 300 mm or average of three cylinders of 100 mm by 200 mm) falls below by more than 3.5 N/mm^2.

5.12.3.4 If either of the requirements of Sec 5.12.3.3 are not met, steps shall be taken to increase the average of the subsequent strength test results. Requirements of Sec 5.12.5 shall be satisfied if the requirement of Sec 5.12.3.3(b) is not met.

5.12.4 Field Cured Specimens

5.12.4.1 The engineer may require strength tests of cylinders cured under field conditions to check adequacy of curing and protection of concrete in the structure.

5.12.4.2 Field cured cylinders shall be cured under field conditions in accordance with "Practice for Making and Curing Concrete Test Specimens in the Field" (ASTM C31/C31M).

5.12.4.3 Field cured test cylinders shall be moulded at the same time and from the same samples as laboratory cured test cylinders.

5.12.4.4 Procedures for protecting and curing concrete shall be improved when the strength of field cured cylinders at the test age designated for determination of f_c' is less than 85 percent of that of companion laboratory cured cylinders. The 85 percent limitation shall not apply if field cured strength exceeds f_c' by more than 3.5 N/mm^2.

5.12.5 Investigation of Low Strength Test Results

5.12.5.1 If the result of any strength test (Sec 5.12.2.4) of laboratory cured cylinders falls below the specified value of by more than 3.5 N/mm^2 (Sec 5.12.3.3(b)) or if tests of field cured cylinders indicate deficiencies in protection and curing (Sec 5.12.4.4), steps shall be taken to assure that the load carrying capacity of the structure is not jeopardized.

5.12.5.2 If the likelihood of low strength concrete is confirmed and computations indicate that load carrying capacity may have been significantly reduced, tests of cores drilled from the area in question may be required in accordance with "Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete" (ASTM C42/C42M). In such cases, three cores shall be taken for each strength test more than 3.5 N/mm^2 below the specified value of f_c'.

5.12.5.3 If concrete in the structure is expected to be dry under service conditions, cores shall be air dried for 7 days before test and shall be tested dry. If concrete in the structure is expected to be more than superficially wet under service conditions, cores shall be immersed in water for at least 40 hours and be tested wet.

5.12.5.4 Concrete in an area represented by core tests shall be considered structurally adequate if the average of three cores is equal to at least 85 percent of $f_c^{'}$ and if no single core is less than 75 percent of $f_c^{'}$. Additional testing of cores extracted from locations represented by erratic core strength results shall be permitted.

5.12.5.5 If the criteria of Sec 5.12.5.4 above are not met, and if structural adequacy remains in doubt, the responsible authority may order load tests for the questionable portion of the structure, or take other appropriate action.

5.13 Properties of Concrete

5.13.1 Strength

Strength of concrete shall be based on $f_c^{'}$ determined in accordance with the provisions of Sec 5.12.1.

5.13.2 Modulus of Elasticity

5.13.2.1 Modulus of elasticity E_c for stone aggregate concrete may be taken as $44w_c^{1.5}\sqrt{f_c^{'}}$ (N/mm^2) for values of w_c between 15 and 25 kN/m^3 and $f_c^{'}$ in N/mm^2. For normal density concrete, E_c may be taken as $4700\sqrt{f_c^{'}}$.

5.13.2.2 Modulus of elasticity E_c for brick aggregate concrete may be taken as $3750\sqrt{f_c^{'}}$.

5.13.3 Creep

The final (30 year) creep strain in concrete ε_{cc} shall be predicted from

$$\varepsilon_{cc} = \frac{stress}{E_t} c_c \qquad (6.5.3)$$

Where,

E_t is the modulus of elasticity of the concrete at the age of loading *t*,

c_c is the creep coefficient.

The creep coefficient may be estimated from Figure 6.5.1. In this Figure, for uniform sections, the effective section thickness is defined as twice the cross-sectional area divided by the exposed perimeter. If drying is prevented by immersion in water or by sealing, the effective section thickness shall be taken as 600 mm.

It can be assumed that about 40%, 60% and 80% of the final creep develops during the first month, 6 months and 30 months under load respectively, when concrete is exposed to conditions of constant relative humidity.

5.13.4 Shrinkage

An estimate of the drying shrinkage of plain concrete may be obtained from Figure 6.5.2. Recommendations for effective section thickness and relative humidity are given in Sec 5.13.3.

Figure 6.5.2 relates to concrete of normal workability made without water reducing admixtures; such concretes shall have an original water content of about 190 litre/m^3. Where concrete is known to have a different water content, shrinkage shall be regarded as proportional to water content within the range 150 to 230 litre /m^3.

The shrinkage of plain concrete is primarily dependent on the relative humidity of the air surrounding the concrete, the surface area from which moisture can be lost relative to the volume of concrete and on the mix proportion. It is increased slightly by carbonation and self-desiccation and reduced by prolonged curing. An estimate of the shrinkage of symmetrically reinforced concrete sections may be obtained from:

$$\frac{\varepsilon_{sh}}{1+k_\rho} \qquad (6.5.4)$$

Where,

ε_{sh} is the shrinkage of the plain concrete;

ρ is the area of steel relative to that of the concrete;

k_ρ is a coefficient, taken as 25 for internal exposure and as 15 for external exposure.

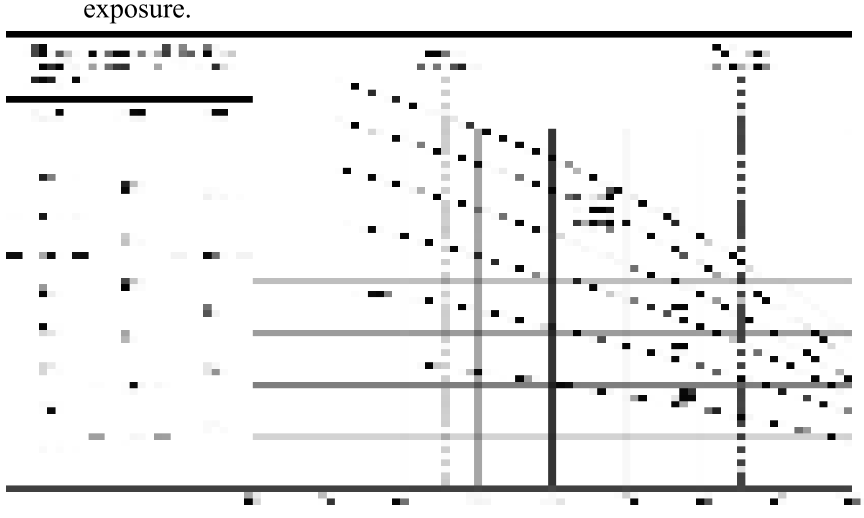


Figure 6.5.1 Effects of relative humidity, age of loading and section thickness upon creep factor

5.13.5 Thermal Strains

Thermal strains shall be calculated from the product of a suitable coefficient of thermal expansion and a temperature change. The temperature change can be determined from the expected service conditions and climatic data. Externally exposed concrete does not respond immediately to air temperature change, and climatic temperature ranges may require adjustment before use in movement calculations.

The coefficient of thermal expansion of concrete is dependent mainly on the expansion coefficients for the aggregate and the cement paste, and the degree of saturation of the concrete. The thermal expansion of aggregate is related to mineralogical composition (See Table 6.5.7)

Cement paste has a coefficient of thermal expansion that is a function of moisture content, and this affects the concrete expansion as shown in Fig 6.5.3. It may be seen that partially dry concrete has a coefficient of thermal expansion that is approximately $2 \times 10^{-6}/^{o}C$ greater than the coefficient for saturated concrete.

5.14 Concreting in Adverse Weather

5.14.1 Concreting shall be avoided during periods of near freezing weather.

5.14.2 During hot weather, proper attention shall be given to ingredients, production methods, handling, placing, protection, and curing to prevent excessive concrete temperatures or water evaporation that could impair required strength or serviceability of the member or structure.

5.14.3 During rainy weather, proper protection shall be given to ingredients, production methods, handling and placing of concrete. If required in the opinion of the engineer, the concreting operation shall be postponed and newly placed concrete shall be protected from rain after forming proper construction joint for future continuation.

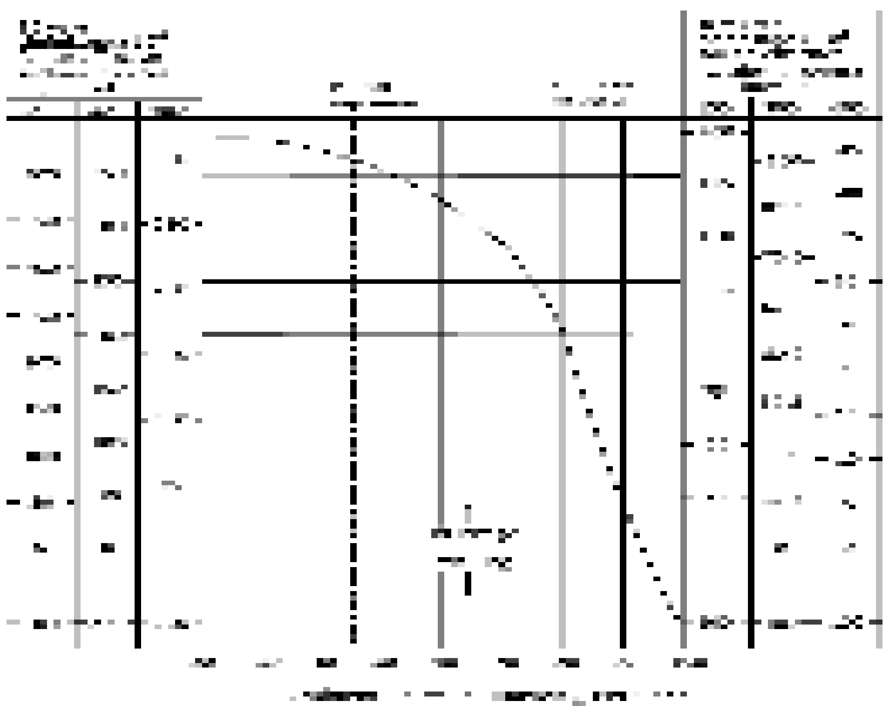


Figure 6.5.2 Drying shrinkage of normal-weight concrete

Table 6.5.7: Thermal Expansion of Rock Group and Related Concrete

Aggregate Type	Typical Coefficient of Expansion ($1 \times 10^{-6}/^{o}C$)	
	Aggregate	**Concrete**
Flint, quartzite	11	12
Granite, basalt	7	10
Limestone	6	8

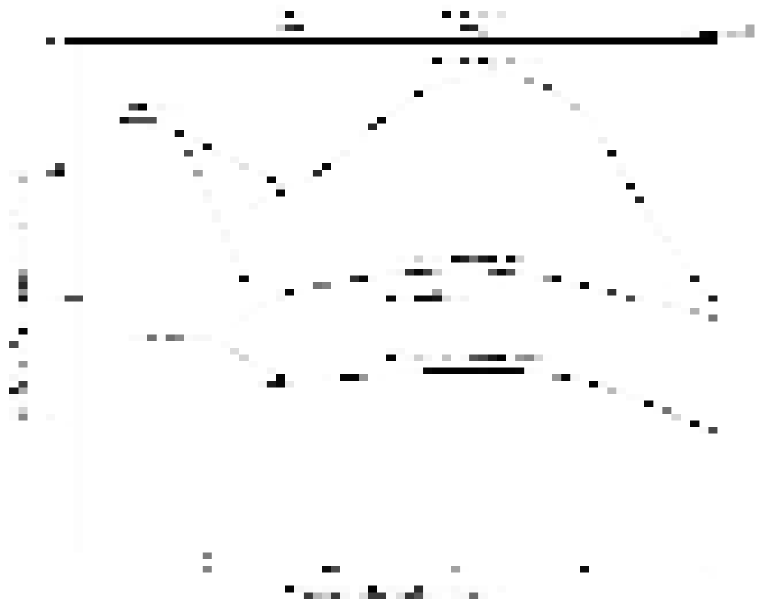


Figure 6.5.3 Effect of dryness upon the coefficient of thermal expansion of hardened cement and concrete

5.15 Surface Finish

5.15.1 Type of Finish

A wide variety of finishes can be produced. Surface cast against forms may be left as cast, e.g. plain or profiled, the initial surface may be removed, e.g. by tooling or sandblasting, or the concrete may be covered, e.g. by paint or tiles; combinations of these techniques may also be adopted, e.g. a ribbed profile with bush hammered ribs. Upper surfaces not cast against forms may be trowelled smooth or profiled, e.g. by tamping; the initial surface may be removed, e.g. by spraying, or it may be covered, e.g. by a screed or plastic floor finish. When selecting the type of finish, consideration shall be given to the ease of producing a finish of the required standard, the viewing distance and the change of appearance with time. In the case of external surfaces, account shall be taken of the weather pattern at the particular location, any impurities in the air and the effect of the shape of the structure upon the flow of water across its surface. Such considerations will often preclude the specification of surfaces of uniform colour as these are very difficult to produce and deteriorate with time, particularly if exposed to the weather.

5.15.2 Quality of Finish

A high quality finish is one that is visually pleasing; it may include colour variations and physical discontinuities but these are likely to be distributed systematically or randomly over the whole surface rather than being concentrated in particular areas. When deciding on the quality of finish to be specified, consideration should be given to the viewing distance and the exposure conditions.

There is no method whereby the quality of finish that will be accepted can unequivocally be defined. To achieve the quality required calls for good communication between experienced personnel conversant with the production of finishes and close collaboration with the site. The quality of finish can be identified in the following very broad terms:

(a) Class 2 applies to surfaces that are to be exposed to view but where appearance is not critical; such surfaces might be the walls of fire escape stairs or plant rooms and columns and beams of structures that are normally viewed in the shade, e.g. car parks and warehouses;

(b) Class 1 is appropriate to most surfaces exposed to view including the external walls of industrial, commercial and domestic buildings;

(c) Special class is appropriate to the highest standards of appearance, such as might be found in prestigious buildings, where it is possible to justify the high cost of their production.

(d) These broad descriptions may be amplified by written descriptions of the method of finish, by photographs, by samples or by reference to existing structures.

5.15.3 Type of Surface Finish

Smooth off-the-form and board marked finishes are not recommended for external use, but where they are specified for interior use the following types may be quoted for the guidance of both designers and contractor. Designers should appreciate that it is virtually impossible to achieve dense, flat, smooth, even coloured blemish free concrete surfaces directly from the form work. Some degree of making good is inevitable, even with precast work.

(a) Type A finish: This finish is obtained by the use of properly designed formwork or moulds of timber, plywood, plastics, concrete or steel. Small blemishes caused by entrapped air or water may be expected, but the surface should be free from voids, honeycombing or other blemishes.

(b) Type B finish: This finish can only be obtained by the use of high quality concrete and formwork. The concrete shall be thoroughly compacted and all surfaces shall be true, with clean arises. Only very minor surface blemishes shall occur, with no staining or discoloration from the release agent.

(c) Type C finish: This finish is obtained by first producing a type B finish. The surface is then improved by carefully removing all fins and other projections, thoroughly washing down, and then filling the most noticeable surface blemishes with a cement and fine aggregate paste to match the colour of the original concrete. The release agent should be carefully chosen to ensure that the concrete surface will not be stained or discoloured. After the concrete has been properly cured, the face shall be rubbed down, where necessary, to produce a smooth and even surface.

5.15.4 Production

The quality of a surface depends on the constituents and proportions of the concrete mix, the efficiency of mixing, the handling and compaction of the concrete and its curing. The characteristics of the formwork and the release agent may also be of critical importance. Requirements may be stated for any aspect of production that might contribute towards the achievement of the required type of quality of finish.

5.15.5 Inspection and Making Good

The surface of the concrete shall be inspected for defects and for conformity with the specification and, where appropriate, for comparison with approved sample finishes. Subject to the strength and durability of the concrete being unimpaired, the making good of surface defects may be permitted but the standard of acceptance shall be appropriate to the type and quality of the finish specified and ensure satisfactory performance and durability. On permanently exposed surfaces great care is essential in selecting the materials and the mix proportions to ensure that the final colour of the faced area blends with the parent concrete in the finished structure.

Voids can be filled with fine mortar, preferably incorporating styrene butadiene rubber (SBR) or polyvinyl acetate (PVA), while the concrete is still green or when it has hardened. Fine cracks can be filled by wiping a cement grout, an SBR, PVA or latex emulsion, a cement/SBR or a cement/PVA slurry across them. Fins and other projections shall be rubbed down.

5.15.6 Protection

High quality surface finishes are susceptible to damage during subsequent construction operations and temporary protection may have to be provided in vulnerable areas. Examples of such protective measures include the strapping of laths to arrises and the prevention of rust being carried from exposed starter bars to finished surfaces.

5.16 Formwork

5.16.1 Design of Formwork

5.16.1.1 Forms shall result in a final structure that conforms to shapes, lines, and dimensions of the members as required by the design drawings and specifications.

5.16.1.2 Forms shall be substantial and sufficiently tight to prevent leakage of mortar.

5.16.1.3 Forms shall be properly braced or tied together to maintain position and shape.

5.16.1.4 Forms and their supports shall be designed so as not to damage previously placed structure.

5.16.1.5 Design of formwork shall include consideration of the following factors:

(a) Rate and method of placing concrete;

(b) Construction loads, including vertical, horizontal and impact loads;

(c) Special form requirements for construction of shells, folded plates, domes, architectural concrete, or similar types of elements.

5.16.1.6 Forms for prestressed concrete members shall be designed and constructed to permit movement of the member without damage during application of prestressing force.

5.16.2 Removal of Forms and Shores

5.16.2.1 No construction loads shall be supported on, nor any shoring removed from, any part of the structure under construction except when that portion of the structure in combination with remaining forming and shoring system has sufficient strength to support safely its weight and loads placed thereon.

5.16.2.2 Sufficient strength shall be demonstrated by structural analysis considering proposed loads, strength of forming and shoring system, and concrete strength data. Structural analysis and concrete strength test data shall be furnished to the engineer when so required.

5.16.2.3 No construction loads exceeding the combinations of superimposed dead load plus specified live load shall be supported on any unshored portion of the structure under construction, unless analysis indicates adequate strength to support such additional loads.

5.16.2.4 Forms shall be removed in such a manner as not to impair safety and serviceability of the structure. All concrete to be exposed by form removal shall have sufficient strength not to be damaged thereby.

5.16.2.5 Forms supporting prestressed concrete members shall not be removed until sufficient prestressing has been applied to enable prestressed members to carry their dead load and anticipated construction loads.

5.16.3 Conduits and Pipes Embedded in Concrete

5.16.3.1 Conduits, pipes and sleeves of any materials not harmful to concrete and within the limitations specified herein shall be permitted to be embedded in concrete with the approval of the engineer, provided they are not considered to replace structurally the displaced concrete.

5.16.3.2 Conduits and pipes of aluminium shall not be embedded in structural concrete unless effectively coated or covered to prevent aluminium concrete reaction or electrolytic action between aluminium and steel.

5.16.3.3 Conduits, pipes, and sleeves passing through a slab, wall, or beam shall not impair significantly the strength of the construction.

5.16.3.4 Conduits and pipes, with their fittings, embedded within a column shall not displace more than 4 percent of the area of cross-section on which strength is calculated or which is required for fire protection.

5.16.3.5 Except when drawings for conduits and pipes are approved by the engineer, conduits and pipes embedded within a slab, wall or beam (other than those merely passing through) shall satisfy the following:

(a) They shall not be larger in outside dimension than one third (1/3) the overall thickness of slab, wall, or beam in which they are embedded.

(b) They shall not be spaced closer than 3 diameters or widths on centre.

(c) They shall not impair significantly the strength of the construction.

5.16.3.6 Conduits, pipes and sleeves shall be permitted to be considered as replacing structurally in compression the displaced concrete provided :

(a) They are not exposed to rusting or other deterioration.

(b) They have nominal inside diameter not over 50 mm and are spaced not less than 3 diameters on centres.

5.16.3.7 Pipes and fittings shall be designed to resist effects of the material, pressure, and temperature to which they will be subjected.

5.16.3.8 No liquid, gas, or vapour, except water not exceeding 30°C nor 0.3 N/mm^2 pressure, shall be placed in the pipes until the concrete has attained its design strength.

5.16.3.9 In solid slabs, piping, unless it is for radiant heating, shall be placed between the top and bottom reinforcements.

5.16.3.10 Concrete cover for pipes, conduits, and fittings shall be not less than 40 mm for concrete exposed to earth or weather, nor 20 mm for concrete not exposed to weather or in contact with ground.

5.16.3.11 Reinforcement with an area not less than 0.002 times the area of concrete section shall be provided normal to piping.

5.16.3.12 Piping and conduit shall be so fabricated and installed that cutting, bending, or displacement of reinforcement will not be required.

5.16.4 Construction Joints

5.16.4.1 Surface of concrete construction joints shall be cleaned and laitance removed.

5.16.4.2 Immediately before new concrete is placed, all construction joints shall be wetted and standing water removed.

5.16.4.3 Construction joints shall be so made and located as not to impair the strength of the structure. Provision shall be made for transfer of shear and other forces through construction joints. See Sec 6.4.5.9.

5.16.4.4 Construction joints in floors shall be located within the middle third of spans of slabs, beams and girders. Joints in girders shall be offset a minimum distance of two times the width of intersecting beams.

5.16.4.5 Beams, girders, or slabs supported by columns or walls shall not be cast or erected until concrete in the columns or walls is no longer plastic.

5.16.4.6 Beams, girders, haunches, drop panels and capitals shall be placed monolithically as part of a slab system unless otherwise shown in the design drawings or specifications.

5.17 Shotcrete

5.17.1 General

Shotcrete shall be defined as mortar or concrete pneumatically projected at high velocity onto a surface. Except as specified in this Section, shotcrete shall conform to the provisions of this Code regarding plain concrete or reinforced concrete.

5.17.2 Proportions and Materials

Shotcrete proportions shall be such that suitable placement is ensured using the delivery equipment selected, and shall result in finished in place hardened shotcrete meeting the strength requirements of Chapter 6.

5.17.3 Aggregate

Coarse aggregate, if used, shall not exceed 20 mm in size.

5.17.4 Reinforcement

The maximum size of reinforcement shall be 16 mm Ø bars unless it can be demonstrated by preconstruction tests that adequate embedment of larger bars can be achieved. When 16 mm Ø or smaller bars are used, there shall be a minimum clearance of 60 mm between parallel reinforcing bars. When bars larger than 16 mm Ø are permitted, there shall be a minimum clearance between parallel bars equal to six diameters of the bars used. When two curtains of steel are provided, the curtain nearest the nozzle shall have a spacing equal to 12 bar diameters and the remaining curtain shall have a minimum spacing of 6 bar diameters.

Lap splices in reinforcing bars shall be by the noncontact lap splice method with at least 50 mm clearance between bars. The engineer may permit the use of contact lap splices when necessary for the support of the reinforcement, provided it can be demonstrated by means of preconstruction testing that adequate embedment of the bars at the splice can be achieved and provided further that the splices are placed so that the plane containing the centres of the two spliced bars is perpendicular to the surface of the shotcrete work. Shotcrete shall not be applied to spirally tied columns.

5.17.5 Preconstruction Tests

When required by the engineer a test panel shall be shot, cured, cored or sawn, examined and tested prior to commencement of the project. The sample panel shall be representative of the project and simulate job conditions as closely as possible. The panel thickness and reinforcing shall reproduce the thickest and the most congested area specified in the structural design. It shall be shot at the same angle, from a similar distance, using the same nozzleman and with the same concrete mix design that will be used on the project.

5.17.6 Rebound

Any rebound or accumulated loose aggregate shall be removed from the surfaces to be covered prior to placing the initial or any succeeding layers of shotcrete. Rebound shall not be reused as aggregate.

5.17.7 Joints

Except where permitted, unfinished work shall not be allowed to stand for more than 30 minutes unless all edges are sloped thin. Before placing additional material adjacent to previously applied work, sloping and square edges shall be cleaned and wetted.

5.17.8 Damage

An in-place shotcrete which exhibits sags or sloughs, segregation, honeycombing, sand pockets or other obvious defects shall be removed and replaced.

5.17.9 Curing

During the curing periods, shotcrete shall be maintained above 5^{O} C and in moist condition. In initial curing, shotcrete shall be kept continuously moist for 24 hours after placement is complete. Final curing shall continue for seven days after shotcreting, for three days if high early strength cement is used, or until the specified strength is obtained. Final curing shall consist of a fog spray or an approved moisture retaining cover or membrane. In sections of a depth in excess of 300 mm, final curing shall be the same as that for initial curing.

5.17.10 Strength Test

Strength test for shotcrete shall be made by an approved agency on three representative specimens of Core or Cube that have been water soaked for at least 24 hours prior to testing. When the maximum size of aggregate is larger than 10 mm, core specimens shall not be less than 75 mm in diameter or the size of cube specimen shall not be less than 75 mm. When the maximum size of aggregate is 10 mm or smaller, core specimens shall not be less than 50 mm in diameter or the size of cube specimen shall not be less than 50 mm. Specimens shall be taken in accordance with one of the following provisions:

(a) From work: taken at least one from each shift but not less than one for each 20 m^3 of shotcrete;

(b) From test panels: taken not less than once each shift nor less than one for each 20 m^3 of shotcrete placed. When the maximum size aggregate is larger than 10 mm, the test panels shall have a minimum dimension of 450 mm by 450 mm. When the maximum size aggregate is 10 mm or smaller, the test panels shall have a minimum dimension of 300 mm by 300 mm. Panels shall be gunned in the same position as the work, during the course of the work and by the same nozzlemen doing the work. The condition under which the panels are cured shall be the same as the work.

The average strength of three cores from a single panel shall be equal to or exceed $0.85\ f_c'$ with no single core less than $0.75\ f_c'$. The average strength of three cubes taken from a single panel must equal or exceed f_c' with no individual cube less than f_c' . To check testing accuracy, locations represented by erratic core strengths may be retested.

5.17.11 Inspections

5.17.11.1 Inspection during placement

When shotcrete is used for columns and beams, a special inspector is required. The special inspector shall provide continuous inspection to the placement of the reinforcement and shotcreting and shall submit a statement indicating compliance with the plans and specifications.

5.17.11.2 Visual examination for structural soundness of in-place shotcrete

Completed shotcrete work shall be checked visually for reinforcing bar embedment, voids, rock pocket, sand streaks and similar deficiencies by examining a minimum of three 75 mm cores taken from three areas chosen by the engineer which represent the worst congestion of reinforcing bars occurring in the project. Extra reinforcing bars may be added to non-congested areas and cores may be taken from these areas. The cores shall be examined by the special inspector and a report submitted to the engineer prior to final approval of the shotcrete.

5.17.12 Equipment

The equipment used in construction testing shall be the same equipment used in the work requiring such testing unless substitute equipment is approved by the Engineer.

PART VI
Chapter 6
Strength Design Of Reinforced Concrete Structures

6.1 Analysis and Design - General Considerations

6.1.1 Definitions

The following terms are defined for general use in this Code. Specialized definitions appear in individual chapters.

COLUMN	Member with a ratio of height- to least lateral dimension exceeding 3 used primarily to support axial compression load. For a tapered member the least lateral dimension is the average of the top and bottom dimensions of the smaller side.
COMPRESSION CONTROLLED SECTIONS	A cross section in which the net tensile strain in the extreme tension steel at nominal strength is less than or equal to the compression-controlled strain limit.
COMPRESSION CONTROLLED STRAIN LIMIT	The net tensile strain at balanced strain condition. See Sec 6.3.3.3.
CONCRETE	Mixture of Portland cement or any other hydraulic cement, fine aggregate, coarse aggregate and water with or without admixture.
CONCRETE, LIGHTWEIGHT	Concrete containing lightweight aggregate and an equilibrium density as determined by ASTM C567, between 1450 - 1850 kg/m^3.
CONCRETE, NORMALWEIGHT	Concrete containing only aggregate that conforms to ASTM C33.
CONCRETE, SPECIFIED COMPRESSIVE STRENGTH OF f_c'	Compressive strength of concrete used in design and evaluated in accordance with provisions of Sec 5.12, expressed in N/mm^2.
CONNECTION	A region that joins two or more members.
CONTRACTION JOINT	Formed, sawed, or tooled groove in a concrete structure to create a weakened plane and regulate the location of cracking resulting from the dimensional change of different parts of the structure.

COVER, SPECIFIED CONCRETE	The distance between the outermost surface of embedded reinforcement and the closest outer surface of the concrete indicated on design drawing or in project specification.
DESIGN DISPLACEMENT	The total lateral displacement expected for the design-basis earthquake, as required by provisions of the Code for earthquake resistant design.
DESIGN LOAD COMBINATION	Combination of factored loads and forces. See Sec 2.7.
DESIGN STORY DRIFT RATIO	Relative difference of design displacement between top and bottom of a story divided by the story height.
DEVELOPMENT LENGTH	Length of embedded reinforcement, required to develop the design strength of reinforcement at a critical section. See Sec 8.2.
DROP PANEL	A projection below the slab used to reduce the amount of negative reinforcement over a column or the minimum required slab thickness, and to increase the slab shear strength. See Sec 6.5.
EFFECTIVE DEPTH OF SECTION	Distance measured from extreme compression fibre to centroid of longitudinal tension reinforcement.
EMBEDMENT LENGTH	Length of embedded reinforcement provided beyond a critical section.
EQUILIBRIUM DENSITY	Density of lightweight concrete after exposure to a relative humidity 50 ± 5 percent and temperature of 73.5 ± 3.5^0 F for a period of time sufficient to reach constant density (see ASTM C567)
EXTREME TENSION STEEL	The reinforcement that is the farthest from the extreme compression fibre.
ISOLATION JOINT	A separation between adjoining parts of a concrete structure, usually a vertical plane, at a designed location such as to interfere least with performance of the structure, yet such as to allow relative movement in three directions and avoid formation of cracks elsewhere in the concrete and through which all or part of the bonded reinforcement is interrupted.

JOINT	Portion of structure common to intersecting members. The effective cross sectional area of a joint of a special moment frame, A_j for shear strength computation is defined in Sec 8.3.7.3.
LICENSED DESIGN PROFESSIONAL	An individual who is licensed to practice structural design as defined by the statutory requirements of the professional licensing laws of the state or jurisdiction in which the project is to be constructed and who is in responsible charge of the structural design.
LOAD, FACTORED	Load, multiplied by appropriate factor, used to proportion members by strength design method of this Code.
MODULUS OF ELASTICITY	Ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material.
PEDESTAL	Member with a ratio of height- to-least lateral dimension less than or equal to 3 used primarily to support axial compression load. For a tapered member the least lateral dimension is the average of the top and bottom dimensions of the smaller side.
PLAIN CONCRETE	Structural concrete with no reinforcement or with less reinforcement than the minimum amount specified for reinforced concrete.
PLASTIC HINGE REGION	Length of frame element over which flexural yielding is intended to occur due to earthquake design displacement, extending not less than a distance h from the critical section where flexural yielding occurs.
PRECAST CONCRETE	Structural concrete element cast elsewhere than its final position in the structure.
REINFORCED CONCRETE	Structural concrete reinforced with no less than the minimum amount of reinforcement specified in the Code.
SEISMIC HOOK	A hook on a stirrup, or cross tie having a bend not less than 135^{o}, except that circular hoops shall have a bend not less than 90^{o}. Hooks shall have a $6d_b$ (but not less than 75 mm) extension that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop.

SPIRAL REINFORCEMENT	Continuously wound reinforcement in the form of a cylindrical helix.
SPLITTING TENSILE STRENGTH (f_{ct})	Tensile strength of concrete determined in accordance with ASTM C496 as described in ASTM C330.
STIRRUPS	Reinforcement used to resist shear and torsion stresses in a structural member, typically bars, wires, or welded wire reinforcements either single leg or bent into L, U, or rectangular shapes and located perpendicular to or at an angle to longitudinal reinforcement. (The term “stirrups” is usually used to lateral reinforcement in flexural members and the term “ties” to those in compression members.
STRENGTH DESIGN	Nominal strength multiplied by a strength reduction factor Ø.
STRENGTH, NOMINAL	Strength of a member or cross section calculated in accordance with provisions and assumptions of the strength design method of this Code before application of any strength reduction factor.
STRENGTH, REQUIRED	Strength of a member or cross section required to resist factored loads or related internal moments and forces in such combination as are stipulated in this Code.
STRUCTURAL CONCRETE	All concrete used for structural purpose including plain and reinforced concrete.
TENSION CONTROLLED SECTION	A cross section in which the net tensile strain in the extreme tensile steel at nominal strength is greater than or equal to 0.005.
TIE	Loop of reinforcing bar or wire enclosing longitudinal reinforcement. A continuously wound bar or wire in the form of a circle, rectangle or other polygon shape without re-entrant corner is acceptable.
YIELD STRENGTH	Specified minimum yield strength or yield point of reinforcement. Yield strength or yield point shall be determined in tension according to applicable ASTM standards.

6.1.2 Notation and Symbols

Unless otherwise explicitly stated, the following units shall be implicit for the corresponding quantities in the design and other expressions provided in this Chapter:

Lengths	mm
Areas	mm^2
Second moments of area	mm^4
Force (axial, shear)	N
Moment, torsion	N-mm
Stress, strength	MPa, N/mm^2

The following notation apply to Chapters 6 and 8, and Appendices A, I, J, K and L of this Part.

a = Depth of equivalent rectangular stress block as defined in Sec 6.3.2.7.1; (mm)

a_v = Shear span, equal to distance from center of concentrated load to either: (a) face of support for continuous or cantilevered members, or (b) center of support for simply supported members, mm, Sec 6.4 and Appendix I

A_b = Area of an individual bar or wire, mm^2, Sec 8.2

A_{brg} = Net bearing area of the head of stud, anchor bolt, or headed deformed bar, mm^2, Sections 8.2.17 and K.5.3

A_c = Cross-sectional area of concrete section resisting shear transfer, mm^2, Sec 6.4.5.5

A_{ch} = Cross-sectional area of a structural member measured to the outside edges of transverse reinforcement, mm^2, Sections 6.3.9, 8.3.5.4

A_{cp} = Area enclosed by outside perimeter of concrete cross section, mm^2, see Sections 6.4.4 and 8.3.8.3

A_{cs} = Cross-sectional area at one end of a strut in a strut-and-tie model, taken perpendicular to the axis of the strut, mm^2, Sec I.3.1 Appendix I.

A_{cv} = Gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, mm^2, Sec 8.3.6.2

A_{cw} = Area of concrete section of an individual pier, horizontal wall segment, or coupling beam resisting shear, mm^2, Sec 8.3.6

A_f = Area of reinforcement in bracket or corbel resisting factored moment, mm^2, see Sec 6.4.7

A_g = Gross area of concrete section, mm^2 For a hollow section, A_g is the area of the concrete only and does not include the area of the void(s), see Sections 6.2, 6.3, 6.4, 6.6, 6.7, 6.10, 8.3.5

A_h = Total area of shear reinforcement parallel to primary tension reinforcement in a corbel or bracket, mm^2, see Sec 6.4.7

A_j = Effective cross-sectional area within a joint in a plane parallel to plane of reinforcement generating shear in the joint, mm^2, see Sec 8.3.7

A_l = Total area of longitudinal reinforcement to resist torsion, mm^2, Sec 6.4

$A_{l,min}$ = Minimum area of longitudinal reinforcement to resist torsion, mm^2, see Sec 6.4.4.5.3

A_n = Area of reinforcement in bracket or corbel resisting tensile force N_{uc}, mm^2, see Sec 6.4.7

A_{nz} = Area of a face of a nodal zone or a section through a nodal zone, mm^2, Sec I.5 Appendix I

A_{Nc} = Projected concrete failure area of a single anchor or group of anchors, for calculation of strength in tension, mm^2, see Sec K.5.2.1, Appendix K

A_{Nco} = Projected concrete failure area of a single anchor, for calculation of strength in tension if not limited by edge distance or spacing, mm^2, see Sec K.5.2.1, Appendix K

A_o = Gross area enclosed by shear flow path, mm^2, Sec 6.4

A_{oh} = Area enclosed by centerline of outermost closed transverse torsional reinforcement, mm^2, Sec 6.4

A_s = Area of nonprestressed longitudinal tension reinforcement, mm^2, Sections 6.3, 6.4, 6.6, 6.8,

A_{s1} = Area of tension reinforcement corresponding to moment of resistance M_{n1} , see Sec 6.3.15.1(b)

A_{s2} = Area of additional tension steel, see Sec 6.3.15.1(b)

A'_s = Area of compression reinforcement, mm^2, Sec I.3.5 Appendix I

A_{sc} = Area of primary tension reinforcement in a corbel or bracket, mm^2, see Sec 6.4.7.3.5

$A_{se,N}$ = Effective cross-sectional area of anchor in tension, mm^2, Sec K.5.1 Appendix K

$A_{se,V}$ = Effective cross-sectional area of anchor in shear, mm^2, Sec K. 6.1 Appendix K

A_{sf} = Area of reinforcement required to balance the longitudinal compressive force in the overhanging portion of the flange of a T-beam, see Sec 6.3.15.2(b)

A_{sh} = Total cross-sectional area of transverse reinforcement (including crossties) within spacing s and perpendicular to dimension h_c, mm^2, Sec 8.3.5

A_{si} = Total area of surface reinforcement at spacing s_i in the i-th layer crossing a strut, with reinforcement at an angle α_i to the axis of the strut, mm^2, Sec I.3.3 Appendix I

$A_{s,min}$ = Minimum area of flexural reinforcement, mm^2, see Sec 6.3.5

A_{st} = Total area of nonprestressed longitudinal reinforcement (bars or steel shapes), mm^2, Sec 6.3.3

A_{sx} = Area of structural steel shape, pipe, or tubing in a composite section, mm^2, Sec 6.3

A_t = Area of one leg of a closed stirrup resisting torsion within spacing s, mm^2, Sec 6.4

A_{tr} = Total cross-sectional area of all transverse reinforcement within spacing s that crosses the potential plane of splitting through the reinforcement being developed, mm^2, Sec 8.2.3

A_{ts} = Area of nonprestressed reinforcement in a tie, mm^2, Sec I.4.1 Appendix I

A_v = Area of shear reinforcement spacing s, mm^2, Sections 6.4, 6.12

A_{Vc} = Projected concrete failure area of a single anchor or group of anchors, for calculation of strength in shear, mm^2, see Sec K.6.2.1 Appendix K

A_{Vco} = Projected concrete failure area of a single anchor, for calculation of strength in shear, if not limited by corner influences, spacing, or member thickness, mm^2, see Sec K.6.2.1 Appendix K

A_{vd} = Total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam, mm^2, Sec 8.3.6

A_{vf} = Area of shear-friction reinforcement, mm^2, Sec 6.4.5

A_{vh} = Area of shear reinforcement parallel to flexural tension reinforcement within spacing s_2, mm^2, Sec 6.4

$A_{v,min}$ = Minimum area of shear reinforcement within spacing s, mm^2, see Sec 6.4.3.5

A_1 = Loaded area, mm^2, Sec 6.3

A_2 = Area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal, mm^2, Sec 6.3

b = Width of compression face of member, mm, Sec 6.3

b_o = Perimeter of critical section for shear in slabs and footings, mm, see Sec 6.4.10.1.2

b_s = Width of strut, mm, Sec I.3.3 Appendix I

b_t = Width of that part of cross section containing the closed stirrups resisting torsion, mm, Sec 6.4

b_v = Width of cross section at contact surface being investigated for horizontal shear, mm, Sec 6.12

b_w = Web width, or diameter of circular section, mm, Sections 6.3, 6.4, 8.2, 8.3.4

b_1 = Dimension of the critical section b_o measured in the direction of the span for which moments are determined, mm, Sec 6.5

b_2 = Dimension of the critical section b_o measured in the direction perpendicular to b_1, mm, Sec 6.5

c = Distance from extreme compression fiber to neutral axis, mm, Sections 6.2, 6.3, 6.6, 8.3.6

C_a, C_b = Moment coefficients, Sec 6.5.8

c_{ac} = Critical edge distance required to develop the basic concrete breakout strength of a post- installed anchor in uncracked concrete without supplementary reinforcement to control splitting, mm, see Sec K.8.6 Appendix K

$c_{a,max}$ = Maximum distance from center of anchor shaft to the edge of concrete, mm, Sec K.5.2.3 Appendix K

$c_{a,min}$ = Minimum distance from center of anchor shaft to the edge of concrete, mm, Sec K.8.6 Appendix K

c_{a1} = Distance from the center of an anchor shaft to the edge of concrete in one direction, mm. If shear is applied to anchor, c_{a1} is taken in the direction of the applied shear. If tension is applied to the anchor, c_{a1} is the minimum edge distance, Sec K.5.2 Appendix K

c_{a2} = Distance from center of an anchor shaft to the edge of concrete in the direction perpendicular to c_{a1}, mm, Sec K.5.4 Appendix K

c_b = Smaller of: (a) the distance from center of a bar or wire to nearest concrete surface, and (b) one-half the center-to-center spacing of bars or wires being developed, mm, Sec 8.2.3

c_c = Clear cover of reinforcement, mm, see Sec 6.3.6.4

c_1 = Dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, mm, Sections 6.4, 6.5, 8.3.4

c_2 = Dimension of rectangular or equivalent rectangular column, capital, or bracket measured in the direction perpendicular to c_1, mm, Sec 6.5

C = Cross-sectional constant to define torsional properties of slab and beam, see Sec 6.5.6.4.2

C_m = Factor relating actual moment diagram to an equivalent uniform moment diagram, Sec 6.3

d = Distance from extreme compression fiber to centroid of longitudinal tension reinforcement, mm, Sections 6.2, 6.3, 6.4, 6.6, 6.12, 8.1.5, 8.2.7, 8.3.4

d' = Distance from extreme compression fiber to centroid of longitudinal compression reinforcement, mm, Sec 6.2

d_a = Outside diameter of anchor or shaft diameter of headed stud, headed bolt, or hooked bolt, mm, see Sec K.8.4, Appendix K

d'_a = Value substituted for d_a when an oversized anchor is used, mm, see Sec K.8.4, Appendix K

d_{pile} = Diameter of pile at footing base, mm, Sec 6.8

d_t = Distance from extreme compression fiber to centroid of extreme layer of longitudinal tension steel, mm, Sections 6.2, 6.3

D = Dead loads, or related internal moments and forces, Sections 6.1, 6.2, 6.11

e_h = Distance from the inner surface of the shaft of a J- or L-bolt to the outer tip of the J- or L-bolt, mm, Sec K.5.3 Appendix K

e'_N = Distance between resultant tension load on a group of anchors loaded in tension and the Centroid of the group of anchors loaded in tension, mm; e'_N is always positive, Sec K.5.2 Appendix K

e'_V = Distance between resultant shear load on a group of anchors loaded in shear in the same direction, and the centroid of the group of anchors loaded in shear in the same direction, mm; e'_V is always positive, Sec K.6.2 Appendix K

E = Load effects of earthquake, or related internal moments and forces, Sections 6.2, 8.3.6

E_c = Modulus of elasticity of concrete, MPa see Sec 6.1.7.1, 6.2, 6.3, 6.6, 6.9

E_{cb} = Modulus of elasticity of beam concrete, MPa, Sec 6.5

E_{cs} = Modulus of elasticity of slab concrete, MPa, Sec 6.5

EI = Flexural stiffness of compression member,N·mm2, see Sec 6.3.10.6

E_s = Modulus of elasticity of reinforcement and structural steel, MPa, see Sections 6.1.7.2, 6.3, 6.6

f'_c = Specified compressive strength of concrete, MPa, Sections 6.1 to 6.4, 6.6, 6.9, 8.2, 8.3, Appendices I, K

f_{ce} = Effective compressive strength of the concrete in a strut or a nodal zone, MPa, Sec 6.8.5, I.3.1 Appendix I

f_{ct} = Average splitting tensile strength of lightweight concrete, MPa, See Sec 6.1.8.1 Sections 6.1, 6.4, 8.2.3.4

f_d = Stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, MPa, Sec 6.4

f_{pc} = Compressive stress in concrete at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, MPa. (In a composite member, f_{pc} is the resultant compressive stress at centroid of composite section, or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone), Sec 6.4

f_r = Modulus of rupture of concrete, MPa, see Sections 6.2.5, 6.6

f_s = Calculated tensile stress in reinforcement at service loads, MPa, Sec 6.3

f_s' = Stress in compression reinforcement under factored loads, MPa, Sec I.3.5 Appendix I

f_{uta} = Specified tensile strength of anchor steel, MPa, Appendix K

f_y = Specified yield strength of reinforcement, MPa, Sections 6.2 to 6.4, 6.6, 6.9, 6.12, 8.1 to 8.3, I.4.1

f_{ya} = Specified yield strength of anchor steel, MPa, Sec K.4.4 Appendix K

f_{yt} = Specified yield strength $f_y$of transverse reinforcement, MPa, Sections 6.3, 6.4, 8.3.3.4

F = Loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or related internal moments and forces, Sec 6.2

F_n = Nominal strength of a strut, tie, or nodal zone, N, Sec I.2.6 Appendix I

F_{nn} = Nominal strength at face of a nodal zone, N, Sec I.5.1 Appendix I

F_{ns} = Nominal strength of a strut, N, Sec I.3.1 Appendix I

F_{nt} = Nominal strength of a tie, N, Sec I.4.1 Appendix I

F_u = Factored force acting in a strut, tie, bearing area, or nodal zone in a strut-and-tie model, N, Sec I.2.6 Appendix I

h = Overall thickness or height of member, mm, Sections 6.2 to 6.4, 6.6, 6.11, 6.12, 8.1.6, 8.3.4, I.1

h_a = Thickness of member in which an anchor is located, measured parallel to anchor axis, mm, Sec K.6.2 Appendix K

h_c = Cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area A_{sh},mm, Sec 8.3.5

h_{ef} = Effective embedment depth of anchor, mm, see Sec K.5.2, Appendix K

h_f = Thickness of overhanging portion of the flange of a T-beam, Sec 6.3.15.2(b)

h_v = Depth of shear head cross section, mm, Sec 6.4

h_w = Height of entire wall from base to top or height of the segment of wall considered, mm, Sections 6.4, 8.3.6

h_x = Maximum center-to-center horizontal spacing of crossties or hoop legs on all faces of the column, mm, Sec 8.3.5

H = Loads due to weight and pressure of soil, water in soil, or other materials, or related internal moments and forces, Sec 6.2

I = Moment of inertia of section about centroidal axis, mm^4, Sections 6.3, 6.4

I_b = Moment of inertia of gross section of beam about centroidal axis, mm^4, Sec 6.5.6

I_{cr} = Moment of inertia of cracked section transformed to concrete, mm^4, Sec 6.2

I_e = Effective moment of inertia for computation of deflection, mm^4, Sec 6.2.5

I_g = Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, mm^4, Sections 6.2, 6.3, 6.6

I_s = Moment of inertia of gross section of slab about centroidal axis defined for calculating α_f and β_t, mm^4, Sec 6.5

I_{se} = Moment of inertia of reinforcement about centroidal axis of member cross section, mm^4, Sec 6.3

I_{sx} = Moment of inertia of structural steel shape, pipe, or tubing about centroidal axis of composite member cross section, mm^4, Sec 6.3

k = Effective length factor for compression members, Sections 6.3, 6.6

k_c = Coefficient for basic concrete breakout strength in tension, Sec K.5.2 Appendix K

k_{cp} = Coefficient for pryout strength, Sec K.6.3 Appendix K

K_{tr} = Transverse reinforcement index, Sec 8.2.3.3

l = Span length of beam or one-way slab; clear projection of cantilever, mm, Sec 6.2

l_a = Additional embedment length beyond centerline of support or point of inflection, mm, Sec 8.2.8

l_a = Length of clear span in short direction, Sec 6.5.8

l_b = Length of clear span in long direction, Sec 6.5.8

l_c = Length of compression member in a frame, measured center-to-center of the joints in the frame, mm, Sections 6.3, 6.6

l_d = Development length in tension of deformed bar, deformed wire, plain and deformed welded wire reinforcement, or mm, Sections 6.9, 8.2.3, 8.3.6

l_{dc} = Development length in compression of deformed bars and deformed wire, mm, Sec 8.2.4

l_{dh} = Development length in tension of deformed bar or deformed wire with a standard hook, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus inside radius of bend and one bar diameter), mm, see Sections 8.2.6, 8.3.6

l_{dt} = Development length in tension of headed deformed bar, measured from the critical section to the bearing face of the head, mm, Sections 8.2.17, 8.3.6

l_e = Load bearing length of anchor for shear, mm, Sec K.6.2.2, Appendix K

l_n = Length of clear span measured face-to-face of supports, mm, Sections 6.1 to 6.5, 6.10, 8.2.9, 8.3.4

l_o = Length, measured from joint face along axis of structural member, over which special transverse reinforcement must be provided, mm, Sec 8.3.5

l_t = Span of member under load test, taken as the shorter span for two-way slab systems, mm. Span is the smaller of: (a) distance between centers of supports, and (b) clear distance between supports plus thickness *h* of member. Span for a cantilever shall be taken as twice the distance from face of support to cantilever end, Sec 6.11

l_u = Unsupported length of compression member, mm, Sec 6.3.10

l_v = Length of shear head arm from centroid of concentrated load or reaction, mm, Sec 6.4

l_w = Length of entire wall or length of segment of wall considered in direction of shear force, mm, Sections 6.4, 6.6, 8.3.6

l_1 = Length of span in direction that moments are being determined, measured center-to-center of supports, mm, Sec 6.5

l_1 = Length of clear span in direction that moment are being determined, Sec 6.5.8

l_2 = Length of clear span transverse to l_1, Sec 6.5.8

l_2 = Length of span in direction perpendicular to l_1, measured center-to-center of supports, mm, Sec 6.5.6

L = Live loads, or related internal moments and forces, Sections 6.1, 6.2, 6.11, 8.3.12

L_r = Roof live load, or related internal moments and forces, Sec 6.2

M_a = Maximum moment in member due to service loads at stage deflection is computed, N·mm, Sections 6.2, 6.6

M_a = Moment in the short direction, Sec 6.5.8

M_b = Moment in the long direction, Sec 6.5.8

M_c = Factored moment amplified for the effects of member curvature used for design of compression member, N·mm, see Sec 6.3.10.6

M_{cr} = Cracking moment, N·mm, see Sec 6.2.5.2.3, Sections 6.2, 6.6

M_{cre} = Moment causing flexural cracking at section due to externally applied loads, N·mm, Sec 6.4

M_m = Factored moment modified to account for effect of axial compression, N·mm, Sec 6.4.2

M_{max} = Maximum factored moment at section due to externally applied loads, N·mm, Sec 6.4

M_n = Nominal flexural strength at section, N·mm, Sections 6.4, 6.6, 8.2.8, 8.3.12

M_{n1} = Nominal flexural strength at section without compression steel, see Sec 6.3.15.1(b), and moment of resistance developed by compression in the overhanging portion of the T-flange, Sec 6.3.15.2

M_{n2} = Additional nominal flexural strength at section due to added compression steel A'_s and additional tension steel A_{s2}, Sec 6.3.15.1, and moment of resistance developed by the web of a T-beam, Sec 6.3.15.2

M_o = Total factored static moment, N·mm, Sec 6.5

M_p = Required plastic moment strength of shear head cross section, N·mm, Sec 6.4

M_{pr} = Probable flexural strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile stress in the longitudinal bars of at least $1.25f_y$ and a strength reduction factor, ϕ, of 1.0, N·mm, Sec 8.3.8

M_s = Factored moment due to loads causing appreciable sway, N·mm, Sec 6.3

M_u = Factored moment at section, N·mm, Sections 6.3, 6.4, 6.5, 6.6, 8.3.6

M_{ua} = Moment at mid height of wall due to factored lateral and eccentric vertical loads, not including P_Δ effects, N·mm, Sec 6.6

M_v = Moment resistance contributed by shear head reinforcement, N·mm, Sec 6.4

M_1 = Smaller factored end moment on a compression member, to be taken as positive if member is bent in single curvature, and negative if bent in double curvature, N·mm, Sec 6.3

M_{1ns} = Factored end moment on a compression member at the end at which M_1acts, due to loads that cause no appreciable side sway, calculated using a first-order elastic frame analysis, N·mm, Sec 6.3

M_{1s} = Factored end moment on compression member at the end at which M_1acts, due to loads that cause appreciable side sway, calculated using a first-order elastic frame analysis, N·mm, Sec 6.3

M_2 = Larger factored end moment on compression member. If transverse loading occurs between supports, M_2 is taken as the largest moment occurring in member. Value of M_2 is always positive, N·mm, Sec 6.3

$M_{2,min}$ = Minimum value of M_2, N·mm, Sec 6.3

M_{2ns} = Factored end moment on compression member at the end at which M_2acts, due to loads that cause no appreciable side sway, calculated using a first-order elastic frame analysis, N·mm, Sec 6.3

M_{2s} = Factored end moment on compression member at the end at which M_2acts, due to loads that cause appreciable sidesway, calculated using a first-order elastic frame analysis, N·mm, Sec 6.3

n = Number of items, such as strength tests, bars, wires, monostrand anchorage devices, anchors, or shear head arms, Sec 6.4, 8.2, K.1 Width of flight, Figure 6.6.29.

N_b = Basic concrete breakout strength in tension of a single anchor in cracked concrete, N, Sec K.5.2.2

N_{cb} = Nominal concrete breakout strength in tension of a single anchor, N, see Sec K.5.2.1

N_{cbg} = Nominal concrete breakout strength in tension of a group of anchors, N, Sec K.5.2.1

N_n = Nominal strength in tension, N, Sec K.3.3

N_p = Pullout strength in tension of a single anchor in cracked concrete, N, Sections K.2.3 K.3.3, K.5.3

N_{pn} = Nominal pullout strength in tension of a single anchor, N, Sections K.4.1, K.5.3

N_{sa} = Nominal strength of a single anchor or group of anchors in tension as governed by the steel strength, N, Sections K.4.1, K.5.1

N_{sb} = Side-face blowout strength of a single anchor, N, Sec K.4.1

N_{sbg} = Side-face blowout strength of a group of anchors, N, Sections K.4.1, K.5.4

N_u = Factored axial force normal to cross section occurring simultaneously with $V_u\, or\, T_u$; to be taken as positive for compression and negative for tension, N, Sec 6.4

N_{ua} = Factored tensile force applied to anchor or group of anchors, N, Sections K.4.1, K.7

N_{uc} = Factored horizontal tensile force applied at top of bracket or corbel acting simultaneously with V_u, to be taken as positive for tension, N, Sec 6.4

p_{cp} = Outside perimeter of concrete cross section, mm, Sec 6.4.4.1

p_h = Perimeter of centerline of outermost closed transverse torsional reinforcement, mm, Sec 6.4

P_b = Nominal axial strength at balanced strain conditions, N, Sections 6.2, 6.3.3

P_c = Critical buckling load, N, Sec 6.3.10

P_n = Nominal axial strength of cross section, N, Sections 6.2, 6.3, 6.6

$P_{n,max}$ = Maximum allowable value of P_n, N, Sec 6.3.3

P_o = Nominal axial strength at zero eccentricity, N, Sec 6.3

P_s = Unfactored axial load at the design (mid height) section including effects of self-weight, N, Sec 6.6

P_u = Factored axial force; to be taken as positive for compression and negative for tension, N, Sections 6.3, 6.6

q_{Du} = Factored dead load per unit area, Sec 6.5

q_{Lu} = Factored live load per unit area, Sec 6.5

q_u = Factored load per unit area, Sec 6.5

Q = Stability index for a story, Sec 6.3.10.5.2

r = Radius of gyration of cross section of a compression member, mm, Sec 6.3

R = Rain load, or related internal moments and forces, Sec 6.2

s = Center-to-center spacing of items, such as longitudinal reinforcement, transverse reinforcement, wires, or anchors, mm, Sections 6.3, 6.4, 6.9, 6.11, 6.12, 8.2.3, 8.3.4, Appendix K

s_i = Center-to-center spacing of reinforcement in the i-th layer adjacent to the surface of the member, mm, Sec I.3.3

s_o = Center-to-center spacing of transverse reinforcement within the length l_o, mm, Sec 8.3.10

s_s = Sample standard deviation, MPa, Sec K.1

s_2 = Center-to-center spacing of longitudinal shear or torsion reinforcement, mm, Sec 6.4

t = Wall thickness of hollow section, mm, Sec 6.4

T = Cumulative effect of temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete, Sec 6.2

T_n = Nominal torsional moment strength, N·mm, Sec 6.4

T_u = Factored torsional moment at section, N·mm, Sec 6.4

U = Required strength to resist factored loads or related internal moments and forces, Sec 6.2

v_n = Nominal shear stress, MPa, Sections 6.4, 8.3.8

V_b = Basic concrete breakout strength in shear of a single anchor in cracked concrete, N, Sec K.6.2

V_c = Nominal shear strength provided by concrete, N, Sections 6.1, 6.4, 6.5, 8.3.8

V_{cb} = Nominal concrete breakout strength in shear of a single anchor, N, Sections K.4.1, K.6.2

V_{cbg} = Nominal concrete breakout strength in shear of a group of anchors, N, Sections K.4.1, K.6.2

V_{ci} = Nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, N, Sec 6.4

V_{cp} = Nominal concrete pryout strength of a single anchor, N, Sec K.6.3.1

V_{cpg} = Nominal concrete pryout strength of a group of anchors, N, Sec K.6.3.1

V_{cw} = Nominal shear strength provided by concrete when diagonal cracking results from high principal tensile stress in web, N, Sec 6.4

V_d = Shear force at section due to unfactored dead load, N, Sec 6.4

V_e = Design shear force corresponding to the development of the probable moment strength of the member, N, Sec 8.3.8

V_n = Nominal shear strength, N, Sections 6.1, 6.3, 6.4, 8.3.6, K.3.3

V_{nh} = Nominal horizontal shear strength, N, Sec 6.12

V_s = Nominal shear strength provided by shear reinforcement, N, Sec 6.4

V_{sa} = Nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, N, see Sections K.3.3, K.6.1.1, K.6.1.2

V_u = Factored shear force at section, N, Sections 6.4, 6.5, 6.12, 8.2.7, 8.3.6

V_{ua} = Factored shear force applied to a single anchor or group of anchors, N, K.4.1

V_{ug} = Factored shear force on critical section of two-way slab action due to gravity loads, N, Sec 8.3.12

V_{us} = Factored horizontal shear in a story, N, Sec 6.3

w = Uniform load, Sec 6.5.8

wl_c = Density (unit weight) of normal weight concrete or equilibrium density of light weight concrete, kg/m^3, Sections 6.1, 6.2

w_u = Factored load per unit length of beam or one way slab, Sec 6.1

W = Wind load, or related internal moments and forces, Sec 6.2

x = Shorter overall dimension of rectangular part of cross section, mm, Sec 6.5

y = Longer overall dimension of rectangular part of cross section, mm, Sec 6.5

y_t = Distance from centroidal axis of gross section, neglecting reinforcement, to tension face, mm, Sections 6.2, 6.4

α = Angle defining the orientation of reinforcement, Sections 6.4, I.3.3

α_c = Coefficient defining the relative contribution of concrete strength to nominal wall shear strength, Sec 8.3.6

α_f = Ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by centerlines of adjacent panels (if any) on each side of the beam, Sections 6.2, 6.4.2, 6.5.6, 6.5.8

α_{fm} = Average value of α_f for all beams on edges of a panel, Sec 6.2

α_{f1} = α_f in direction of l_1, Sec 6.5

α_{f2} = α_f in direction of l_2, Sec 6.5

α_i = Angle between the axis of a strut and the bars in the i-th layer of reinforcement crossing that strut, Sec I.3.3

α_s = Constant used to compute V_c in slabs and footings, Sec 6.4

α_v = Ratio of flexural stiffness of shear head arm to that of the surrounding composite slab section, Sec 6.4.10

β = Ratio of long to short dimensions: clear spans for two-way slabs, Sec 6.2.5 sides of column, concentrated load or reaction area, Sec 6.4.10; or sides of a footing, Sections 6.2, 6.4, 6.8.4

β_b = Ratio of area of reinforcement cut off to total area of tension reinforcement at section, Sec 8.2.7

β_{dns} = Ratio used to account for reduction of stiffness of columns due to sustained axial loads, Sec 6.3.10

β_{ds} = Ratio used to account for reduction of stiffness of columns due to sustained lateral loads, Sec 6.3.10.4

β_n = Factor to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone, Sec I.5.2

β_s = Factor to account for the effect of cracking and confining reinforcement on the effective compressive strength of the concrete in a strut, Sec I.3.2

β_t = Ratio of torsional stiffness of edge beam section to flexural stiffness of a width of slab equal to span length of beam, center-to-center of supports, Sec 6.5.6.4

β_1 = Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, Sec 6.3.2.7

γ_f = Factor used to determine the unbalanced moment transferred by flexure at slab-column connections, Sections 6.4, 6.5.5.3

γ_s = Factor used to determine portion of reinforcement located in center band of footing, Sec 6.8.4.4

γ_v = Factor used to determine the unbalanced moment transferred by eccentricity of shear at slab-column connections, Sec 6.4.10.7

δ = Moment magnification factor to reflect effects of member curvature between ends of compression member, Sec 6.3

δ_s = Moment magnification factor for frames not braced against side sway, to reflect lateral drift resulting from lateral and gravity loads, Sec 6.3

δ_u = Design displacement, mm, Sec 8.3.6

Δ_{cr} = Computed, out-of-plane deflection at mid height of wall corresponding to cracking moment, M_{cr}, mm, Sec 6.6

Δ_n = Computed, out-of-plane deflection at mid height of wall corresponding to nominal flexural strength, M_n, mm, Sec 6.6

Δ_o = Relative lateral deflection between the top and bottom of a story due to lateral forces computed using a first-order elastic frame analysis and stiffness values satisfying Sec 6.3

Δ_r = Difference between initial and final (after load removal) deflections for load test or repeat load test, mm, Sec 6.11

Δ_s = Computed, out-of-plane deflection at mid height of wall due to service loads, mm, Sec 6.6

Δ_u = Computed deflection at mid height of wall due to factored loads, mm, Sec 6.6

Δ_1 = Measured maximum deflection during first load test, mm, Sec 6.11.5.2

Δ_2 = Maximum deflection measured during second load test relative to the position of the structure at the beginning of second load test, mm, Sec 6.11.5.2

ε_t = Net tensile strain in extreme layer of longitudinal tension steel at nominal strength, creep, shrinkage, and temperature, Sections 6.1 to 6.3

θ = Angle between axis of strut, compression diagonal, or compression field and the tension chord of the member, Sec 6.4.4

λ = Modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normal weight concrete of the same compressive strength, Sections 6.1.8.1, 6.2, 6.4.5.4, 6.9, 8.2.3.4, 8.2.6.2, 8.2.10.2, 8.3.6, I.3.2, K.5.2

λ_Δ = Multiplier for additional deflection due to long-term effects, Sec 6.2.5.2.5

μ = Coefficient of friction, Sec 6.4.5.4.3

ξ = Time-dependent factor for sustained load, Sec 6.2.5.2

ρ = Ratio of A_s to bd, Sections 6.4, 6.5, 8.3.4

ρ' = Ratio of A'_s to bd, Sections 6.2, 6.3.15.1

ρ_b = Ratio of A_s to bd producing balanced strain conditions, Sections 6.3.3.2, 6.5, 6.6

ρ_f = Ratio of A_{sf} to $b_w d$, Sec 6.3.15.2

ρ_l = Ratio of area of distributed longitudinal reinforcement to gross concrete area perpendicular to that reinforcement, Sections 6.4, 6.6, 8.3.6

ρ_{max} = Maximum reinforcement ratio allowed for beams corresponding to $\varepsilon_t = 0.004$, Sec 6.3.15.1

ρ_s = Ratio of volume of spiral reinforcement to total volume of core confined by the spiral (measured out-to-out of spirals), Sections 6.3, 8.3.5

ρ_t = Ratio of area distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement, Sections 6.4, 6.6, 8.3.6

ρ_v = Ratio of tie reinforcement area to area of contact surface, Sec 6.12.5.3

ρ_w = Ratio of A_s to $b_w d$, Sections 6.3.15.2, 6.4

φ = Strength reduction factor, see Sec 6.2.3, Sections 6.1 to 6.6, 6.9, 6.11, 6.12, 8.3.12, I.2.6, K.2.1

$\psi_{c,N}$ = Factor used to modify tensile strength of anchors based on presence or absence of cracks in concrete, Sec K.5.2

$\psi_{c,P}$ = Factor used to modify pullout strength of anchors based on presence or absence of cracks in concrete, Sec K.5.3

$\psi_{c,V}$ = Factor used to modify shear strength of anchors based on presence or absence of cracks in concrete and presence or absence of supplementary reinforcement, Sec K.6.2 for anchors in shear

ψ_e = Factor used to modify development length based on reinforcement coating, Sec 8.2.3

$\psi_{ec,N}$ = Factor used to modify tensile strength of anchors based on eccentricity of applied loads, Sec K.5.2

$\psi_{ec,V}$ = Factor used to modify shear strength of anchors based on eccentricity of applied loads, Sec K.6.2

$\psi_{ed,N}$ = Factor used to modify tensile strength of anchors based on proximity to edges of concrete member, Sec K.5.2

$\psi_{ed,V}$ = Factor used to modify shear strength of anchors based on proximity to edges of concrete member, Sec K.6.2

$\psi_{h,V}$ = Factor used to modify shear strength of anchors located in concrete members with $h_a < 1.5c_{a1}$, Sec K.6.2

ψ_s = Factor used to modify development length based on reinforcement size, Sec 8.2.3

ψ_t = Factor used to modify development length based on reinforcement location, Sec 8.2.3

ψ_w = Factor used to modify development length for welded deformed wire reinforcement in tension, Sec 8.2.18

6.1.3 General

6.1.3.1 Members shall be designed for adequate strength in accordance with the provisions of this Chapter, using load factors specified in Sec 2.7.3.1 and strength reduction factors ϕ in Sec 6.2.3.1.

6.1.3.2 Design of reinforced concrete members using Working Stress Design method (Appendix J) is also permitted.

6.1.3.3 Structures and structural members shall be designed to have design strength at all sections at least equal to the required strength (U) calculated for the factored loads and forces in such combinations as are stipulated in Chapter 2, Loads. The nominal strength provided for the section multiplied by the strength reduction factor ϕ shall be equal to or greater than the calculated required
strength U.

6.1.3.4 Members shall also meet all the other requirements of this Code to ensure adequate performance at service loads.

6.1.3.5 Design strength of reinforcement represented by the values of f_y and f_{yt} used in design calculations shall not exceed 550 MPa, and for transverse reinforcement in Sections 6.3.9.3 and 8.3. f_y or f_{yt} may exceed 420 MPa, only if the ratio of the actual tensile strength to the actual yield strength is not less than 1.20, and the elongation percentage is not less than 16.

6.1.3.6 For structural concrete, f_c' shall not be less than 17 MPa. No maximum value of f_c' shall apply unless restricted by a specific Code provision.

6.1.4 Loading

6.1.4.1 Loads and their combinations shall be in accordance with the requirements specified in Chapter 2 of this Part.

6.1.4.2 Structures shall be designed to resist all applicable loads.

6.1.4.3 Effects of forces due to crane loads, vibration, impact, shrinkage, temperature changes, creep, expansion of shrinkage-compensating concrete, and unequal settlement of supports shall be duly considered.

6.1.4 Methods of Analysis

6.1.4.1 Members of frames or continuous construction (beams or one-way slabs) shall be designed for the maximum effects of factored loads as determined by the theory of elastic analysis, except as modified for redistribution of moments in continuous flexural members according to Sec 6.1.5. Design is permitted to be simplified by using the assumptions specified in Sections 6.1.6, 6.1.9 to 6.1.12.

6.1.4.2 Frame analysis by approximate methods shall be permitted for buildings of usual types of construction, spans, and story heights.

6.1.4.3 Provided (a) to (e) below are satisfied, the approximate moments and shears given here shall be permitted for design of continuous beams and one-way slabs (slabs reinforced to resist flexural stresses in only one direction), as an alternate to frame analysis:

(a) There are two or more spans;

(b) Spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20 percent;

(c) Loads are uniformly distributed;

(d) Unfactored live load, L, does not exceed three times unfactored dead load, D; and

(e) Members are prismatic.

For calculating negative moments, l_n is taken as the average of the adjacent clear span lengths.

Positive moment	
End spans	
Discontinuous end unrestrained	$w_u l_n^2/11$
Discontinuous end integral with support	$w_u l_n^2/14$
Interior spans	$w_u l_n^2/16$
Negative moments at exterior face of first interior support	
Two spans	$w_u l_n^2/9$
More than two spans	$w_u l_n^2/10$
Negative moment at other faces of interior supports	$w_u l_n^2/11$
Negative moment at face of all supports for Slabs with spans not exceeding 3.048 m; and beams where ratio of sum of column stiffness to beam stiffness exceeds 8 at each end of the span	$w_u l_n^2/12$
Negative moment at interior face of exterior support for members built integrally with supports	
Where support is spandrel beam	$w_u l_n^2/24$
Where support is a column	$w_u l_n^2/16$
Shear in end members at face of first interior support	$1.15 w_u l_n/2$
Shear at face of all other supports	$w_u l_n/2$

6.1.4.4 Strut-and-tie models, provided in Appendix I, shall be permitted to be used in the design of structural concrete.

6.1.5 Redistribution of Moments in Continuous Flexural Members

6.1.5.1 It shall be permitted to decrease factored moments calculated by elastic theory at sections of maximum negative or maximum positive moment in any span of continuous flexural members for any assumed loading arrangement by not more than $1000\varepsilon_t$ percent, with a maximum of 20 percent, except where approximate values for moments are used.

6.1.5.2 Redistribution of moments shall be made only when ε_t is equal to or greater than 0.0075 at the section at which moment is reduced.

6.1.5.3 At all other sections within the spans, the reduced moment shall be used for calculating redistributed moments. Static equilibrium shall have to be maintained after redistribution of moments for each loading arrangement.

6.1.6 Span Length

6.1.6.1 The span length of a simply supported beam shall be taken as the smaller of the distance between the centres of bearings, or the clear distance between supports plus the effective depth.

6.1.6.2 For determination of moments in analysis of frames or continuous construction, span length shall be taken as the distance center-to-center of supports.

6.1.6.3 Design on the basis of moments at faces of support shall be permitted for beams built integrally with supports.

6.1.6.4 It shall be permitted to analyze solid or ribbed slabs built integrally with supports, with clear spans not more than 3 m, as continuous slabs on knife edge supports with spans equal to the clear spans of the slab and width of beams otherwise neglected.

6.1.6.5 The effective length of a cantilever is its length to the face of the support plus half its effective depth, except where it forms the end of a continuous beam, where the length to the centre of support shall be used.

6.1.7 Modulus of Elasticity

6.1.7.1 Modulus of elasticity, E_c, for concrete shall be permitted to be taken as $w_c^{1.5}0.043\sqrt{f_c'}$ (in MPa) for values of w_c between 1440 and 2560 kg/m^3. For normal weight concrete, E_c shall be permitted to be taken as $4700\sqrt{f_c'}$ (in MPa).

6.1.7.2 Modulus of elasticity, E_s, for reinforcement shall be permitted to be taken as 200,000 MPa.

6.1.8 Lightweight Concrete

6.1.8.1 To account for the use of lightweight concrete, unless specifically noted otherwise, a modification factor λ appears as a multiplier of $\sqrt{f_c'}$ in all applicable equations and sections of this Code, where, $\lambda = 0.85$ for sand-lightweight concrete and 0.75 for all-lightweight concrete. Linear interpolation between 0.75 and 0.85 shall be permitted, on the basis of volumetric fractions, when a portion of the lightweight fine aggregate is replaced with normal weight fine aggregate. Linear interpolation between 0.85 and 1.0 shall be permitted, on the basis of volumetric fractions, for concrete containing normal weight fine aggregate and a blend of lightweight and normal weight coarse aggregates. For normal weight concrete, $\lambda = 1.0$. If average splitting tensile strength of lightweight concrete, f_{ct}, is specified, $\lambda = \dfrac{f_{ct}}{0.56\sqrt{f_c'}} \leq 1.0$

6.1.9 Stiffness

6.1.9.1 For computing relative flexural and torsional stiffnesses of columns, walls, floors, and roof systems, use of any set of reasonable assumptions shall be permitted. The assumptions adopted shall be consistent throughout analysis.

6.1.9.2 Both in determining moments and in design of members, effect of haunches shall be considered.

6.1.10 Effective Stiffness for Determining Lateral Deflections

6.1.10.1 Lateral deflections resulting from service lateral loads for reinforced concrete building systems shall be computed by either a linear analysis with member stiffness determined using 1.4 times the flexural stiffness defined in Sections 6.1.11.2 and 6.1.11.3 or by a more detailed analysis. Member properties shall not be taken greater than the gross section properties.

6.1.10.2 Lateral deflections resulting from factored lateral loads for reinforced concrete building systems shall be computed either by linear analysis with member stiffness defined by (a) or (b), or by a more detailed analysis considering the reduced stiffness of all members under the loading conditions:

(a) By section properties defined in Sec 6.3.10.4.1(a) to (c); or

(b) 50 percent of stiffness values based on gross section properties.

6.1.10.3 Lateral deflections resulting from factored lateral loads shall be permitted to be computed by using linear analysis, where two-way slabs without beams are designated as part of the seismic-force-resisting system. The stiffness of slab members shall be defined by a model that is in substantial agreement with results of comprehensive tests and analysis and the stiffness of other frame members shall be as defined in Sec 6.1.11.2.

6.1.11 Considerations for Columns

6.1.11.1 Columns shall be designed to resist the axial forces from factored loads on all floors or roof and the maximum moment from factored loads on a single adjacent span of the floor or roof under consideration. Loading condition resulting the maximum ratio of moment to axial load shall also be considered.

6.1.11.2 In frames or continuous construction, consideration shall be given to the effect of unbalanced floor or roof loads on both exterior and interior columns and of eccentric loading due to other causes.

6.1.11.3 It shall be permitted to assume far ends of columns built integrally with the structure to be fixed, while computing gravity load moments in columns.

6.1.11.4 Resistance to moments at any floor or roof level shall be provided by distributing the moment between columns immediately above and below the given floor in proportion to the relative column stiffnesses and conditions of restraint.

6.1.12 Live Load Arrangement

6.1.12.1 The following shall be permitted to assume:

(a) The live load is applied only to the floor or roof under consideration; and

(b) The far ends of columns built integrally with the structure are considered to be fixed.

6.1.12.2 Arrangement of live load shall be permitted to be assumed to be limited to combinations of:

(a) Factored dead load on all spans with full factored live load on two adjacent spans; and

(b) Factored dead load on all spans with full factored live load on alternate spans.

6.1.13 Construction of T-beam

6.1.13.1 In the construction of T-beam, the flange and web shall be built integrally or otherwise effectively bonded together.

6.1.13.2 Width of slab effective as a T-beam flange shall not exceed one-quarter of the span length of the beam, and the effective overhanging flange width on each side of the web shall not exceed:

(a) Eight times the slab thickness; and

(b) One-half the clear distance to the next web.

6.1.13.3 The effective overhanging flange width for beams with a slab on one side only shall not exceed:

(a) One-twelfth the span length of the beam;

(b) Six times the slab thickness; and

(c) One-half the clear distance to the next web.

6.1.13.4 Isolated beams, in which the T-shape is used to provide a flange for additional compression area, shall have a flange thickness not less than one-half the width of web and an effective flange width not more than four times the width of web.

6.1.13.5 When primary flexural reinforcement in a slab that is considered as a T-beam flange (excluding joist construction) is parallel to the beam, reinforcement shall be provided in the top of the slab in the direction perpendicular to the beam and in accordance with the following:

6.1.13.5.1 Transverse reinforcement shall be designed to carry the factored load on the overhanging slab width assumed to act as a cantilever. For isolated beams, the full width of overhanging flange shall be considered. For other T-beams, only the effective overhanging slab width need be considered.

6.1.13.5.2 Spacing of transverse reinforcement shall be not farther apart than five times the slab thickness, nor farther apart than 450 mm.

6.1.14 Construction of Joist

6.1.14.1 Construction of joist consists of a monolithic combination of regularly spaced ribs and a top slab arranged to span in one direction or two orthogonal directions.

6.1.14.2 Width of ribs shall not be less than 100 mm, and the ribs shall have a depth of not more than 3.5 times the minimum width of rib.

6.1.14.3 Clear spacing between ribs shall not exceed 750 mm.

6.1.14.4 Joist construction not meeting the limitations of Sections 6.1.15.1 to 6.1.15.3 shall be designed as slabs and beams.

6.1.14.5 When permanent burned clay or concrete tile fillers of material having a unit compressive strength at least equal to f_c' in the joists are used:

6.1.14.5.1 For shear and negative moment strength computations, the vertical shells of fillers in contact with the ribs shall be permitted to include. Other portions of fillers shall not be included in strength computations.

6.1.14.5.2 Slab thickness over permanent fillers shall be not less than $1/12^{th}$ the clear distance between ribs, nor less than 40 mm.

6.1.14.5.3 Reinforcement normal to the ribs shall be provided in the in one-way joists, as required by Sec 8.1.11

6.1.14.6 When removable forms or fillers are used, which do not comply with Sec 6.1.15.5, then:

6.1.14.6.1 Slab thickness shall be not less than 1/12th the clear distance between ribs, nor less than 50 mm.

6.1.14.6.2 Reinforcement normal to the ribs shall be provided in the slab as required for flexure, considering load concentrations, if any, but not less than required by Sec 8.1.11

6.1.14.7 Where conduits or pipes as permitted by relevant provisions of embedments in concrete are embedded within the slab, slab thickness shall be at least 25 mm greater than the total overall depth of the conduits or pipes at any point. Conduits or pipes shall not impair significantly the strength of the construction.

6.1.14.8 For joist construction, V_c shall be permitted to be 10 percent more than that specified in Sec 6.4.

6.1.15 Separate Floor Finish

6.1.15.1 Unless placed monolithically with the floor slab or designed in accordance with requirements of Sec. 6.12, floor finish shall not be included as part of a structural member.

6.1.15.2 All concrete floor finishes shall be permitted to be considered as part of required cover or total thickness for nonstructural considerations.

6.2 Strength and Serviceability Requirements

6.2.1 General

6.2.1.1 Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated in this Code.

6.2.1.2 Members also shall meet all other requirements of this Code to ensure adequate performance at service load levels.

6.2.2 Required Strength

6.2.2.1 Required strength U shall be at least equal to the effects of factored loads in such combinations as are stipulated in Chapter 2, Loads.

6.2.2.2 If resistance to impact effects is taken into account in design, such effects shall be included with L.

6.2.2.3 Estimations of differential settlement, creep, shrinkage, expansion of shrinkage-compensating concrete, or temperature change shall be based on a realistic assessment of such effects occurring in service.

6.2.2.4 For structures like emergency preparedness centre, cyclone shelters etc. in coastal zone, in load combination 4 of Sec 2.7.3.1 of Chapter 2, the coefficient of live load L shall be taken 1.6 instead of 1.0.

6.2.3 Design Strength

6.2.3.1 Design strength provided by a member, and its connections to other members, in terms of flexure, axial load, shear, and torsion, shall be taken as the nominal strength calculated in accordance with the requirements and assumptions of this Chapter, multiplied by a strength reduction factors ϕ as stipulated in Sections 6.2.3.2 to 6.2.3.4.

6.2.3.2 Strength reduction factor ϕ is given in Sections 6.2.3.2.1 to 6.2.3.2.6:

6.2.3.2.1 For tension-controlled sections as defined in Sec 6.3.3.4: 0.90

6.2.3.2.2 For compression-controlled sections, as defined in Sec 6.3.3.3:

Members with spiral reinforcement conforming to Sec 6.3.9.3: 0.75

Other reinforced members: 0.65

For sections in which the net tensile strain in the extreme tension steel at nominal strength, ε_t, is between the limits for compression-controlled and tension-controlled sections, ϕ shall be permitted to be linearly increased from that for compression-controlled sections to 0.90 as ε_t increases from the compression controlled strain limit to 0.005 (Also see Figure 6.6.1). While interpolating, it shall be permitted to round ϕ to second digit after decimal.

6.2.3.2.3 It shall be permitted for compression-controlled sections, as defined in Sec 6.3.3.3, the following optional, more conservative alternative values of strength reduction factor ϕ, where less controlled construction environment justifies such selection according to engineering judgment of the designer:

For members with spiral reinforcement conforming to Sec 6.3.9.3:0.70

For other reinforced members: 0.60

For sections in which the net tensile strain in the extreme tension steel at nominal strength, ε_t, is between the limits for compression-controlled and tension-controlled sections, ϕ shall be permitted to be linearly increased from that for compression-controlled sections to 0.90 as ε_t increases from the compression controlled strain limit to 0.005 (Also see Figure 6.6.2). While interpolating, it shall be permitted to round ϕ to second digit after decimal.

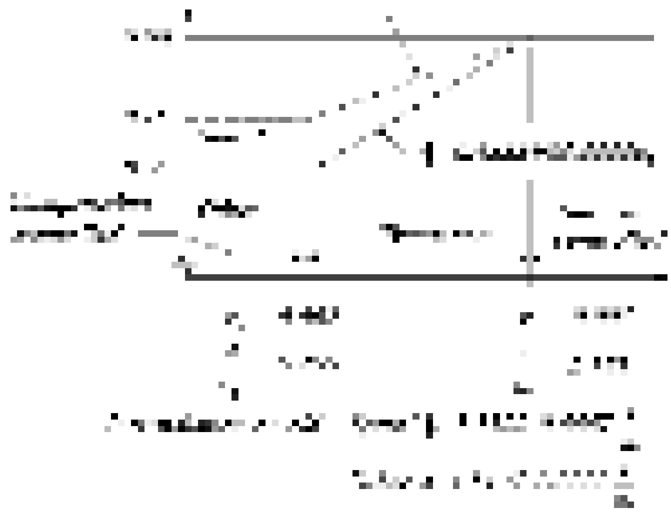


Figure 6.6.1 Variation of ϕ with net tensile strain in extreme tension steel, ε_t and c/d_t for Grade 420 reinforcement and for prestressing steel

6.2.3.2.4 Strength reduction factor for shear and torsion: 0.75

6.2.3.2.5 Strength reduction factor for bearing on concrete (except for post-tensioned anchorage zones and strut-and-tie models): 0.65

6.2.3.2.6 Strength reduction factor for strut-and-tie models (Appendix I), and struts, ties, nodal zones, and bearing areas in such models: 0.75

6.2.3.2.7 Calculation of development length specified in Sec 8.2 does not require strength reduction factor ϕ.

6.2.3.3 For structures relying on intermediate precast structural walls in Seismic Design Category D, special moment frames, or special structural walls to resist earthquake effects, E, ϕ shall be modified as given in (a) through (c):

(a) For any structural member that is designed to resist E, if the nominal shear strength of the member is less than the shear corresponding to the development of the nominal flexural strength of the member, ϕ for shear shall be 0.60. The nominal flexural strength shall be determined considering the most critical factored axial loads and including E;

(b) For diaphragms, ϕ for shear shall not exceed the minimum ϕ for shear used for the vertical components of the primary seismic-force-resisting system;

(c) For joints and diagonally reinforced coupling beams, ϕ for shear shall be 0.85.

6.2.3.4 Strength reduction factor ϕ shall be 0.60 for flexure, compression, shear, and bearing of structural plain concrete.

6.2.4 Design Strength for Reinforcement

The values of f_y and f_{yt} used in design calculations shall not exceed 550 MPa, except for transverse reinforcement in Sections 6.3.9.3 and 8.3.

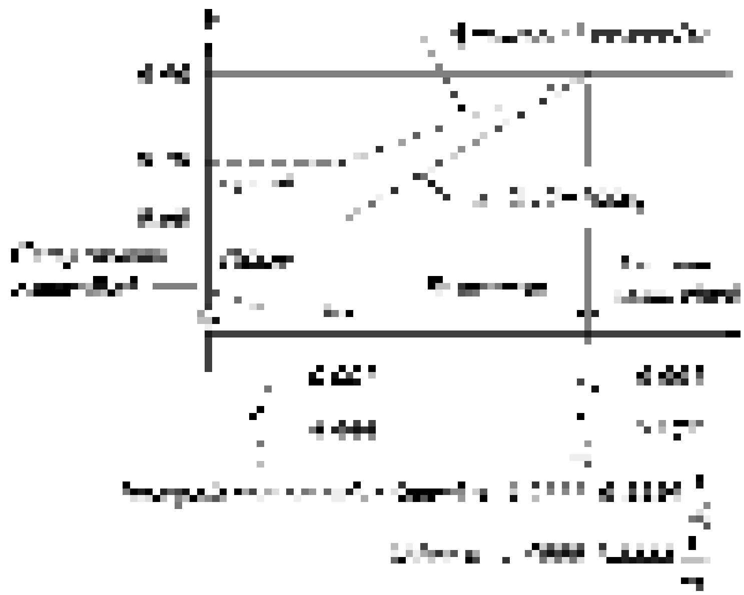


Figure 6.6.2 Variation of ϕ with net tensile strain in extreme tension steel, ε_t and c/d_t for Grade 420 reinforcement and for prestressing steel with reduced values of ϕ (0.6 and 0.7) for compression controlled sections (see Sec.6.2.3.2.3, Optional application in case of less controlled environment as per engineering judgment)

6.2.5 Control of Deflections

6.2.5.1 Reinforced concrete members subjected to flexure shall be designed to have adequate stiffness to limit deflections or any deformations that may adversely affect strength or serviceability of a structure.

6.2.5.2 One-way construction (non prestressed)

6.2.5.2.1 Minimum thickness stipulated in Table 6.6.1 shall apply for one-way construction not supporting or attached to partitions or other construction likely to be damaged by large deflections, unless computation of deflection indicates a lesser thickness can be used without adverse effects.

6.2.5.2.2 Where deflections are to be computed, deflections that occur immediately on application of load shall be computed by usual methods or formulas for elastic deflections, considering effects of cracking and reinforcement on member stiffness.

6.2.5.2.3 If not stiffness values are obtained by a more comprehensive analysis, immediate deflection shall be computed with the modulus of elasticity for concrete, E_C, as specified in 6.1.7.1 (normal weight or lightweight concrete) and with the effective moment of inertia, Ie, as follows, but not greater than I_g

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad (6.6.1)$$

Where,

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad (6.6.2)$$

And,

$$f_r = 0.62\lambda\sqrt{f_c'} \qquad (6.6.3)$$

Table 6.6.1: Minimum Thickness of Non prestressed beams or one-Way slabs Unless Deflections are calculated

Member	Minimum thickness, h			
	Simply supported	One end continuous	Both ends continuous	Cantilever
	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections			
Solid one- way slabs	1/20	1/24	1/28	1/10
Beams or ribbed one-way slabs	1/16	1/18.5	1/21	1/8

Notes:

Values given shall be used directly for members with normal weight concrete and Grade 420 reinforcement. For other conditions, the values shall be modified as follows:

(a) For lightweight concrete having equilibrium density, w_c, in the range of 1440 to 1840 kg/m^3,the values shall be multiplied by

$(1.65 - 0.0003w_c)$ but not less than 1.09.

(b) For f_y other than 420MPa, the values shall be multiplied by $(0.4 + f_y/700)$.

6.2.5.2.4 I_e shall be permitted to be taken for continuous members as the average of values obtained from Eq. 6.6.1 for the critical positive and negative moment sections. For prismatic members, I_e shall be permitted to be taken as the value obtained from Eq. 6.6.1 at mid span for simple and continuous spans, and at support for cantilevers.

6.2.5.2.5 If the values are not obtained by a more comprehensive analysis, additional long-term deflection resulting from creep and shrinkage of flexural members (normal weight or lightweight concrete) shall be determined by multiplying the immediate deflection caused by the sustained load considered, by the factor λ_Δ

$$\lambda_\triangle = \frac{\xi}{1+50\rho'} \qquad (6.6.4)$$

Where, ρ' shall be the value at midspan for simple and continuous spans, and at support for cantilevers. It shall be permitted to assume ξ, the time-dependent factor for sustained loads, to be equal to:

5 years or more	2.0
12 months	1.4
6 months	1.2
3 months	1.0

6.2.5.2.6 The value of deflection computed in accordance with Sections 6.2.5.2.2 to 6.2.5.2.5 shall not exceed limits stipulated in Table 6.6.2.

6.2.5.3 Two-way construction (non prestressed)

6.2.5.3.1 The minimum thickness of slabs or other two-way construction designed in accordance with the provisions of Sec. 6.5 and conforming to the requirements of Sec 6.5.6.1.2 shall be governed by Sec 6.2.5.3. The thickness of slabs without interior beams spanning between the supports on all sides shall satisfy the requirements of Sec 6.2.5.3.2 or Sec 6.2.5.3.4. The thickness of slabs with beams spanning between the supports on all sides shall satisfy requirements of Sec 6.2.5.3.3 or Sec 6.2.5.3.4.

6.2.5.3.2 If slabs are without interior beams spanning between the supports and have a ratio of long to short span not greater than 2, the minimum thickness shall be in accordance with the provisions of Table 6.6.3 and shall not be less than the following values:

Slabs without drop panels as defined in Sec 6.5.2.5:	125 mm
Slabs with drop panels as defined in Sec 6.5.2.5:	100 mm

Table 6.6.2: Maximum Allowable Computed Deflections

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	l/180*
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections		l/360
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)†	l/480‡
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		l/240§

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

† Long-term deflection shall be determined in accordance with Sec 6.2.5.2.5, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

‡ Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.

§ Limit shall not be greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

Table 6.6.3: Minimum Thickness of Slabs without Interior Beams*

f_y, MPa†	Without drop panels‡			With drop panels‡		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams§		Without edge beams	With edge beams§	
280	$l_n/33$	$l_n/36$	$l_n/36$	$l_n/36$	$l_n/40$	$l_n/40$
420	$l_n/30$	$l_n/33$	$l_n/33$	$l_n/33$	$l_n/36$	$l_n/36$
520	$l_n/28$	$l_n/31$	$l_n/31$	$l_n/31$	$l_n/34$	$l_n/34$

* For two-way construction, l_n is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.

† For f_y between the values given in the table, minimum thickness shall be determined by linear interpolation.

‡ Drop panels as defined in Sec 6.5.2.5.

§ Slabs with beams between columns along exterior edges. The value of α_f for the edge beam shall not be less than 0.8.

6.2.5.3.3 The minimum thickness, h for slabs with beams spanning between the supports on all sides, shall be as follows:

(a) For α_{fm} equal to or less than 0.2, the provisions of Sec 6.2.5.3.2 shall apply;

(b) For α_{fm} greater than 0.2 but not greater than 2.0, h shall not be less than

$$h = \frac{l_n\left(0.8+\frac{f_y}{1400}\right)}{36+5\beta(\alpha_{fm}-0.2)} \geq 125 \text{ mm} \qquad (6.6.5)$$

(c) For α_{fm} greater than 2.0, h shall not be less than

$$h = \frac{l_n\left(0.8+\frac{f_y}{1400}\right)}{36+9\beta} \geq 90 \text{ mm} \qquad (6.6.6)$$

(d) An edge beam with a stiffness ratio α_f not less than 0.80 shall be provided at discontinuous edges, or the minimum thickness required by Eq. 6.6.5 or Eq. 6.6.6 shall be increased by at least 10 percent in the panel with a discontinuous edge.

Term l_n in (b) and (c) is length of clear span in long direction measured face-to-face of beams. Term β in (b) and (c) is ratio of clear spans in long to short direction of slab.

6.2.5.3.4 When computed deflections do not exceed the limits of Table 6.6.2, slab thickness less than the minimum required by Sections 6.2.5.3.1 to 6.2.5.3.3 shall be permitted. Deflections shall be computed taking into account size and shape of the panel, conditions of support, and nature of restraints at the panel edges. The modulus of elasticity of concrete, E_c, shall be as specified in Sec 6.1.7.1. The effective moment of inertia, I_e, shall be that given by Eq. 6.6.1; other values shall be permitted to be used if they result in computed deflections in reasonable agreement with results of comprehensive tests. Additional long-term deflection shall be computed in accordance with Sec 6.2.5.2.5.

6.2.5.4 Composite construction

6.2.5.4.1 Shored construction

Where composite flexural members are supported during construction so that, after removal of temporary supports, dead load is resisted by the full composite section, it shall be permitted to consider the composite member equivalent to a monolithically cast member for computation of deflection. For non prestressed members, the portion of the member in compression shall determine whether values in Table 6.6.1 for normal weight or lightweight concrete shall apply. If deflection is computed, account shall be taken of curvatures resulting from differential shrinkage of precast and cast-in-place components, and of axial creep effects in a prestressed concrete member.

6.2.5.4.2 Unshored construction

When the thickness of a non prestressed precast flexural member meets the requirements of Table 6.6.1, deflection need not be computed. If the thickness of a non prestressed composite member meets the requirements of Table 6.6.1, it is not required to compute deflection occurring after the member becomes composite, but the long-term deflection of the precast member shall be investigated for magnitude and duration of load prior to beginning of effective composite action.

6.2.5.4.3 The computed deflection in accordance with Sec 6.2.5.4.1 or Sec 6.2.5.4.2 shall not exceed limits stipulated in Table 6.6.2.

6.3 Axial Loads and Flexure

6.3.1 Scope

The provisions of Sec. 6.3 shall be applicable to the design of members subject to flexure or axial loads or a combination thereof.

6.3.2 Design Assumptions

6.3.2.1 The assumptions given in Sections 6.3.2.2 to 6.3.2.7, and satisfaction of applicable conditions of equilibrium and compatibility of strains shall form the basis of strength design of members for flexure and axial loads.

6.3.2.2 The strains in reinforcement and concrete hall be assumed to be directly proportional to the distance from the neutral axis, except that, for deep beams as defined in Sec 6.3.7.1, an analysis that considers a nonlinear distribution of strain shall be used. Alternatively, it shall be permitted to use a strut-and-tie model. See Sections 6.3.7, 6.4.6, and Appendix I.

6.3.2.3 The maximum usable strain at extreme concrete compression fiber shall be assumed to be 0.003.

6.3.2.4 For stress in reinforcement below f_y, it shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y, stress in reinforcement shall be considered independent of strain and equal to f_y.

6.3.2.5 In axial and flexural calculations of reinforced concrete, the tensile strength of concrete shall be neglected.

6.3.2.6 The relationship between concrete compressive stress distribution and concrete strain shall be assumed to be rectangular, trapezoidal, parabolic, or any other shape that results in prediction of strength in substantial agreement with results of comprehensive tests.

6.3.2.7 An equivalent rectangular concrete stress distribution defined by Sections 6.3.2.7.1 to 6.3.2.7.3 below shall satisfy the requirements of Sec 6.3.2.6.

6.3.2.7.1 Concrete stress of $0.85\ f_c'$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = \beta_1 c$ from the fibre of maximum compressive strain.

6.3.2.7.2 Distance from the fibre of maximum strain to the neutral axis, $\boldsymbol{c}$, shall be measured in a direction perpendicular to the neutral axis.

6.3.2.7.3 For f_c' between 17 and 28 MPa, β_1 shall be taken as 0.85. For f_c' above 28 MPa, β_1 shall be reduced linearly at a rate of 0.05 for each 7 MPa of strength in excess of 28 MPa, but β_1 shall not be taken less than 0.65. For f_c' between 28 and 56 MPa, β_1 may be calculated from Eq. 6.6.7.

$$\beta_1 = 0.85 - 0.007143(f_c' - 28)\ and\ \ 0.65 \le \beta_1 \le 0.85 \qquad (6.6.7)$$

6.3.3 General Principles and Requirements

6.3.3.1 Stress and strain compatibility using assumptions in Sec 6.3.2 shall be the basis for design of cross sections subject to flexure or axial loads, or a combination thereof.

6.3.3.2 A cross section shall be considered to be in balanced strain conditions when the tension reinforcement reaches the strain corresponding to f_y just as concrete in compression reaches its assumed ultimate strain of 0.003.

6.3.3.3 Sections are compression-controlled if the net tensile strain in the extreme tension steel, ε_t, is equal to or less than the compression-controlled strain limit when the concrete in compression reaches its assumed strain limit of 0.003, Figure 6.6.3. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 420 reinforcement, it shall be permitted to set the compression-controlled strain limit equal to 0.002. For other grades compression-controlled strain limit may be determined by dividing the yield strength by modulus of elasticity E and then rounding the value obtained to four significant digits after the decimal. For example, for Grade 500 reinforcement, the compression-controlled strain limit shall equal to 0.0025.

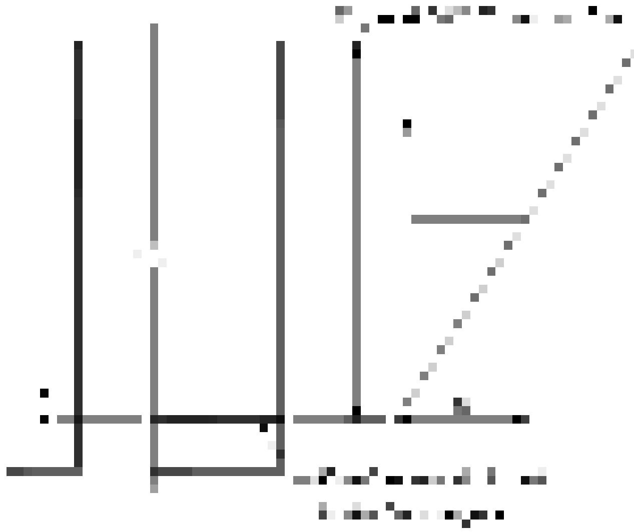


Figure 6.6.3 Strain distribution and net tensile strain

6.3.3.4 Sections are tension-controlled if the net tensile strain in the extreme tension steel, ε_t, is equal to or greater than 0.005 when the concrete in compression reaches its assumed strain limit of 0.003. Sections with ε_t between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections.

6.3.3.5 Net tensile strain in the extreme tension steel at nominal strength, ε_t shall not be less than 0.004 for non prestressed flexural members and non prestressed members with factored axial compressive load less than $0.10 f_c' A_g$

6.3.3.5.1 Use of compression reinforcement shall be permitted in conjunction with additional tension reinforcement to increase the strength of flexural members.

6.3.3.6 For compression members, design axial strength ϕP_n shall not be taken greater than $\phi P_{n,max}$, computed by Eq. 6.6.8 or Eq. 6.6.9.

6.3.3.6.1 For non prestressed members with spiral reinforcement conforming to Sec. 8.1 or composite members conforming to 6.3.13:

$$\phi P_{n,max} = 0.85\phi\left[0.85 f_c'(A_g - A_{st}) + f_y A_{st}\right] \qquad (6.6.8)$$

6.3.3.6.2 For non prestressed members with tie reinforcement conforming to Sec. 8.1:

$$\phi P_{n,max} = 0.80\phi\left[0.85 f_c'(A_g - A_{st}) + f_y A_{st}\right] \qquad (6.6.9)$$

6.3.3.7 Members subject to compressive axial load shall be designed for the maximum moment that can accompany the axial load. The factored axial force P_u at given eccentricity shall not exceed the value that given in Sec 6.3.3.6. The maximum factored moment M_u shall be magnified for slenderness effects in accordance with Sec 6.3.10.

6.3.4 Spacing of Lateral Supports for Flexural Members

6.3.4.1 Distance between lateral supports for a beam shall not exceed 50 times b, the least width of compression flange or face.

6.3.4.2 Effects of lateral eccentricity of load shall be taken into account in determining spacing of lateral supports.

6.3.5 Minimum Reinforcement for Members in Flexure

6.3.5.1 At every section of a flexural member where tensile reinforcement is required by analysis, except as provided in Sections 6.3.5.2 to 6.3.5.4, A_s provided shall not be less than that given by Equations 6.6.10a and 6.6.10b.

$$A_{s,min} = \frac{0.25\sqrt{f_c'}}{f_y} b_w d \tag{6.6.10a}$$

$$A_{s,min} = \frac{1.4 b_w d}{f_y} \tag{6.6.10b}$$

6.3.5.2 For statically determinate members with a flange in tension, $A_{s,min}$ shall not be less than the value given by Equations 6.6.10, except that b_w is replaced by either $2b_w$ or the width of flange, whichever is smaller.

6.3.5.3 If, at every section, A_s provided is at least one-third greater than that required by analysis, the requirements of Sections 6.3.5.1 and 6.3.5.2 need not be applied.

6.3.5.4 For structural slabs and footings including raft that help support the structure vertically of uniform thickness, $A_{s,min}$ in the direction of the span shall be the same as that required by Sec 8.1.11. Maximum spacing of this reinforcement shall not exceed three times the thickness, nor 450 mm.

6.3.6 Distribution of Flexural Reinforcement in One-Way Slabs and Beams

6.3.6.1 Rules for distribution of flexural reinforcement to control flexural cracking in beams and in one-way slabs (slabs reinforced to resist flexural stresses in only one direction) are prescribed in this section.

6.3.6.2 Distribution of flexural reinforcement in two-way slabs shall be as required by Sec 6.5.3.

6.3.6.3 As stated in Sec 6.3.6.4, flexural tension reinforcement shall be well distributed within maximum flexural tension zones of a member cross section.

6.3.6.4 The spacing of reinforcement closest to the tension face, s, shall be less than that given by

$$s = 380\left(\frac{280}{f_s}\right) - 2.5c_c \qquad (6.6.11)$$

But, shall not exceed, $300\left(\frac{280}{f_s}\right)$ where, c_c is the least distance from surface of reinforcement to the tension face. If there is only one bar or wire nearest to the extreme tension face, s used in Eq. 6.6.11 is the width of the extreme tension face.

Calculated stress f_s in reinforcement closest to the tension face at service load shall be computed based on the unfactored moment. It shall be permitted to take f_s as $\frac{2}{3}f_y$.

6.3.6.5 For structures subject to very aggressive exposure or designed to be watertight, provisions of Sec 6.3.6.4 are not sufficient. For such structures, special investigations and precautions are required.

6.3.6.6 When flanges of T-beam construction are in tension, part of the flexural tension reinforcement shall be distributed over an effective flange width as defined in Sec 6.1.13, or a width equal to one-tenth the span, whichever is smaller. If the effective flange width exceeds one-tenth the span, some longitudinal reinforcement shall be provided in the outer portions of the flange.

6.3.6.7 Longitudinal skin reinforcement shall be uniformly distributed along both side faces of a member (Figure 6.6.4), where h of a beam or joist exceeds 900 mm. Skin reinforcement shall extend for a distance $\frac{h}{2}$ from the tension face. The spacing s shall be as provided in Sec 6.3.6.4, where c_c is the least distance from the surface of the skin reinforcement to the side face. It shall be permitted to include such reinforcement in strength computations if a strain compatibility analysis is made to determine stress in the individual bars or wires.

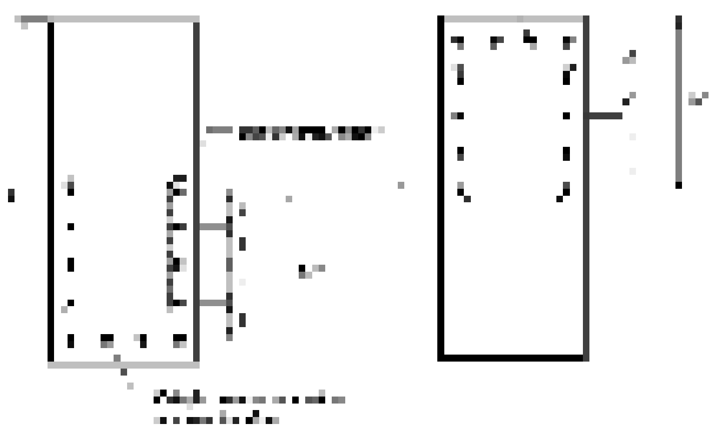


Figure 6.6.4 Skin reinforcement for beams and joists with h > 900 mm.

6.3.7 Deep Beams

6.3.7.1 Deep beams are members loaded on one face and supported on the opposite face so that compression struts can develop between the loads and the supports, and have either:

(a) Clear spans, l_n, equal to or less than four times the overall member depth; or

(b) Regions with concentrated loads within twice the member depth from the face of the support.

Deep beams shall be designed either taking into account nonlinear distribution of strain, or by Appendix I. (See also Sections 6.4.6.1 and 8.2.7.6) Lateral buckling shall be considered.

6.3.7.2 V_n of deep beams shall be in accordance with Sec 6.4.6.

6.3.7.3 Minimum area of flexural tension reinforcement, $A_{s,min}$, shall conform to Sec 6.3.5.

6.3.7.4 Minimum horizontal and vertical reinforcement in the side faces of deep beams shall satisfy either Sec I.3.3 or Sec 6.4.6.4 and Sec 6.4.6.5.

6.3.8 Design Dimensions for Compression Members

6.3.8.1 Isolated compression member with multiple spirals

Outer limits of the effective cross section of a compression member with two or more interlocking spirals shall be taken at a distance outside the extreme limits of the spirals equal to the minimum concrete cover required by Sec 8.1.7.

6.3.8.2 Monolithically built compression member with wall

Outer limits of the effective cross section of a spirally reinforced or tied reinforced compression member built monolithically with a concrete wall or pier shall be taken not greater than 40 mm outside the spiral or tie reinforcement.

6.3.8.3 Equivalent circular compression member replacing other shapes

In lieu of using the full gross area for design of a compression member with a square, octagonal, or other shaped cross section, it shall be permitted to use a circular section with a diameter equal to the least lateral dimension of the actual shape. Gross area considered, required percentage of reinforcement, and design strength shall be based on that circular section.

6.3.8.4 Limits of section

For a compression member with a cross section larger than required by considerations of loading, it shall be permitted to base the minimum reinforcement and strength on a reduced effective area A_g not less than one-half the total area. This provision shall not apply to special moment frames or special structural walls designed in accordance with Sec. 8.3.

6.3.9 Limits of Reinforcement for Compression Members

6.3.9.1 For noncomposite compression members, the area of longitudinal reinforcement, A_{st}, shall be not less than $0.01A_g$ or more than $0.06A_g$. To avoid practical difficulties in placing and compacting of concrete as well as to deliver ductility to noncomposite compression members, area of longitudinal reinforcement, A_{st}, is preferred not to exceed $0.04A_g$ unless absolutely essential.

6.3.9.2 Minimum number of longitudinal bars in compression members shall be 4 for bars within rectangular or circular ties, 3 for bars within triangular ties, and 6 for bars enclosed by spirals conforming to Sec 6.3.9.3.

6.3.9.3 Volumetric spiral reinforcement ratio, ρ_s, shall be not less than the value given by

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}} \qquad (6.6.12)$$

Where the value of f_{yt} used in Eq. 6.6.12 shall not exceed 700 MPa. For f_{yt} greater than 420 MPa, lap splices according to 8.1.9.3(e) shall not be used.

6.3.10 Slenderness Effects in Compression Members

6.3.10.1 Slenderness effects shall be permitted to be neglected in the following cases:

(a) for compression members not braced against side sway when:

$$\frac{kl_u}{r} \leq 22 \qquad (6.6.13)$$

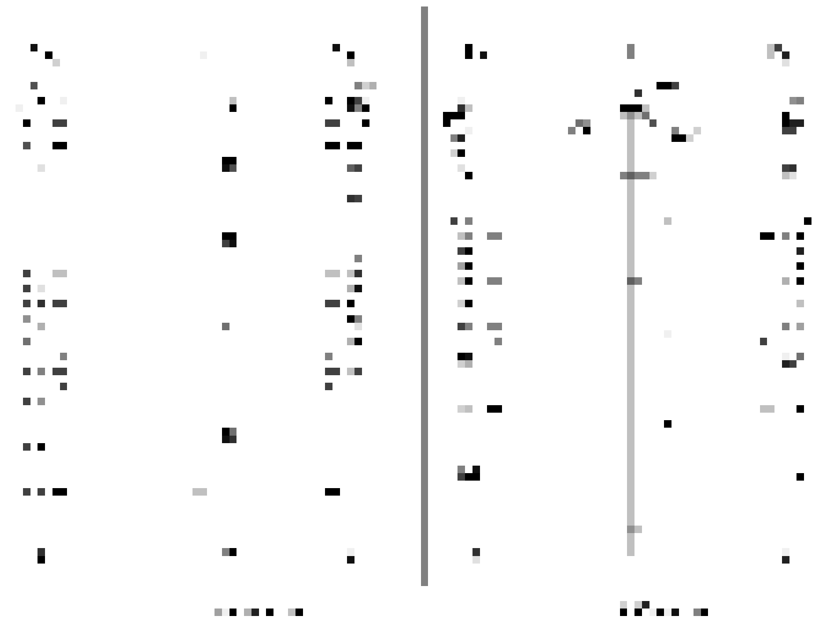
(b) for compression members braced against side sway when:

$$\frac{kl_u}{r} \leq 34 - 12(M_1/M_2) \leq 40 \qquad (6.6.14)$$

Where, M_1/M_2 is positive if the column is bent in single curvature, and negative if the member is bent in double curvature.

Compression members may be considered to be braced against side sway when bracing elements have a total stiffness, resisting lateral movement of that story, of at least 12 times the gross stiffness of the columns within the story.

The Jackson and Moreland Alignment Charts (Figure 6.6.5), which allow a graphical determination of k for a column of constant cross section in a multibay frame may be used as the primary design aid to estimate the effective length factor k.



$\Psi =$ ratio of $\Sigma\left(\frac{EI}{l_c}\right)$ of compression members to $\Sigma\left(\frac{EI}{l}\right)$ of flexural members in plane at one end of a compression member

$l =$ span length of flexural member measured center to center of joints

Figure 6.6.5 Effective length factors k.

6.3.10.1.1 The unsupported length of a compression member, l_u, shall be taken as the clear distance between floor slabs, beams, or other members capable of providing lateral support in the direction being considered. Where column capitals or haunches are present, l_u shall be measured to the lower extremity of the capital or haunch in the plane considered.

6.3.10.1.2 It shall be permitted to take the radius of gyration, r equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members and 0.25 times the diameter for circular compression members. For other shapes, it shall be permitted to compute r for gross concrete section.

6.3.10.2 When slenderness effects are not neglected as permitted by Sec 6.3.10.1, the design of compression members, restraining beams, and other supporting members shall be based on the factored forces and moments from a second-order analysis satisfying Sec 6.3.10.3, Sec 6.3.10.4, or Sec 6.3.10.5. These members shall also satisfy Sections 6.3.10.2.1 and 6.3.10.2.2. The dimensions of each member cross section used in the analysis shall be within 10 percent of the dimensions of the members shown on the design drawings or the analysis shall be repeated.

6.3.10.2.1 Total moment including second-order effects in compression members, restraining beams, or other structural members shall not exceed 1.4 times the moment due to first-order effects.

6.3.10.2.2 Second-order effects shall be considered along the length of compression members. It shall be permitted to account for these effects using the moment magnification procedure outlined in Sec 6.3.10.6.

6.3.10.3 Nonlinear second-order analysis

Second-order analysis shall consider material nonlinearity, member curvature and lateral drift, duration of loads, shrinkage and creep, and interaction with the supporting foundation. The analysis procedure shall have been shown to result in prediction of strength in substantial agreement with results of comprehensive tests of columns in statically indeterminate reinforced concrete structures.

6.3.10.4 Elastic second-order analysis

Elastic second-order analysis shall consider section properties determined taking into account the influence of axial loads, the presence of cracked regions along the length of the member, and the effects of load duration.

6.3.10.4.1 It shall be permitted to use the following properties for the members in the structure:

(a) Modulus of elasticity, E_c from Sec 6.1.7.1; (b) Moments of inertia, I as follows; and (c) Area $1.0A_g$

Compression members:	**Value of *I***
Columns	$0.70I_g$
Walls:	
Uncracked	$0.70I_g$
Cracked	$0.35I_g$

Flexural members:	**Value of *I***
Beams	$0.35I_g$
Flat plates and flat slabs	$0.25I_g$

Alternatively, the moments of inertia of compression and flexural members, $I,$ shall be permitted to be computed as follows:

(i) Compression members:

$$I = \left(0.80 + 25\frac{A_{st}}{A_g}\right)\left(1 - \frac{M_u}{P_u h} - 0.5\frac{P_u}{P_o}\right) I_g \leq 0.875 I_g \qquad (6.6.15)$$

Where, P_u and M_u shall be determined from the particular load combination under consideration, or the combination of P_u and M_u determined in the smallest value of I. The value of I need not be taken less than $0.35 I_g$.

(ii) Flexural members:

$$I = (0.10 + 25\rho)\left(1.2 - 0.2\frac{b_w}{d}\right) I_g \leq 0.5 I_g \qquad (6.6.16)$$

For continuous flexural members, I shall be permitted to be taken as the average of values obtained from Eq. 6.6.16 for the critical positive and negative moment sections. The value of I need not be taken less than $0.25 I_g$. The cross-sectional dimensions and reinforcement ratio used in the above formulas shall be within 10 percent of the dimensions and reinforcement ratio shown on the design drawings or the stiffness evaluation shall be repeated.

6.3.10.4.2 When sustained lateral loads are present, I for compression members shall be divided by $(1+\beta_{ds})$. The term β_{ds} shall be taken as the ratio of maximum factored sustained shear within a story to the maximum factored shear in that story associated with the same load combination, but shall not be taken greater than 1.0.

6.3.10.5 Procedure for moment magnification

Columns and stories in structures shall be designated as nonsway or sway columns or stories. The design of columns in nonsway frames or stories shall be based on Sec 6.3.10.6. The design of columns in sway frames or stories shall be based on Sec 6.3.10.7.

6.3.10.5.1 A column in a structure shall be permitted to be assumed as nonsway if the increase in column end moments due to second-order effects does not exceed 5 percent of the first-order end moments.

6.3.10.5.2 A story within a structure is permitted to be assumed as nonsway, if:

$$Q = \frac{\sum P_u \Delta_o}{V_{us} l_c} \leq 0.05 \qquad (6.6.17)$$

Where $\sum P_u$ and V_{us} are the total factored vertical load and the horizontal story shear, respectively, in the story being evaluated, and Δ_o is the first-order relative lateral deflection between the top and the bottom of that story due to V_{us}.

6.3.10.6 Procedure for moment magnification - nonsway

Compression members shall be designed for factored axial force P_u and the factored moment amplified for the effects of member curvature M_c where

$$M_c = \delta_{ns} M_2 \qquad (6.6.18)$$

Where,

$$\delta_{ns} = \frac{C_m}{1-\frac{P_u}{0.75P_c}} \geq 1.0 \quad (6.6.19)$$

And,

$$P_c = \frac{\pi^2 EI}{(kl_u)^2} \quad (6.6.20)$$

6.3.10.6.1 EI shall be taken as

$$EI = \frac{(0.2E_cI_g + E_sI_{se})}{1+\beta_{dns}} \quad (6.6.21)$$

Or,

$$EI = \frac{0.4E_cI_g}{1+\beta_{dns}} \quad (6.6.22)$$

Alternatively, EI shall be permitted to compute the value of I from Equation 6.6.15 dividing by $(1 + \beta_{dns})$.

6.3.10.6.2 The term β_{dns} shall be taken as the ratio of maximum factored axial sustained load to maximum factored axial load associated with the same load combination, but shall not be taken greater than 1.0.

6.3.10.6.3 The effective length factor, k shall be permitted to be taken as 1.0.

6.3.10.6.4 For members with no transverse load between supports, C_m shall be taken as

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \quad (6.6.23)$$

Where, M_1/M_2 is positive if the column is bent in single curvature, and negative if the member is bent in double curvature. For members with transverse loads between supports, C_m shall be taken as 1.0.

6.3.10.6.5 Factored moment, M_2, about each axis separately, in Equation 6.6.18 shall not be taken less than

$$M_{2,min} = P_u(15 + 0.03h) \quad (6.6.24)$$

Where, h is in mm and P_u in N. For members in which $M_{2,min}$ exceeds M_2, the value of C_m in Equation 6.6.23 shall either be taken equal to 1.0, or shall be based on the ratio of the computed end moments, M_1/M_2.

6.3.10.7 Procedure for moment magnification - Sway

Moments M_1 and M_2 at the ends of an individual compression member shall be taken as

$$M_1 = M_{1ns} + \delta_s M_{1s} \quad (6.6.25)$$

$$M_2 = M_{2ns} + \delta_s M_{2s} \quad (6.6.26)$$

Where, δ_s is computed according to Sec 6.3.10.7.3 or Sec 6.3.10.7.4.

6.3.10.7.1 Flexural members shall be designed for the total magnified end moments of the compression members at the joint.

6.3.10.7.2 The values of E_c and I given in Sec 6.3.10.4 shall be used for determining the effective length factor K and it shall not be less than 1.0.

6.3.10.7.3 The moment magnifier δ_s shall be calculated as

$$\delta_s = \frac{1}{1-Q} \geq 1 \qquad (6.6.27)$$

If δ_s calculated by Equation 6.6.27 exceeds 1.5, δ_s shall be calculated using second-order elastic analysis or 6.3.10.7.4.

6.3.10.7.4 Alternatively, it shall be permitted to calculate δ_s as

$$\delta_s = \frac{1}{1-\frac{\sum P_u}{0.75\sum P_c}} \geq 1 \qquad (6.6.28)$$

Where, $\sum P_u$ is the summation for all the factored vertical loads in a story and $\sum P_c$ is the summation for all sway-resisting columns in a storey. P_c is calculated using Equation 6.6.20 with k determined from Sec 6.3.10.7.2 and EI from Sec 6.3.10.6.1.

6.3.11 Axially Loaded Members Supporting Slab System

Axially loaded members supporting a slab system included within the scope of Sec 6.5.1 shall be designed as provided in Sec. 6.3 and in accordance with the additional requirements of Sec. 6.5.

6.3.12 Column Load Transmission through Floor System

If f_c' of a column is greater than 1.4 times that of the floor system, transmission of load through the floor system shall be provided by Sections 6.3.12.1, 6.3.12.2, or 6.3.12.3.

6.3.12.1 Concrete of strength specified for the column shall be placed in the floor at the column location. Top surface of the column concrete shall extend 600 mm into the slab from face of column. Column concrete shall be well integrated with floor concrete, and shall be placed in accordance with relevant provisions for construction joints of columns, walls etc. with beams, slabs etc. To avoid accidental placing of lower strength concrete in the columns, the structural designer shall indicate on the drawing where the high and low strength concretes are to be placed.

6.3.12.2 Strength of a column through a floor system shall be based on the lower value of concrete strength with vertical dowels and spirals as required.

6.3.12.3 For columns laterally supported on four sides by beams of approximately equal depth or by slabs, it shall be permitted to base strength of the column on an assumed concrete strength in the column joint equal to 75 percent of column concrete strength plus 35 percent of floor concrete strength. In the application of Sec 6.3.12.3, ratio of column concrete strength to slab concrete strength shall not be taken larger than 2.5 in design.

6.3.13 Composite Compression Members

6.3.13.1 All members reinforced longitudinally with structural steel shapes, pipe, or tubing with or without longitudinal bars shall be included in composite compression members.

6.3.13.2 A composite member strength shall be computed for the same limiting conditions applicable to ordinary reinforced concrete members.

6.3.13.3 Any axial load strength assigned to concrete of a composite member shall be transferred to the concrete by members or brackets in direct bearing on the composite member concrete.

6.3.13.4 All axial load strength not assigned to concrete of a composite member shall be developed by direct connection to the structural steel shape, pipe, or tube.

6.3.13.5 For evaluation of slenderness effects, radius of gyration, r, of a composite section shall be not greater than the value given by

$$r = \sqrt{\frac{(E_c I_g/5) + E_s I_{sx}}{(E_c A_g/5) + E_s A_{sx}}} \quad (6.6.29)$$

And, as an alternative to a more accurate calculation, EI in Equation 6.6.20 shall be taken either as Equation 6.6.21 or

$$EI = \frac{(E_c I_g/5)}{1+\beta_d} + E_s I_{sx} \quad (6.6.30)$$

6.3.13.6 Concrete core encased by structural steel

6.3.13.6.1 When a composite member is a structural steel encased concrete core, the thickness of the steel encasement shall be not less than $b\sqrt{\frac{f_y}{3E_s}}$ for each face of width b nor $b\sqrt{\frac{f_y}{8E_s}}$ for circular sections of diameter h

6.3.13.6.2 When computing A_{sx} and I_{sx}, longitudinal bars located within the encased concrete core shall be permitted to be used.

6.3.13.7 Spiral reinforcement around structural steel core

A composite member with spirally reinforced concrete around a structural steel core shall conform to Sections 6.3.13.7.1 to 6.3.13.7.4.

6.3.13.7.1 Design yield strength of structural steel core shall be the specified minimum yield strength for the grade of structural steel used but not to exceed 350 MPa.

6.3.13.7.2 Spiral reinforcement shall conform to Sec 6.3.9.3.

6.3.13.7.3 Longitudinal bars located within the spiral shall be not less than 0.01 nor more than 0.06 times net area of concrete section.

6.3.13.7.4 Longitudinal bars located within the spiral shall be permitted to be used in computing A_{sx} and I_{sx}.

6.3.13.8 Tie reinforcement around structural steel core

Laterally tied concrete around a structural steel core forming a composite member shall conform to Sections 6.3.13.8.1 to 6.3.13.8.7.

6.3.13.8.1 Design yield strength of structural steel core shall be the specified minimum yield strength for the grade of structural steel used but not to exceed 350 MPa.

6.3.13.8.2 Lateral ties shall extend completely around the structural steel core.

6.3.13.8.3 Lateral ties shall have a diameter not less than 0.02 times the greatest side dimension of composite member, except that ties shall not be smaller than 10 mm diameter and are not required to be larger than 16 mm diameter. Welded wire reinforcement of equivalent area shall be permitted.

6.3.13.8.4 Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 tie bar diameters, or 0.5 times the least side dimension of the composite member.

6.3.13.8.5 Longitudinal bars located within the ties shall be not less than 0.01 nor more than 0.06 times net area of concrete section.

6.3.13.8.6 A longitudinal bar shall be located at every corner of a rectangular cross section, with other longitudinal bars spaced not farther apart than one half the least side dimension of the composite member.

6.3.13.8.7 Longitudinal bars located within the ties shall be permitted to be used in computing A_{sx} and I_{sx}.

6.3.14 Bearing strength

6.3.14.1 Design bearing strength of concrete shall not exceed $\varphi(0.85f_c'A_1)$, except when the supporting surface is wider on all sides than the loaded area, then the design bearing strength of the loaded area shall be permitted to be multiplied by $\sqrt{A_2 / A_1}$ but by not more than 2 Figure. 6.6.6.

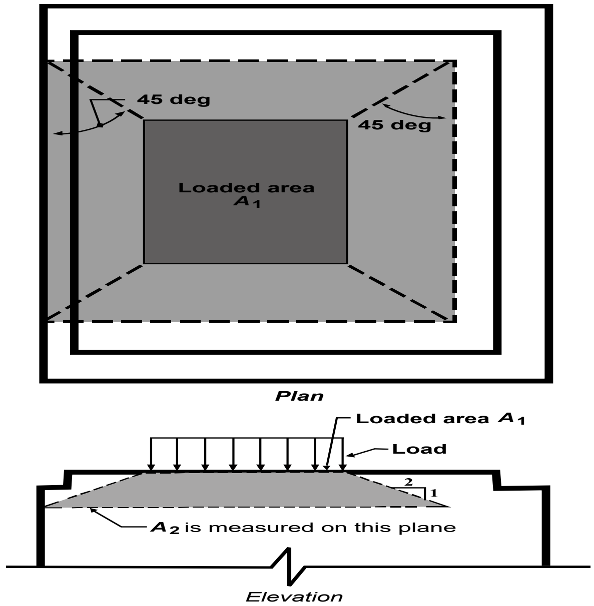


Figure 6.6.6 Determination of area A_2 in stepped or sloped supports using frustum

6.3.15 Design for Flexure

6.3.15.1 Design of Rectangular Beams

(a) Formula for singly reinforced beams: The following equations which are based on the simplified stress block of Sec 6.3.2.7, are applicable to singly reinforced rectangular beams along with T-beams where the neutral axis lies within the flange.

$$A_s = \frac{M_n}{f_y(d-a/2)} \qquad (6.6.31)$$

Where,

$$a = \frac{A_s f_y}{0.85 f_c' b} \qquad (6.6.32)$$

By estimating an initial value of a, Equation 6.6.31 can be used to determine an approximate value of A_s. The value can be substituted in Equation 6.6.32 to get a better estimate of a and hence a new $(d - \frac{a}{2})$ can be determined for substitution in Equation 6.6.31.

In Equation 6.6.31, a preliminary value of nominal flexural strength of section, M_n may be taken as factored moment at section, M_u divided by strength reduction factor, $\varphi = 0.9$. Reinforcement ratio, $\rho = A_s/bd$ calculated on the basis of A_s determined from Equation 6.6.31 shall not exceed ρ_{max} , where

$$\rho_{max} = 0.85\beta_1 \frac{f_c^{'}}{f_y}\frac{\varepsilon_u}{\varepsilon_u + 0.004} \qquad (6.6.33)$$

and, $\varepsilon_u = 0.003$

Additionally, A_s determined from Equation 6.6.31 shall have to satisfy the requirements of minimum reinforcement for members in flexure as per Sec 6.3.5.

Revised φ shall be determined from Sec 6.2.3.2 based on either $c/d_t = a/\beta_1 d_t$ or ε_t, where, ε_t is the net tensile strain in the reinforcement furthest from the compression face of the concrete at the depth d_t. Strain, ε_t may be calculated from Equation 6.6.33 by replacing 0.004 by ε_t and ρ_{max} by ρ respectively.

(b) Design formulae for doubly reinforced beams: A doubly reinforced beam shall be designed only when there is a restriction on depth of beam and maximum tensile reinforcement allowed cannot produce the required moment M_u.

To establish if doubly reinforced beam is required the following approach can be followed:

Determine,

$$\rho_{0.005} = 0.85\beta_1 \frac{f_c^{'}}{f_y}\frac{\varepsilon_u}{\varepsilon_u + 0.005} \qquad (6.6.34)$$

$$A_s = \rho_{0.005} bd$$

$$a = \frac{A_s f_y}{0.85 f_c^{'} b}$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) \qquad (6.6.35)$$

If ϕM_n is less than required moment M_u with $= 0.9$, a doubly reinforced beam is needed and then taking values of A_s and ϕM_n from above, put

$A_{s1} = A_s$ and $\phi M_{n1} = \phi M_n$

Then, the following values are to be evaluated,

$$\phi M_{n2} = M_u - \phi M_{n1} \qquad (6.6.36)$$

$$A_{s2} = \frac{\phi M_{n2}}{\phi f_y (d - d')}$$

Assuming compression steel yields (needs to be checked later),

$$A_s' = A_{s2}$$

$$A_s = A_{s1} + A_{s2}$$

Check $\rho \geq \bar{\rho}_{cy}$ for compression steel yielding, where

$$\bar{\rho}_{cy} = 0.85\beta_1 \frac{f_c'}{f_y}\frac{d'}{d}\frac{\varepsilon_u}{\varepsilon_u - \varepsilon_y} + \rho' \qquad (6.6.37)$$

If $\rho \geq \bar{\rho}_{cy}$ (i.e. compression steel yields),

Find $a = \frac{(A_s - A_s')f_y}{0.85 f_c' b}$ and find c, ε_t and confirm $\phi = 0.9$ in the above equations. Value of ϕ shall be determined from Sec 6.2.3.2 based on either $c/d_t = a/\beta_1 d_t$ or ε_t, as stated above for rectangular beams.

If compression steel does not yield, c is to be found from concrete section force equilibrium condition, C=T which will result in a quadratic equation of c. f_s' needs to be calculated from strain diagram and A_s' revised.

$$A_s' = A_{s2}\frac{f_y}{f_s'}$$

$$A_s = A_{s1} + A_{s2}$$

ε_t shall be calculated from c for finding ϕ.

6.3.15.2 Design of T-Beams

(a) General: For effective widths and other parameters for T, L or isolated beams, Sections 6.1.13.2 to 6.1.13.4 shall apply.

(b) Formulae for T-beams : A T-beam shall be treated as a rectangular beam if $a \leq h_f$ where $a$is obtained from Eq. 6.6.32 In using Eq. 6.6.32, if $A_s$is not known, it may be initially assumed as :

$$A_s = \frac{M_n}{f_y(d - h_f/2)} \qquad (6.6.38)$$

If a, thus obtained, is greater than h_f the beam shall be considered as a T-beam, in which case the following formulae shall be applicable :

$$A_{sf} = \frac{0.85 f_c'(b - b_w)h_f}{f_y} \qquad (6.6.39)$$

$$M_{n1} = A_{sf} f_y\left(d - h_f/2\right) \qquad (6.6.40)$$

$$M_{n2} = M_n - M_{n1} \qquad (6.6.41)$$

$$A_s - A_{sf} = \frac{M_{n2}}{f_y(d - a/2)} \qquad (6.6.42)$$

$$a = \frac{\left(A_s - A_{sf}\right)f_y}{0.85 f_c' b_w} \qquad (6.6.43)$$

By estimating an initial value of a, Eq. 6.6.42 can be used to obtain an approximate value of $(A_s - A_{sf})$ That value of $(A_s - A_{sf})$ can be substituted in Eq. 6.6.43 to get a better estimate of a.

Net tensile strain requirements will be satisfied as long as depth to neutral axis, $c \leq 0.429\, d_t$. This will occur if:

$$\rho_w < \rho_{w,max}$$

Where,

$$\rho_w = \frac{A_s}{b_w d} \qquad (6.6.44)$$

$$\rho_{w,max} = \rho_{max} + \rho_f \qquad (6.6.45)$$

$$\rho_f = \frac{A_{sf}}{b_w d} \qquad (6.6.46)$$

and, ρ_{max} is as defined by Eq. 6.6.33. For c/d_t ratios between 0.429 and 0.375, equivalent to ρ_w between the $\rho_{w,max}$ from Eq. 6.6.45 and $\rho_{w,max}$ calculated by substituting ρ from Eq. 6.6.33 with 0.005 in place of 0.004 and ρ for ρ_{max} , the strength reduction factor, ϕ must be adjusted for ε_t in accordance with Sec 6.2.3.2.

6.4 Shear and Torsion

6.4.1 Shear Strength

6.4.1.1 Except for members designed in accordance with Appendix I, design of cross sections subject to shear shall be based on

$$\phi V_n \geq V_u \qquad (6.6.47)$$

Where, V_u is the factored shear force at the section considered and V_n is nominal shear strength given by

$$V_n = V_c + V_s \qquad (6.6.48)$$

Where, V_c is nominal shear strength provided by concrete calculated in accordance with Sec 6.4.2, or Sec 6.4.10, and V_s is nominal shear strength provided by shear reinforcement calculated in accordance with Sec 6.4.3, Sec 6.4.8.9, or Sec 6.4.10.

6.4.1.1.1 The effect of any openings in members shall be considered in determining V_n .

6.4.1.1.2 In evaluating V_c , whenever applicable, effects of axial tension due to creep and shrinkage in restrained members shall be considered and effects of inclined flexural compression in variable depth members shall be permitted to be included.

6.4.1.2 Except as allowed in Sec 6.4.1.2.1, the values of $\sqrt{f_c'}$ used in this Chapter shall not exceed 8.3 MPa.

6.4.1.2.1 Values of $\sqrt{f_c'}$ greater than 8.3 MPa shall be permitted in computing V_c, $\boldsymbol{V_{ci}}$, and $\boldsymbol{V_{cw}}$ for reinforced concrete beams and concrete joist construction having minimum web reinforcement in accordance with Sec 6.4.3.5.3, or Sec 6.4.4.5.2.

6.4.1.3 Computation of maximum V_u at supports in accordance with Sec 6.4.1.3.1 shall be permitted if all conditions (a), (b), and (c) are satisfied:

(a) Support reaction, in direction of applied shear, introduces compression into the end regions of member;

(b) Loads are applied at or near the top of the member;

(c) No concentrated load occurs between face of support and location of critical section defined in Sec 6.4.1.3.1.

6.4.1.3.1 Sections located less than a distance d from face of support shall be permitted to be designed for V_u computed at a distance d.

6.4.1.4 For deep beams, brackets and corbels, walls, and slabs and footings, the special provisions of Sections 6.4.6 to 6.4.10 shall apply.

6.4.2 Contribution of Concrete to Shear Strength

6.4.2.1 V_c shall be computed by provisions of Sections 6.4.2.1.1 to 6.4.2.1.3, unless a more detailed calculation is made in accordance with Sec 6.4.2.2. Throughout this Chapter, except in Sec 6.4.5, λ shall be as defined in Sec 6.1.8.1.

6.4.2.1.1 For members subject to shear and flexure only,

$$V_c = 0.17\lambda\sqrt{f'_c}\,b_w d \qquad (6.6.49)$$

6.4.2.1.2 For members subject to axial compression,

$$V_c = 0.17\left(1 + \frac{N_u}{14A_g}\right)\lambda\sqrt{f'_c}\,b_w d \qquad (6.6.50)$$

Quantity N_u/A_g shall be expressed in MPa.

6.4.2.1.3 For members subject to significant axial tension, V_c shall be taken as zero unless a more detailed analysis is made using Sec 6.4.2.2.3.

6.4.2.2 V_c shall be permitted to be computed by more detailed calculation of Sections 6.4.2.2.1 to 6.4.2.2.3.

6.4.2.2.1 For members subject to shear and flexure only,

$$V_c = \left(0.16\lambda\sqrt{f'_c} + 17\rho_w \frac{V_u d}{M_u}\right)b_w d \qquad (6.6.51)$$

But, not greater than $0.29\lambda\sqrt{f_c'}b_w d$. When computing V_c by Eq. 6.6.51, $V_u d/M_u$ shall not be taken greater than 1.0, where M_u occurs simultaneously with V_u at section considered.

6.4.2.2.2 For members subject to axial compression, it shall be permitted to compute V_c using Eq. 6.6.51 with M_m substituted for M_u and $V_u d / M_u$ not then limited to 1.0, where

$$M_m = M_u - N_u \frac{(4h-d)}{8} \quad (6.6.52)$$

However, V_c shall not be taken greater than

$$V_c = 0.29\lambda\sqrt{f'_c}b_w d\sqrt{1+\frac{0.29N_u}{A_g}} \quad (6.6.53)$$

N_u/A_g shall be expressed in MPa. When M_m as computed by Eq. 6.6.52 is negative, V_c shall be computed by Eq. 6.6.53.

6.4.2.2.3 For members subject to significant axial tension,

$$V_c = 0.17(1+\frac{0.29N_u}{A_g})\lambda\sqrt{f'_c}b_w d \quad (6.6.54)$$

But, not less than zero, where N_u is negative for tension. N_u/A_g shall be expressed in MPa.

6.4.2.3 For circular members, the area used to compute V_c shall be taken as the product of the diameter and effective depth of the concrete section. It shall be permitted to take d as 0.80 times the diameter of the concrete section.

6.4.3 Shear Strength Contribution of Reinforcement

6.4.3.1 Types of shear reinforcement

6.4.3.1.1 The following types of shear reinforcement shall be permitted:

(a) Stirrups perpendicular to axis of member;

(b) Welded wire reinforcement with wires located perpendicular to axis of member;

(c) Spirals, circular ties, or hoops.

(d) Stirrups making an angle of 45° or more with longitudinal tension reinforcement;

(e) Longitudinal reinforcement with bent portion making an angle of 30° or more with the longitudinal tension reinforcement;

(f) Combinations of stirrups and bent longitudinal reinforcement.

6.4.3.2 The values of f_y and f_{yt} used in design of shear reinforcement shall not exceed 420 MPa, except the value shall not exceed 550 MPa for welded deformed wire reinforcement.

6.4.3.3 Stirrups and other bars or wires used as shear reinforcement shall extend to a distance $d$from extreme compression fiber and shall be developed at both ends according to Sec 8.2.10.

6.4.3.4 Limits in spacing for shear reinforcement

6.4.3.4.1 Spacing of shear reinforcement placed perpendicular to member axis shall not exceed $\frac{d}{2}$ nor 600 mm.

6.4.3.4.2 The spacing of inclined stirrups and bent longitudinal reinforcement shall be such that every 45-degree line, extending toward the reaction from mid-depth of member $\frac{d}{2}$ to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.

6.4.3.4.3 Where, V_s exceeds $0.33\sqrt{f_c'}b_w d$ maximum spacing given in Sections 6.4.3.4.1 and 6.4.3.4.2 shall be reduced by one-half.

6.4.3.5 Minimum shear reinforcement

6.4.3.5.1 A minimum area of shear reinforcement, $A_{v,min}$, shall be provided in all reinforced concrete flexural members, where V_u exceeds $0.5\phi V_C$, except in members satisfying one or more of (a) to (f):

(a) Footings and solid slabs;

(b) Hollow-core units with total untopped depth not greater than 315 mm and hollow-core units where V_u is not greater than $0.5\phi V_{cw}$;

(c) Concrete joist construction defined by Sec 6.1.14;

(d) Beams with h not greater than 250 mm;

(e) Beam integral with slabs with $h$not greater than 600 mm and not greater than the larger of 2.5 times thickness of flange, and 0.5 times width of web;

(f) Beams constructed of steel fiber-reinforced, normal weight concrete with f_c' not exceeding 40 MPa, h not greater than 600 mm, and V_u not greater than $0.17\phi\sqrt{f_c'}b_w d$.

6.4.3.5.2 Minimum shear reinforcement requirements of Sec 6.4.3.5.1 shall be permitted to be waived if shown by test that required M_n and V_n can be developed when shear reinforcement is omitted. Such tests shall simulate effects of differential settlement, creep, shrinkage, and temperature change, based on a realistic assessment of such effects occurring in service.

6.4.3.5.3 Where shear reinforcement is required by Sec 6.4.3.5.1 or for strength and where Sec 6.4.4.1 allows torsion to be neglected, $A_{v,min}$ shall be computed by

$$A_{v,min} = 0.062\sqrt{f'_c}\frac{b_w s}{f_{yt}} \tag{6.6.55}$$

But, shall not be less than $(0.35 b_w s)/f_{yt}$.

6.4.3.6 Design of shear reinforcement

6.4.3.6.1 Where V_u exceeds ϕV_c, shear reinforcement shall be provided to satisfy Equations 6.6.47 and 6.6.48, where V_s shall be computed in accordance with Sections 6.4.3.6.2 to 6.4.3.6.9.

6.4.3.6.2 Where shear reinforcement perpendicular to axis of member is used,

$$V_s = \frac{A_v f_{yt} d}{s} \tag{6.6.56}$$

Where, A_v is the area of shear reinforcement within spacing s.

6.4.3.6.3 Where circular ties, hoops, or spirals are used as shear reinforcement, V_s shall be computed using Eq. 6.6.56 where d is defined in Sec 6.4.2.3 for circular members, A_v shall be taken as two times the area of the bar in a circular tie, hoop, or spiral at a spacing S, $\boldsymbol{s}$ is measured in a direction parallel to longitudinal reinforcement, and f_{yt} is the specified yield strength of circular tie, hoop, or spiral reinforcement.

6.4.3.6.4 Where inclined stirrups are used as shear reinforcement,

$$V_s = \frac{A_v f_{yt}(\sin\propto + \cos\propto) d}{s} \tag{6.6.57}$$

Where, α is angle between inclined stirrups and longitudinal axis of the member, and s is measured in direction parallel to longitudinal reinforcement.

6.4.3.6.5 Where shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support,

$$V_s = A_v f_y \sin\propto \tag{6.6.58}$$

But, not greater than $0.25\sqrt{f_c'} b_w d$, where α is angle between bent-up reinforcement and longitudinal axis of the member.

6.4.3.6.6 Where shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, V_s shall be computed by Eq. 6.6.57.

6.4.3.6.7 Only the center three-fourths of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

6.4.3.6.8 Where more than one type of shear reinforcement is used to reinforce the same portion of a member, V_s shall be computed as the sum of the values computed for the various types of shear reinforcement.

6.4.3.6.9 V_s shall not be taken greater than $0.66\sqrt{f_c' d_w d}$.

6.4.4 Design for Torsion

Design for torsion shall be done as per Sections 6.4.4.1 to 6.4.4.6. A beam subjected to torsion is idealized as a thin-walled tube with the core concrete cross section in a solid beam neglected as shown in Figure 6.6.7.

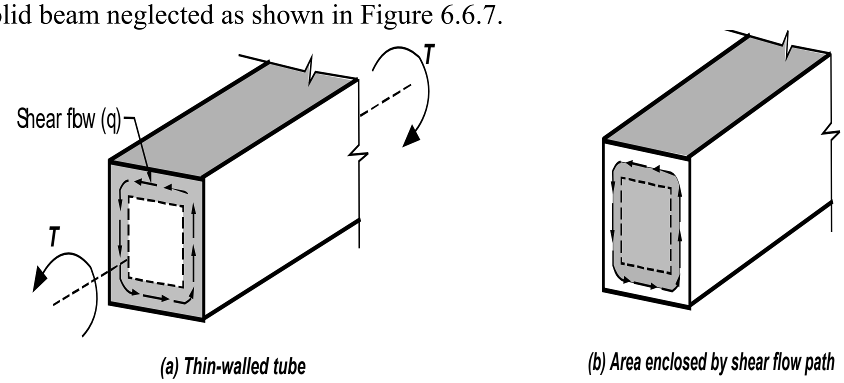


Figure 6.6.7 (a) Torsional resistance by thin-walled tube; (b) Ineffective inner area enclosed by shear flow path

6.4.4.1 Threshold torsion

It shall be permitted to neglect torsion effects if the factored torsional moment T_u is less than:

(a) For members not subjected to axial tension or compression

$$0.083\phi\lambda\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)$$

(b) For members subjected to an axial compressive or tensile force

$$0.083\phi\lambda\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{0.33A_g\lambda\sqrt{f_c'}}}$$

The overhanging flange width used in computing A_{cp} and p_{cp} for members cast monolithically with a slab shall conform to Sec 6.5.2.4. For a hollow section, A_g shall be used in place of A_{cp} in Sec 6.4.4.1, and the outer boundaries of the section shall conform to Sec 6.5.2.4.

6.4.4.1.1 For members cast monolithically with a slab and for isolated members with flanges, the overhanging flange width used to compute A_{cp} and P_{cp} shall conform to Sec 6.5.2.4, except that the overhanging flanges shall be neglected in cases where the parameter A^2cp / P_{cp} calculated for a beam with flanges is less than that computed for the same beam ignoring the flanges.

6.4.4.2 Evaluation of factored torsional moment

6.4.4.2.1 If the factored torsional moment, T_u, in a member is required to maintain equilibrium Figure 6.6.8 and exceeds the minimum value given in Sec 6.4.4.1, the member shall be designed to carry T_u in accordance with Sections 6.4.4.3 to 6.4.4.6.

6.4.4.2.2 In a statically indeterminate structure where reduction of the torsional moment in a member can occur due to redistribution of internal forces upon cracking Figure 6.6.9, the maximum T_u shall be permitted to be reduced to the values given in (a), or (b) as applicable:

(a) For members, at the sections described in Sec 6.4.4.2.4 and not subjected to axial tension or compression

$$0.33\phi\lambda\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)$$

(b) For members subjected to an axial compressive or tensile force

$$0.33\phi\lambda\sqrt{f_c'}\left(\frac{A_{cp}^2}{p_{cp}}\right)\sqrt{1+\frac{N_u}{0.33A_g\lambda\sqrt{f_c'}}}$$

In (a), or (b), the correspondingly redistributed bending moments and shears in the adjoining members shall be used in the design of these members. For hollow sections, A_{cp} shall not be replaced with A_g in Sec 6.4.4.2.2.

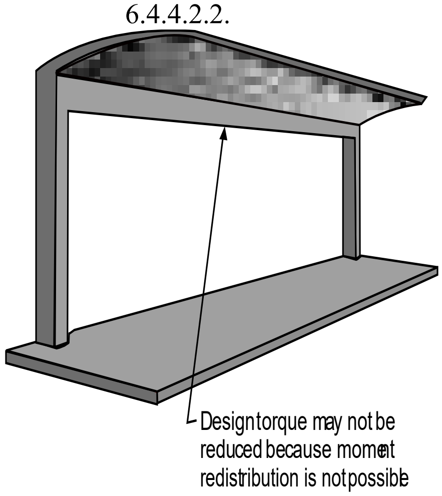


Figure 6.6.8 Design torque may not be reduced

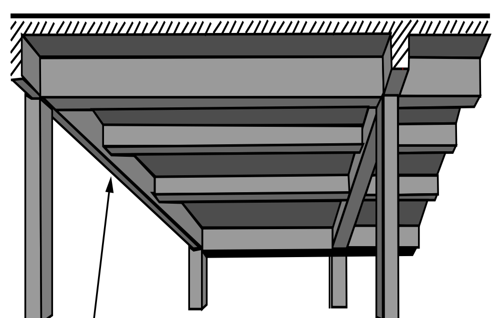


Figure 6.6.9 Design torque may be reduced

6.4.4.2.3 It shall be permitted to take the torsional loading from a slab as uniformly distributed along the member, if not determined by a more exact analysis.

6.4.4.2.4 Sections located closer than a distance *d* from the face of a support shall be designed for not less than T_u computed at a distance *d*. If a concentrated torque occurs within this distance, the critical section for design shall be at the face of the support.

6.4.4.3 Torsional moment strength

6.4.4.3.1 The cross-sectional dimensions shall be such that:

(a) For solid sections

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7 A_{oh}^2}\right)^2} \leq \phi\left(\frac{V_c}{b_w d} + 0.66\sqrt{f_c'}\right) \qquad (6.6.59)$$

(b) For hollow sections

$$\left(\frac{V_u}{b_w d}\right) + \left(\frac{T_u P_h}{1.7 A_{oh}^2}\right) \leq \phi\left(\frac{V_c}{b_w d} + 0.66\sqrt{f_c'}\right) \qquad (6.6.60)$$

Superposition of shear stresses due to shear and torsion in hollow sections given by the left side of the inequality Sec 6.4.14 is illustrated by Figure 6.6.10(a) and that in solid sections given by the left side of the inequality Sec 6.4.13 is illustrated by Figure 6.6.10(b).

6.4.4.3.2 If the wall thickness varies around the perimeter of a hollow section, Eq. 6.6.60 shall be evaluated at the location where the left-hand side of Eq. 6.6.60 is a maximum.

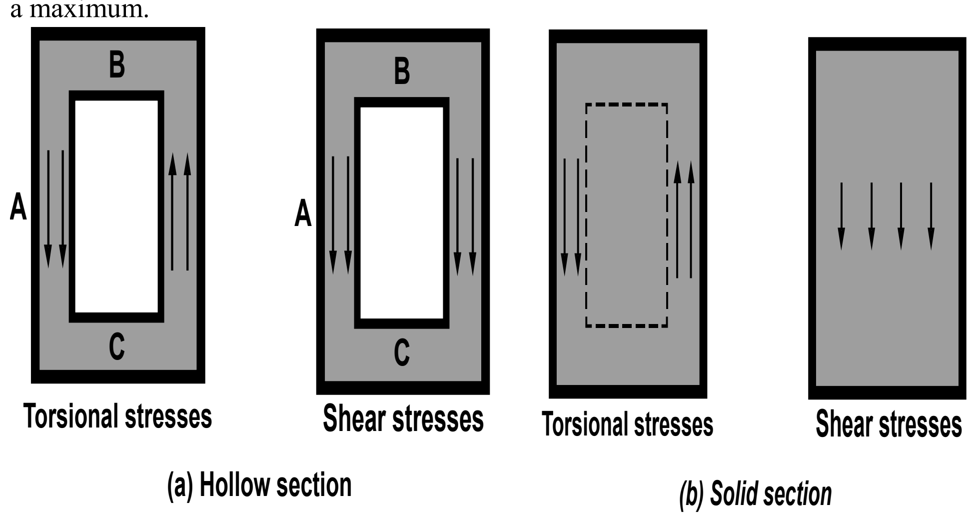


Figure 6.6.10 Superposition of torsional and shear stresses

6.4.4.3.3 If the wall thickness is less than A_{oh} / p_h, the second term in Eq. 6.4.14 shall be taken as $\left(\frac{T_u}{1.7 A_{oh} t}\right)$

Where, t is the thickness of the wall of hollow section at the location where the stresses are being checked.

6.4.4.3.4 The values of f_y and f_{yt} used for design of torsional reinforcement shall not exceed 420 MPa.

6.4.4.3.5 Where T_u exceeds the threshold torsion, design of the cross section shall be based on

$$\phi T_n \geq T_u \tag{6.6.61}$$

6.4.4.3.6 T_u shall be computed by

$$T_n = \frac{2A_o A_t f_{yt}}{s} cot\theta \tag{6.6.62}$$

Where, A_o shall be determined by analysis except that it shall be permitted to take A_o equal to $0.85A_{oh}$; θ shall not be taken smaller than 30º nor larger than 60º. It shall be permitted to take θ equal to 45º.

6.4.4.3.7 The additional area of longitudinal reinforcement to resist torsion, A_l, shall not be less than

$$A_l = \frac{A_t}{s} p_h \left(\frac{f_{yt}}{f_y}\right) cot^2\theta \tag{6.6.63}$$

Where, θ shall be the same value used in Eq. 6.6.62 and A_t/s shall be taken as the amount computed from Eq. 6.6.62 not modified in accordance with Sec 6.4.4.5.2 or Sec 6.4.4.5.3; f_{yt} refers to closed transverse torsional reinforcement, and f_y refers to longitudinal torsional reinforcement.

6.4.4.3.8 Reinforcement required for torsion shall be added to that required for the shear, moment, and axial force that act in combination with the torsion. The most restrictive requirements for reinforcement spacing and placement shall be met.

6.4.4.3.9 It shall be permitted to reduce the area of longitudinal torsion reinforcement in the flexural compression zone by an amount equal to, $M_u / (0.9 d f_y)$, where M_u occurs at the section simultaneously with T_u, except that the reinforcement provided shall not be less than that required by Sec 6.4.4.5.3 or Sec 6.4.4.6.2.

6.4.4.4 Details of torsional reinforcement

6.4.4.4.1 Torsion reinforcement shall consist of longitudinal bars or tendons and one or more of the following:

(a) Closed stirrups or closed ties, perpendicular to the axis of the member;

(b) A closed cage of welded wire reinforcement with transverse wires perpendicular to the axis of the member;

(c) Spiral reinforcement.

6.4.4.4.2 Transverse torsional reinforcement shall be anchored by one of the following:

(a) A 135^o standard hook, or seismic hook as defined in Sec 8.1.2.1(d) Chapter 8, around a longitudinal bar;

(b) According to Sec 8.2.10.2 Chapter 8 in regions where the concrete surrounding the anchorage is restrained against spalling by a flange or slab or similar member.

6.4.4.4.3 Longitudinal torsion reinforcement shall be developed at both ends.

6.4.4.4.4 For hollow sections in torsion, the distance from the centerline of the transverse torsional reinforcement to the inside face of the wall of the hollow section shall not be less than $0.5 A_{oh} / P_h$.

6.4.4.5 Minimum torsion reinforcement

6.4.4.5.1 A minimum area of torsional reinforcement shall be provided in all regions, where T_u exceeds the threshold torsion given in Sec 6.4.4.1.

6.4.4.5.2 Where torsional reinforcement is required by Sec 6.4.4.5.1, the minimum area of transverse closed stirrups shall be computed by

$$A_v + 2A_t = 0.062\sqrt{f'_c}\frac{b_w s}{f_{yt}} \tag{6.6.64}$$

But, shall not be less than $(0.35 b_w\, s)/f_{yt}$.

6.4.4.5.3 Where torsional reinforcement is required by Sec 6.4.4.5.1, the minimum total area of longitudinal torsional reinforcement, $A_{l,min}$, shall be computed by

$$A_{l,min} = \frac{0.42\sqrt{f'_c}A_{cp}}{f_y} - (\frac{A_t}{s})p_h\left(\frac{f_{yt}}{f_y}\right) \tag{6.6.65}$$

Where, A_t/s shall not be taken less than $0.175 b_w/f_{yt}$; f_{yt} refers to closed transverse torsional reinforcement, and f_y refers to longitudinal reinforcement.

6.4.4.6 Spacing of torsion reinforcement

6.4.4.6.1 The spacing of transverse torsion reinforcement shall not exceed the smaller of $P_h / 8$ or 300 mm.

6.4.4.6.2 The longitudinal reinforcement required for torsion shall be distributed around the perimeter of the closed stirrups with a maximum spacing of 300 mm. The longitudinal bars shall be inside the stirrups. There shall be at least one longitudinal bar in each corner of the stirrups. Longitudinal bars shall have a diameter at least 0.042 times the stirrup spacing, but not less than 10 mm diameter.

6.4.4.6.3 Torsional reinforcement shall be provided for a distance of at least $(\boldsymbol{b_t} + \boldsymbol{d})$ beyond the point required by analysis.

6.4.5 Shear-Friction

6.4.5.1 Application of provisions of Sec 6.4.5 shall be for cases where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times.

6.4.5.2 Design of cross sections subject to shear transfer as described in Sec 6.4.5.1 shall be based on Eq. 6.6.47, where V_n is calculated in accordance with provisions of Sec 6.4.5.3 or Sec 6.4.5.4.

6.4.5.3 A crack shall be assumed to occur along the shear plane considered. The required area of shear-friction reinforcement A_{vf} across the shear plane shall be designed using either Sec 6.4.5.4 or any other shear transfer design methods that result in prediction of strength in substantial agreement with results of comprehensive tests.

6.4.5.3.1 Provisions of Sections 6.4.5.5 to 6.4.5.10 shall apply for all calculations of shear transfer strength.

6.4.5.4 Design method for shear-friction

6.4.5.4.1 Where shear-friction reinforcement is perpendicular to the shear plane, V_n shall be computed by

$$V_n = A_{vf} f_y \mu \qquad (6.6.66)$$

Where, μ is coefficient of friction in accordance with Sec 6.4.5.4.3.

6.4.5.4.2 Where shear-friction reinforcement is inclined to the shear plane, such that the shear force produces tension in shear-friction reinforcement Figure 6.6.11, V_n shall be computed by

$$V_n = A_{vf} f_y (\mu sin\alpha + cos\alpha) \qquad (6.6.67)$$

Where, α is angle between shear-friction reinforcement and shear plane.

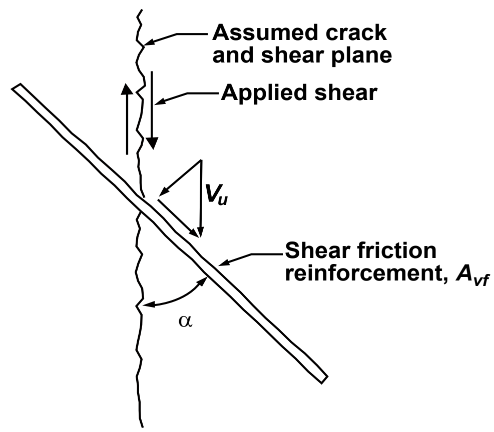


Figure 6.6.11 Shear-friction reinforcement at an angle to assumed crack

6.4.5.4.3 The coefficient of friction μ in Eq. 6.6.66 and Eq. 6.6.67 shall be taken as:

(a)	Concrete placed monolithically	1.4λ
(b)	Concrete placed against hardened concrete with surface intentionally roughened as specified in Sec 6.4.5.9	1.0λ
(c)	Concrete placed against hardened concrete not intentionally roughened	0.6λ
(d)	Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see 6.4.5.10)	0.7λ

Where, $\lambda = 1.0$ for normal weight concrete and 0.75 for all light weight concrete. Otherwise, λ shall be determined based on volumetric proportions of light weight and normal weight aggregates as specified in Sec 6.1.8.1, but shall not exceed 0.85.

6.4.5.5 For normal weight concrete either placed monolithically or placed against hardened concrete with surface intentionally roughened as specified in Sec 6.4.5.9, V_n shall not exceed the smallest of $0.2f_c^{'}A_c$, $(3.3 + 0.08f_c^{'})A_c$ and $11A_c$, where A_c is area of concrete section resisting shear transfer. For all other cases, V_n shall not exceed the smaller of $0.2f_c^{'}A_c$ or $5.5A_c$. Where concretes of different strengths are cast against each other, the value of $f_c^{'}$ used to evaluate V_n shall be that of the lower-strength concrete.

6.4.5.6 The value of f_y used for design of shear-friction reinforcement shall not exceed 420 MPa.

6.4.5.7 Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane shall be permitted to be taken as additive to $A_{vf} f_y$, the force in the shear-friction reinforcement, when calculating required A_{vf}.

6.4.5.8 Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop f_y on both sides by embedment, hooks, or welding to special devices.

6.4.5.9 For the purpose of Sec 6.4.5, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If μ is assumed equal to 1.0λ, interface shall be roughened to a full amplitude of approximately 6 mm.

6.4.5.10 When shear is transferred between as-rolled steel and concrete using headed studs or welded reinforcing bars, steel shall be clean and free of paint.

6.4.6 Deep Beams

6.4.6.1 The provisions of Sec 6.4.6 shall apply to members with l_n not exceeding four times the overall member depth or regions of beams with concentrated loads within twice the member depth from the support that are loaded on one face and supported on the opposite face so that compression struts can develop between the loads and supports. See also Sec 8.2.7.6 Chapter 8.

6.4.6.2 Deep beams shall be designed using provisions of either nonlinear analysis as permitted in Sec 6.3.7.1, or Appendix I.

6.4.6.3 V_n for deep beams shall not exceed $0.83\sqrt{f_c'}b_w d$.

6.4.6.4 The area of shear reinforcement perpendicular to the flexural tension reinforcement, A_v, shall not be less than $0.0025 b_w s$, and s shall not exceed the smaller of $\frac{d}{5}$ and 300 mm.

6.4.6.5 The area of shear reinforcement parallel to the flexural tension reinforcement, A_{vh}, shall not be less than $0.0015 b_w s_2$, and s_2 shall not exceed the smaller of $\frac{d}{5}$ and 300 mm.

6.4.6.6 It shall be permitted to provide reinforcement satisfying Sec I.3.3 Appendix I instead of the minimum horizontal and vertical reinforcement specified in Sections 6.4.6.4 and 6.4.6.5.

6.4.7 Provisions for Brackets and Corbels

6.4.7.1 Brackets and corbels, Figures 6.6.12 and 6.6.13, with a shear span-to-depth ratio $\frac{a_v}{d}$ less than 2 shall be permitted to be designed using Appendix I. Design shall be permitted using Sections 6.4.7.3 and 6.4.7.4 for brackets and corbels with:

(a) $\frac{a_v}{d}$ not greater than 1, and

(b) Subject to factored horizontal tensile force, N_{uc}, not larger than V_u.

The requirements of Sections 6.4.7.2, 6.4.7.5, 6.4.7.6, and 6.4.7.7 shall apply to design of brackets and corbels. Effective depth d shall be determined at the face of the support.

6.4.7.2 Depth at outside edge of bearing area shall not be less than $0.5d$.

6.4.7.3 Section at face of support shall be designed to resist simultaneously V_u, a factored moment $\left[V_u a_v + N_{uc}(h-d)\right]$, and a factored horizontal tensile force N_{uc}.

6.4.7.3.1 In all design calculations in accordance with Sec 6.4.7, ϕ shall be taken equal to 0.75.

6.4.7.3.2 Design of shear-friction reinforcement, A_{vf} to resist V_u shall be in accordance with Sec 6.4.5.

(a) For normal weight concrete, V_n shall not exceed the smallest of (i) $0.2f_c^{'}b_w d$, (ii) $(3.3 + 0.08f_c^{'})b_w d$, and (iii) $11b_w d$.

(b) For all-lightweight or sand-lightweight concrete, V_n shall not be taken greater than the smaller of $\left(0.2 - \frac{0.07a_v}{d}\right) f_c^{'}b_w d$ and $(5.5 - \frac{1.9a_v}{d})b_w d$.

6.4.7.3.3 Reinforcement A_f to resist factored moment $\left[V_u a_v + N_{uc}(h-d)\right]$ shall be computed in accordance with Sections 6.3.2 and 6.3.3.

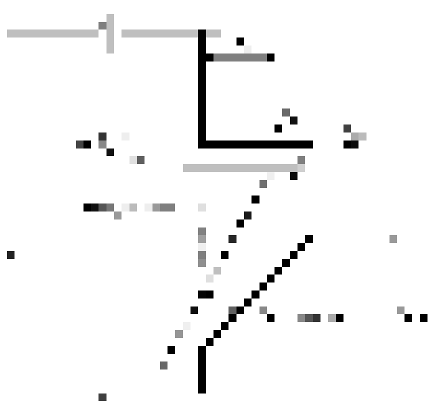


Figure 6.6.12 Structural action of a corbel

Figure 6.6.13 Notation used in Section 6.4.7

6.4.7.3.4 Reinforcement A_n to resist factored tensile force N_{uc} shall be determined from $\phi A_n f_y \geq N_{uc}$. Factored tensile force, N_{uc}, shall not be taken less than $0.2V_u$ unless provisions are made to avoid tensile forces. N_{uc} shall be regarded as live load even if tension results from restraint of creep, shrinkage, or temperature change.

6.4.7.3.5 Area of primary tension reinforcement A_{sc} shall not be less than the larger of $(A_f + A_n)$ and $\left(\frac{2A_{vf}}{3} + A_n\right)$.

6.4.7.4 Total area, A_h, of closed stirrups or ties parallel to primary tension reinforcement shall not be less than $0.5(A_{sc} - A_n)$. Distribute A_h uniformly within $\left(\frac{2}{3}\right) d$ adjacent to primary tension reinforcement.

6.4.7.5 $\frac{A_{sc}}{bd}$ shall not be less than $0.04\left(\frac{f_c'}{f_y}\right)$.

6.4.7.6 At front face of bracket or corbel, primary tension reinforcement shall be anchored by one of the following:

(a) By a structural weld to a transverse bar of at least equal size; weld to be designed to develop f_y of primary tension reinforcement;

(b) By bending primary tension reinforcement back to form a horizontal loop; or

(c) By some other means of positive anchorage.

6.4.7.7 Bearing area on bracket or corbel neither shall project beyond straight portion of primary tension reinforcement, nor shall project beyond interior face of transverse anchor bar (if one is provided).

6.4.8 Provisions for Walls

6.4.8.1 Design of walls for shear forces perpendicular to face of wall shall be in accordance with provisions for slabs in Sec 6.4.10. Design for horizontal in-plane shear forces in a wall shall be in accordance with Sections 6.4.8.2 to 6.4.8.9. Alternatively, it shall be permitted to design walls with a height not exceeding two times the length of the wall for horizontal shear forces in accordance with Appendix I and Sections 6.4.8.9.2 to 6.4.8.9.5.

6.4.8.2 Design of horizontal section for shear in plane of wall shall be based on Equations 6.6.47 and 6.6.48, where V_c shall be in accordance with Sec 6.4.8.5 or Sec 6.4.8.6 and V_s shall be in accordance with Sec 6.4.8.9.

6.4.8.3 V_n at any horizontal section for shear in plane of wall shall not be taken greater than $0.83\sqrt{f_c'}hd$, where h is thickness of wall, and d is defined in Sec 6.4.8.4.

6.4.8.4 For design for horizontal shear forces in plane of wall, d shall be taken equal to $0.8l_w$. A larger value of d, equal to the distance from extreme compression fiber to center of force of all reinforcement in tension, shall be permitted to be used when determined by a strain compatibility analysis.

6.4.8.5 If a more detailed calculation is not made in accordance with Sec 6.4.8.6, V_c shall not be taken greater than $0.17\lambda\sqrt{f_c'}hd$ for walls subject to axial compression, or V_c shall not be taken greater than the value given in 6.4.2.2.3 for walls subject to axial tension.

6.4.8.6 V_c shall be permitted to be the lesser of the values computed from Equations 6.6.68 and 6.6.69

$$V_c = 0.27\lambda\sqrt{f_c'}hd + \frac{N_u d}{4l_w} \qquad (6.6.68)$$

Or,

$$V_c = \left[0.05\lambda\sqrt{f'_c} + \frac{l_w\left(0.1\lambda\sqrt{f_c'}+0.2\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u}-\frac{l_w}{2}}\right]hd \qquad (6.6.69)$$

Where, l_w is the overall length of the wall, and N_u is positive for compression and negative for tension. If $\left(\frac{M_u}{V_u}-\frac{l_w}{2}\right)$ is negative, Eq. 6.6.69 shall not apply.

6.4.8.7 Sections located closer to wall base than a distance $\frac{l_w}{2}$ or one-half the wall height, whichever is less, shall be permitted to be designed for the same V_c as that computed at a distance $\frac{l_w}{2}$ or one-half the height.

6.4.8.8 Where V_u is less than $0.5\phi V_c$, reinforcement shall be provided in accordance with Sec 6.4.8.9 or in accordance with Sec. 6.6. Where V_u exceeds $0.5\phi V_c$, wall reinforcement for resisting shear shall be provided in accordance with Sec 6.4.8.9.

6.4.8.9 Design of shear reinforcement for walls

6.4.8.9.1 Where V_u exceeds ϕV_c, horizontal shear reinforcement shall be provided to satisfy Equations 6.6.47 and 6.6.48, where V_s shall be computed by

$$V_s = \frac{A_v f_y d}{s} \qquad (6.6.70)$$

Where, A_v is area of horizontal shear reinforcement within spacing s, and d is determined in accordance with Sec 6.4.8.4. Vertical shear reinforcement shall be provided in accordance with Sec 6.4.8.9.4.

6.4.8.9.2 Ratio of horizontal shear reinforcement area to gross concrete area of vertical section, ρ_t shall not be less than 0.0025.

6.4.8.9.3 Spacing of horizontal shear reinforcement shall not exceed the smallest of $\frac{l_w}{5}$, $3h$, and 450 mm, where l_w is the overall length of the wall.

6.4.8.9.4 Ratio of vertical shear reinforcement area to gross concrete area of horizontal section, ρ_l shall not be less than the larger of 0.0025 and the value obtained from:

$$\rho_l = 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025) \qquad (6.6.71)$$

The value of ρ_l calculated by Eq. 6.6.71 need not be greater than ρ_t required by Sec 6.4.8.9.1. In Eq. 6.6.71, l_w is the overall length of the wall, and h_w is the overall height of the wall.

6.4.8.9.5 Spacing of vertical shear reinforcement shall not exceed the smallest of $\frac{l_w}{3}$, $3h$, and 450 mm, where l_w is the overall length of the wall.

6.4.9 Transfer of Moments to Columns

6.4.9.1 When gravity load, wind, earthquake, or other lateral forces cause transfer of moment at connections of framing elements to columns, the shear resulting from moment transfer shall be considered in the design of lateral reinforcement in the columns.

6.4.9.2 Except for connections not part of a primary seismic load-resisting system that are restrained on four sides by beams or slabs of approximately equal depth, connections shall have lateral reinforcement not less than that required by Eq. 6.6.55 within the column for a depth not less than that of the deepest connection of framing elements to the columns. See also Sec. 8.1.13 Chapter 8.

6.4.10 Provisions for Footings and Slabs

6.4.10.1 The shear strength of footings and slabs in the vicinity of columns, concentrated loads, or reactions is governed by the more severe of the following two conditions:

6.4.10.1.1 Beam action where each critical section to be investigated extends in a plane across the entire width. The slab or footing shall be designed in accordance with Sections 6.4.1 to 6.4.3 for beam action.

6.4.10.1.2 For two-way action, each of the critical sections to be investigated shall be located so that its perimeter b_o is a minimum but need not approach closer than $\frac{d}{2}$ to:

(a) Edges or corners of columns, concentrated loads, or reaction areas; and

(b) Changes in slab thickness such as edges of capitals, drop panels, or shear caps.

For two-way action, the slab or footing shall be designed in accordance with Sections 6.4.10.2 to 6.4.10.6.

6.4.10.1.3 For square or rectangular columns, concentrated loads, or reaction areas, the critical sections with four straight sides shall be permitted.

6.4.10.2 For two-way action, the design of a slab or footing is based on Equations 6.6.47 and 6.6.48. V_c shall be computed in accordance with Sec 6.4.10.2.1, or Sec 6.4.10.3.1. V_s shall be computed in accordance with 6.4.10.3. For slabs with shearheads, V_n shall be in accordance with Sec 6.4.10.4. Where moment is transferred between a slab and a column, Sec 6.4.10.6 shall apply.

6.4.10.2.1 For slabs and footings, V_c shall be the smallest of the values given by Equations 6.6.72, 6.6.73 and 6.6.74:

$$V_c = 0.17(1 + \frac{2}{\beta})\lambda\sqrt{f_c'}\, b_o d \qquad (6.6.72)$$

Where, β is the ratio of long side to short side of the column, concentrated load or reaction area;

$$V_c = 0.083(\frac{\alpha_s d}{b_o} + 2)\lambda\sqrt{f_c'}\, b_o d \qquad (6.6.73)$$

Where, $\alpha_s$is 40 for interior columns, 30 for edge columns, 20 for corner columns; and

$$V_c = 0.33\lambda\sqrt{f_c'}\, b_o d \qquad (6.6.74)$$

6.4.10.3 Bars or wires and single- or multiple-leg stirrups as shear reinforcement shall be permitted in slabs and footings with d greater than or equal to 150 mm, but not less than 16 times the shear reinforcement bar diameter. Shear reinforcement shall be in accordance with Sections 6.4.10.3.1 to 6.4.10.3.4.

6.4.10.3.1 For computing V_n, Eq. 6.6.48 shall be used and V_c shall not be taken greater than $0.17\lambda\sqrt{f_c' bd}$ and V_s shall be calculated in accordance with Sec 6.4.3. In Eq. 6.6.56, A_v shall be taken as the cross-sectional area of all legs of reinforcement on one peripheral line that is geometrically similar to the perimeter of column section.

6.4.10.3.2 V_n shall not be taken greater than. $0.5\sqrt{f_c' bd}$

6.4.10.3.3 The distance from the column face to the first line of stirrup legs that surround the column shall not exceed $\frac{d}{2}$. The spacing between adjacent stirrups legs in the first line of shear reinforcement shall not exceed $2d$ measured in a direction parallel to the column face. The spacing between successive lines of shear reinforcement that surround the column shall not exceed $\frac{d}{2}$ measured in a direction perpendicular to the column face. In a slab-column connection for which the moment transfer is negligible, the shear reinforcement should be symmetrical about the centroid of the critical section Figure 6.6.14. Spacing limits defined above are also shown in Figure 6.6.14 for interior column and in Figure 6.6.15 for edge column. At edge columns or for interior connections where moment transfer is significant, closed stirrups are recommended in a pattern as symmetrical as possible.

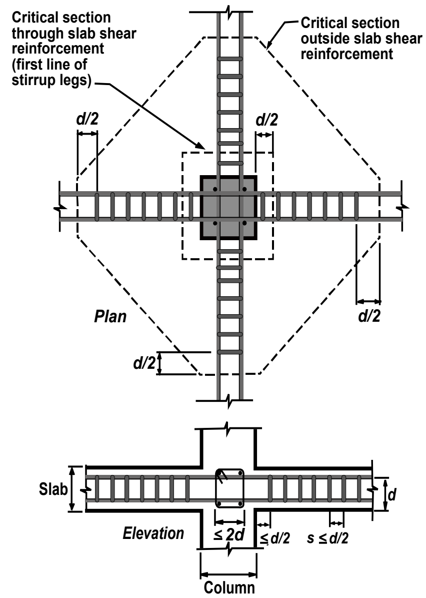


Figure 6.6.14 Arrangement of stirrup shear reinforcement around interior column

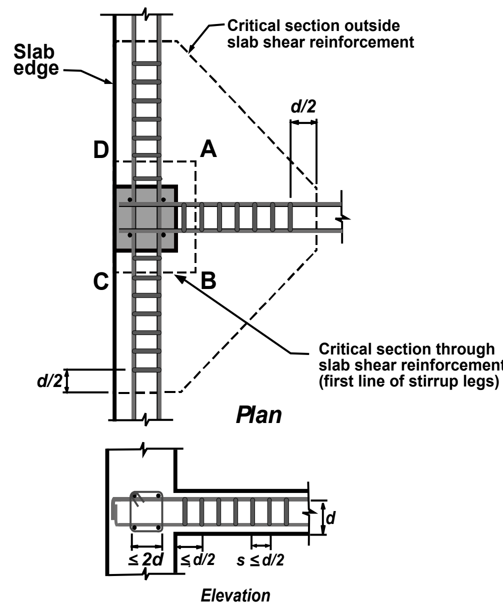


Figure 6.6.15 Arrangement of stirrup shear reinforcement around edge column

6.4.10.3.4 Slab shear reinforcement shall satisfy the anchorage requirements of Sec 8.2.10 Chapter 8 and shall engage the longitudinal flexural reinforcement in the direction being considered.

6.4.10.4 Shear reinforcement consisting of structural steel I- or channel-shaped sections (shearheads) shall be permitted in slabs. The provisions of Sections 6.4.10.4.1 to 6.4.10.4.9 shall apply where shear due to gravity load is transferred at interior column supports. Where moment is transferred to columns, Sec 6.4.10.7.3 shall apply.

6.4.10.4.1 Each shearhead shall consist of steel shapes fabricated by welding with a full penetration weld into identical arms at right angles. Shearhead arms shall not be interrupted within the column section.

6.4.10.4.2 A shearhead shall not be deeper than 70 times the web thickness of the steel shape.

6.4.10.4.3 The ends of each shearhead arm shall be permitted to be cut at angles not less than 30 degrees with the horizontal, provided the plastic moment strength of the remaining tapered section is adequate to resist the shear force attributed to that arm of the shearhead.

6.4.10.4.4 All compression flanges of steel shapes shall be located within $0.3d$ of compression surface of slab.

6.4.10.4.5 The ratio α_v between the flexural stiffness of each shearhead arm and that of the surrounding composite cracked slab section of width $(c_2 + d)$ shall not be less than 0.15.

6.4.10.4.6 Plastic moment strength, M_p, required for each arm of the shearhead shall be computed by

$$M_p = \frac{V_u}{2\phi n}\left[h_v + \alpha_v\left(l_v - \frac{c_1}{2}\right)\right] \quad (6.6.75)$$

Where, ϕ is for tension-controlled members, n is number of shearhead arms, and l_v is minimum length of each shearhead arm required to comply with requirements of Sections 6.4.10.4.7 and 6.4.10.4.8.

6.4.10.4.7 The critical slab section for shear shall be perpendicular to the plane of the slab and shall cross each shearhead arm at three-quarters the distance $\left[l_v - \left(\frac{c_1}{2}\right)\right]$ from the column face to the end of the shearhead arm. The critical section shall be located so that its perimeter b_o is a minimum, but need not be closer than the perimeter defined in Sec 6.4.10.1.2(a).

6.4.10.4.8 V_n shall not be taken larger than $0.33\sqrt{f'_c}b_o d$ on the critical section defined in Sec 6.4.10.4.7. When shearhead reinforcement is provided, V_n shall not be taken greater than $0.58\sqrt{f'_c}b_o d$ on the critical section defined in Sec 6.4.10.1.2(a).

6.4.10.4.9 Moment resistance M_v contributed to each slab column strip by a shearhead shall not be taken greater than

$$M_v = \frac{\phi \alpha_v V_u}{2n}\left(l_v - \frac{c_1}{2}\right) \quad (6.6.76)$$

Where, ϕ is for tension-controlled members, n is number of shearhead arms, and l_v is length of each shearhead arm actually provided. However, M_v shall not be taken larger than the smallest of:

(a) 30 percent of the total factored moment required for each slab column strip;

(b) The change in column strip moment over the length l_v;

(c) M_p computed by Eq. 6.6.75.

6.4.10.4.10 When unbalanced moments are considered, the shearhead must have adequate anchorage to transmit M_p to the column.

6.4.10.5 Headed shear stud reinforcement, placed perpendicular to the plane of a slab or footing, shall be permitted in slabs and footings in accordance with 6.4.10.5.1 through 6.4.10.5.4. The overall height of the shear stud assembly shall not be less than the thickness of the member less the sum of: (1) the concrete cover on the top flexural reinforcement; (2) the concrete cover on the base rail; and (3) one-half the bar diameter of the tension flexural reinforcement. Where flexural tension reinforcement is at the bottom of the section, as in a footing, the overall height of the shear stud assembly shall not be less than the thickness of the member less the sum of: (1) the concrete cover on the bottom flexural reinforcement; (2) the concrete cover on the head of the stud; and (3) one-half the bar diameter of the bottom flexural reinforcement.

6.4.10.5.1 For the critical section defined in Sec 6.4.10.1.2, V_n shall be computed using Eq. 6.6.48, with V_c and V_n not exceeding $0.25\lambda\sqrt{f_c' b_o d}$ and $0.66\lambda\sqrt{f_c' b_o d}$ respectively. V_s shall be calculated using Eq. 6.6.56 with A_v equal to the cross-sectional area of all the shear reinforcement on one peripheral line that is approximately parallel to the perimeter of the column section, where ***s*** is the spacing of the peripheral lines of headed shear stud reinforcement. $\dfrac{A_v f_{yt}}{b_o s}$ shall not be less than $0.17\sqrt{f_c'}$

6.4.10.5.2 The spacing between the column face and the first peripheral line of shear reinforcement shall not exceed $\frac{d}{2}$. The spacing between peripheral lines of shear reinforcement, measured in a direction perpendicular to any face of the column, shall be constant. For all slabs and footings, the spacing shall be based on the value of the shear stress due to factored shear force and unbalanced moment at the critical section defined in Sec 6.4.10.1.2, and shall not exceed:

(a) $0.75d$, where maximum shear stresses due to factored loads are less than or equal to $0.5\phi\sqrt{f_c'}$; and

(b) $0.5d$, where maximum shear stresses due to factored loads are greater than $0.5\phi\sqrt{f_c'}$.

6.4.10.5.3 The spacing between adjacent shear reinforcement elements, measured on the perimeter of the first peripheral line of shear reinforcement, shall not exceed $2d$.

6.4.10.5.4 Shear stress due to factored shear force and moment shall not exceed $0.17\phi\lambda\sqrt{f_c'}$ at the critical section located $\frac{d}{2}$ outside the outermost peripheral line of shear reinforcement.

6.4.10.6 Openings in slabs

If openings in slabs are located at a distance less than 10 times the slab thickness from a concentrated load or reaction area, or when openings in flat slabs are located within column strips as defined in Sec. 6.5, the critical slab sections for shear defined in Sections 6.4.10.1.2 and 6.4.10.4.7 shall be modified as follows:

6.4.10.6.1 For slabs without shearheads, that part of the perimeter of the critical section that is enclosed by straight lines projecting from the centroid of the column, concentrated load, or reaction area and tangent to the boundaries of the openings shall be considered ineffective Figure 6.6.16.

6.4.10.6.2 For slabs with shearheads, the ineffective portion of the perimeter shall be one-half of that defined in Sec 6.4.10.6.1.

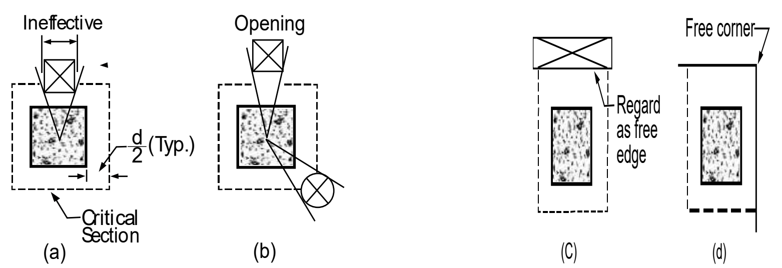


Figure 6.6.16 Effective perimeter (in dashed lines) to consider effect of openings and free edges

6.4.10.7 Transfer of moment in slab-column connections

6.4.10.7.1 Where gravity load, wind, earthquake, or other lateral forces cause transfer of unbalanced moment M_u between a slab and column, $\gamma_f M_u$ shall be transferred by flexure in accordance with Sec 6.5.5.3. The remainder of the unbalanced moment, $\gamma_V M_u$, shall be considered to be transferred by eccentricity of shear about the centroid of the critical section defined in Sec 6.4.10.1.2 where

$$\gamma_v = (1 - \gamma_f) \qquad (6.6.77)$$

6.4.10.7.2 The shear stress resulting from moment transfer by eccentricity of shear shall be assumed to vary linearly about the centroid of the critical sections defined in Sec 6.4.10.1.2. The maximum shear stress due to V_u and M_u shall not exceed ϕv_n :

(a) For members without shear reinforcement,

$$\phi V_n = \phi V_c/(b_o d) \tag{6.6.78}$$

Where, V_c is as defined in Sec 6.4.10.2.1.

(b) For members with shear reinforcement other than shearheads,

$$\phi V_n = \phi (V_c + V_s)/(b_o d) \tag{6.6.79}$$

Where, V_c and V_s are defined in Sec 6.4.10.3.1. The design shall take into account the variation of shear stress around the column. The shear stress due to factored shear force and moment shall not exceed $\left(0.17\phi\lambda\sqrt{f_c'}\right)$at the critical section located $\frac{d}{2}$ outside the outermost line of stirrup legs that surround the column.

The maximum factored shear stress may be obtained from the combined shear stresses on the left and right faces of the column (Figure 6.6.17) as given by the following Equations:

$$v_l = \frac{V_u}{A_c} - \frac{\gamma_v M_u c_l}{J_c} \tag{6.6.80a}$$

$$v_r = \frac{V_u}{A_c} + \frac{\gamma_v M_u c_r}{J_c} \tag{6.6.80b}$$

Where, A_c = area of concrete of assumed critical section = $2d(c_1 + c_2 + 2d)$

c_l, c_r = distances from centroid of critical section to left and right face of section respectively

c_1, c_2 = width and depth of the column

J_c = property of assumed critical section analogous to polar moment of inertia. For an interior column, the quantity J_c is

$$J_c = \frac{2d(c_1 + d)^3}{12} + \frac{2(c_1 + d)d^3}{12} + 2d(c_2 + d)\left(\frac{c_1 + d}{2}\right)^2 \tag{6.6.80c}$$

6.4.10.7.3 When shear reinforcement consisting of structural steel I- or channel-shaped sections (shearheads) is provided, the sum of the shear stresses due to vertical load acting on the critical section defined by Sec 6.4.10.4.7 and the shear stresses resulting from moment transferred by eccentricity of shear about the centroid of the critical section defined in Sec 6.4.10.1.2(a) and 6.4.10.1.3 shall not exceed . $0.33\phi\lambda\sqrt{f_c'}$.

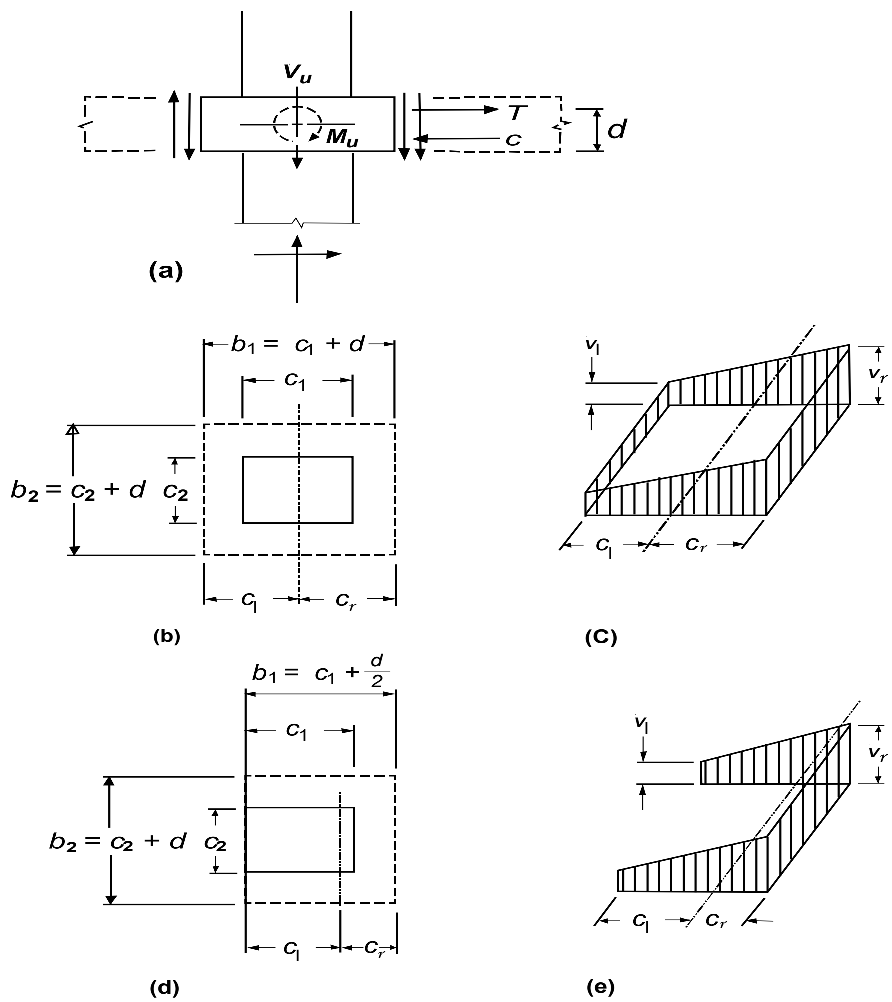


Figure 6.6.17 Transfer of moment from slab to column: (a) forces resulting from vertical load and unbalanced moment; (b) critical section for an interior column; (c) shear stress distribution for an interior column; (d) critical section for an edge column; (e) shear stress distribution for an edge column

6.5 Two-Way Slab Systems: Flat Plates, Flat Slabs and Edge-Supported Slabs

6.5.1 Scope

The provisions of this section shall apply to all slabs, solid, ribbed or hollow, spanning in more than one direction, with or without beams between the supports. Flat plate is a term normally attributed to slabs without beams and without drop panels, column capitals, or brackets. On the other hand, slabs without beams, but with drop panels, column capital or brackets are commonly known as flat slabs. While this section covers the requirements for all types of slabs, the provisions of Sec 6.5.8. Alternative Design of Two-way Edge-Supported slabs, may be used as an alternative for slabs supported on all four edges by walls, steel beams or monolithic concrete beams having a total depth not less than 3 times the slab thickness.

6.5.1.1 For a slab system supported by columns or walls, dimensions c_1, c_2, and l_n shall be based on an effective support area defined by the intersection of the bottom surface of the slab, or of the drop panel or shear cap if present, with the largest right circular cone, right pyramid, or tapered wedge whose surfaces are located within the column and the capital or bracket and are oriented no greater than 45^o to the axis of the column.

6.5.1.2 Minimum thickness of slabs designed in accordance with Sec. 6.5 shall be as required by Sec 6.2.5.3.

6.5.2 General

6.5.2.1 Column strip is a design strip with a width on each side of a column centerline equal to $0.25l_2$ or $0.25l_1$, whichever is less. Column strip includes beams, if any.

6.5.2.2 Middle strip is a design strip bounded by two column strips.

6.5.2.3 A panel is bounded by column, beam, or wall centerlines on all sides.

6.5.2.4 For monolithic or fully composite construction, a beam includes that portion of slab on each side of the beam extending a distance equal to the projection of the beam above or below the slab, whichever is greater, but not greater than four times the slab thickness (Figure 6.6.18).

6.5.2.5 When used to reduce the amount of negative moment reinforcement over a column or minimum required slab thickness, a drop panel shall:

(a) project below the slab at least one-quarter of the adjacent slab thickness; and

(b) extend in each direction from the centerline of support a distance not less than one-sixth the span length measured from center-to-center of supports in that direction.

When used to increase the critical condition section for shear at a slab-column joint, a shear cap shall project below the slab and extend a minimum horizontal distance from the face of the column that is equal to the thickness of the projection below the slab soffit.

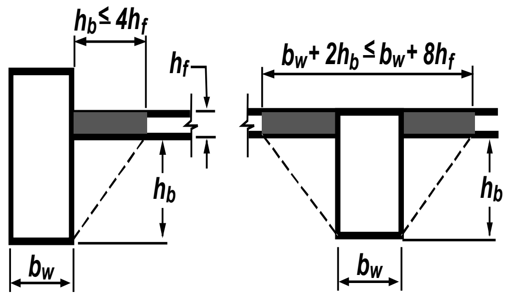


Figure 6.6.18 Portion of slab to be included with the beam

6.5.3 Slab Reinforcement

6.5.3.1 Area of reinforcement in each direction for two-way slab systems shall be determined from moments at critical sections, but shall not be less than required by Sec. 8.1.11.2 Chapter 8.

6.5.3.2 Spacing of reinforcement at critical sections shall not exceed two times the slab thickness, except for portions of slab area of cellular or ribbed construction. In the slab over cellular spaces, reinforcement shall be provided as required by Sec. 8.1.11 Chapter 8.

6.5.3.3 Positive moment reinforcement perpendicular to a discontinuous edge shall extend to the edge of slab and have embedment, straight or hooked, at least 150 mm in spandrel beams, columns, or walls.

6.5.3.4 Negative moment reinforcement perpendicular to a discontinuous edge shall be bent, hooked, or otherwise anchored in spandrel beams, columns, or walls, and shall be developed at face of support according to provisions of Sec. 8.2 Chapter 8.

6.5.3.5 Where a slab is not supported by a spandrel beam or wall at a discontinuous edge, or where a slab cantilevers beyond the support, anchorage of reinforcement shall be permitted within the slab.

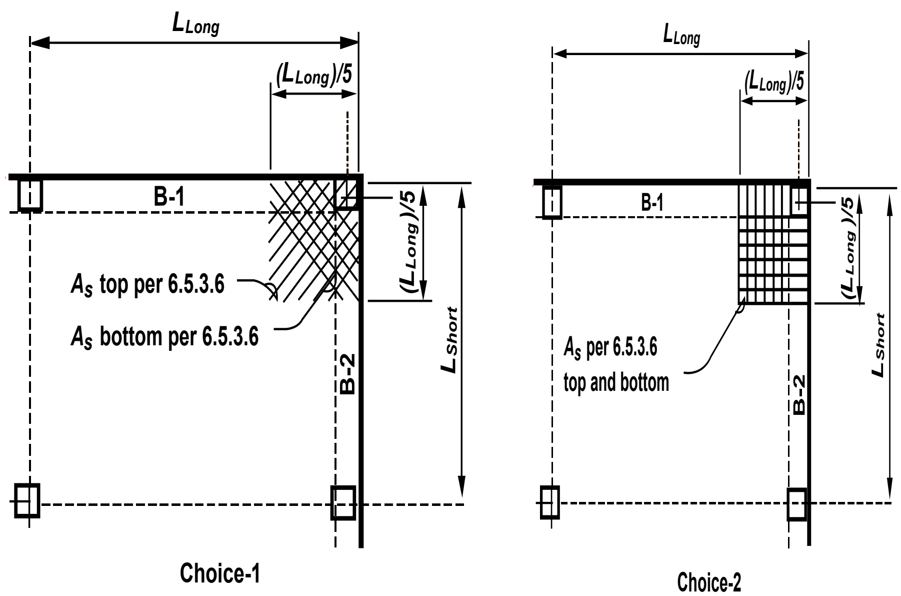
6.5.3.6 At exterior corners of slabs supported by edge walls or where one or more edge beams have a value of α_f greater than 1.0, top and bottom slab reinforcement shall be provided at exterior corners in accordance with Sections 6.5.3.6.1 to 6.5.3.6.4 and as shown in Figure 6.6.19.

6.5.3.6.1 Corner reinforcement in both top and bottom of slab shall be sufficient to resist a moment per unit of width equal to the maximum positive moment per unit width in the slab panel.

6.5.3.6.2 The moment shall be assumed to be about an axis perpendicular to the diagonal from the corner in the top of the slab and about an axis parallel to the diagonal from the corner in the bottom of the slab.

6.5.3.6.3 Corner reinforcement shall be provided for a distance in each direction from the corner equal to one-fifth the longer span.

6.5.3.6.4 Corner reinforcement shall be placed parallel to the diagonal in the top of the slab and perpendicular to the diagonal in the bottom of the slab. Alternatively, reinforcement shall be placed in two layers parallel to the sides of the slab in both the top and bottom of the slab.



Notes:

1. Applies if B-1 or B-2 has αf > 1.0
2. Maximum bar spacing 2h, where h = slab thickness
3. Reinforcement same as maximum +ve reinforcement of the panel

Figure 6.6.19 Corner reinforcement in slabs

6.5.3.7 When a drop panel is used to reduce the amount of negative moment reinforcement over the column of a flat slab, the dimensions of the drop panel shall be in accordance with Sec 6.5.2.5. In computing required slab reinforcement, the thickness of the drop panel below the slab shall not be assumed to be greater than one-quarter the distance from the edge of drop panel to the face of column or column capital.

6.5.3.8 Details of reinforcement in slabs without beams

6.5.3.8.1 In addition to the other requirements of Sec 6.5.3, reinforcement in slabs without beams shall have minimum extensions as prescribed in Figure 6.6.20.

6.5.3.8.2 Where adjacent spans are unequal, extensions of negative moment reinforcement beyond the face of support as prescribed in Figure 6.6.20 shall be based on requirements of the longer span.

6.5.3.8.3 Bent bars shall be permitted only when depth-span ratio permits use of bends of 45 degrees or less.

6.5.3.8.4 In frames where two-way slabs act as primary members resisting lateral loads, lengths of reinforcement shall be determined by analysis but shall not be less than those prescribed in Figure 6.6.20.

6.5.3.8.5 All bottom bars or wires within the column strip, in each direction, shall be continuous or spliced with Class B tension splices or with mechanical or welded splices satisfying Sec. 8.2.12.3 Chapter 8. Splices shall be located as shown in Figure 6.6.20. At least two of the column strip bottom bars or wires in each direction shall pass within the region bounded by the longitudinal reinforcement of the column and shall be anchored at exterior supports.

6.5.3.8.6 In slabs with shearheads and in lift-slab construction where it is not practical to pass the bottom bars required by 6.5.3.8.5 through the column, at least two bonded bottom bars or wires in each direction shall pass through the shearhead or lifting collar as close to the column as practicable and be continuous or spliced with a Class A splice. At exterior columns, the reinforcement shall be anchored at the shearhead or lifting collar.

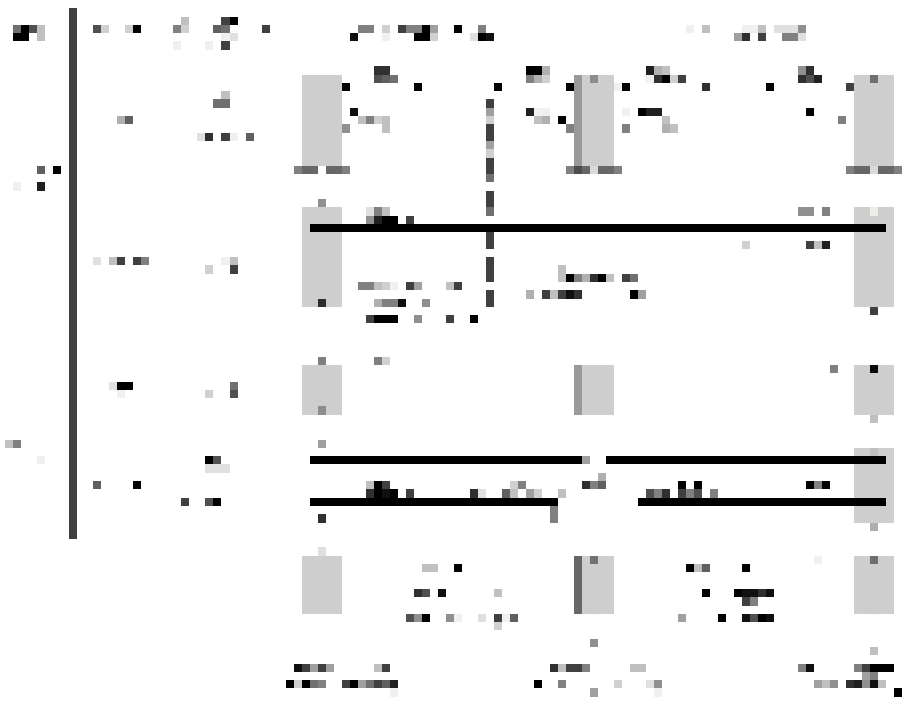


Figure 6.6.20 Minimum extensions for reinforcement in slabs without beams for reinforcement extension into supports

6.5.4 Openings in Slab Systems

6.5.4.1 Openings of any size shall be permitted in slab systems if shown by analysis that the design strength is at least equal to the required strength set forth in Sections 6.2.2 and 6.2.3, and that all serviceability conditions, including the limits on deflections, are met.

6.5.4.2 As an alternate to analysis required by Sec 6.5.4.1, openings shall be permitted in slab systems without beams only, in accordance with Sections 6.5.4.2.1 to 6.5.4.2.4.

6.5.4.2.1 Openings of any size shall be permitted in the area common to intersecting middle strips, provided total amount of reinforcement required for the panel without the opening is maintained.

6.5.4.2.2 In the area common to intersecting column strips, not more than one-eighth the width of column strip in either span shall be interrupted by openings. An amount of reinforcement equivalent to that interrupted by an opening shall be added on the sides of the opening.

6.5.4.2.3 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 openings. An amount of reinforcement equivalent to that interrupted by an opening shall be added on the sides of the opening.

6.5.4.2.4 Shear requirements of Sec 6.4.10.6 shall be satisfied.

6.5.5 Design Procedures

6.5.5.1 A slab system shall be designed by any procedure satisfying conditions of equilibrium and geometric compatibility, if shown that the design strength at every section is at least equal to the required strength set forth in Sections 6.2.2 and 6.2.3, and that all serviceability conditions, including limits on deflections, are met.

6.5.5.1.1 Design of a slab system for gravity loads, including the slab and beams (if any) between supports and supporting columns or walls forming orthogonal frames, by either the Direct Design Method of Sec 6.5.6 or the Equivalent Frame Method of Sec 6.5.7, shall be permitted.

6.5.5.1.2 For lateral loads, analysis of frames shall take into account effects of cracking and reinforcement on stiffness of frame members.

6.5.5.1.3 Combining the results of the gravity load analysis with the results of the lateral load analysis shall be permitted.

6.5.5.2 The slab and beams (if any) between supports shall be proportioned for factored moments prevailing at every section.

6.5.5.3 When gravity load, wind, earthquake, or other lateral forces cause transfer of moment between slab and column, a fraction of the unbalanced moment shall be transferred by flexure in accordance with Sections 6.5.5.3.2 to 6.5.5.3.4.

6.5.5.3.1 The fraction of unbalanced moment not transferred by flexure shall be transferred by eccentricity of shear in accordance with Sec 6.4.10.7.

6.5.5.3.2 A fraction of the unbalanced moment given by $\gamma_f M_u$ shall be considered to be transferred by flexure within an effective slab width between lines that are one and one-half slab or drop panel thickness $(1.5h)$ outside opposite faces of the column or capital, where M_u is the factored moment to be transferred and

$$\gamma_f = \frac{1}{1+(2/3)\sqrt{b_1/b_2}} \tag{6.6.81}$$

6.5.5.3.3 For slabs with unbalanced moments transferred between the slab and columns, it shall be permitted to increase the value of γ_f given by Eq. 6.6.81 in accordance with the following:

(a) For edge columns with unbalanced moments about an axis parallel to the edge, $\gamma_f = 1.0$ provided that V_u at an edge support does not exceed $0.75\phi V_c$, or at a corner support does not exceed $0.5\phi V_c$.

(b) For unbalanced moments at interior supports, and for edge columns with unbalanced moments about an axis perpendicular to the edge, increase γ_f to as much as 1.25 times the value from Eq. 6.6.81, but not more than $\gamma_f = 1.0$, provided that V_u at the support does not exceed $0.4\phi V_c$. The net tensile strain ε_t calculated for the effective slab width defined in Sec 6.5.5.3.2 shall not be less than 0.010.

The value of V_c in items (a) and (b) shall be calculated in accordance with Sec 6.4.10.2.1.

6.5.5.3.4 Concentration of reinforcement over the column by closer spacing or additional reinforcement shall be used to resist moment on the effective slab width defined in Sec 6.5.5.3.2.

6.5.5.4 Design for transfer of load from slabs to supporting columns or walls through shear and torsion shall be in accordance with Sec. 6.4.

6.5.6 Direct Design Method

6.5.6.1 Limitations

Design of slab systems within the limitations of Sections 6.5.6.1.1 to 6.5.6.1.8 by the direct design method shall be permitted.

6.5.6.1.1 There shall be a minimum of three continuous spans in each direction.

6.5.6.1.2 Panels shall be rectangular, with a ratio of longer to shorter span center-to-center of supports within a panel not greater than 2.

6.5.6.1.3 Successive span lengths center-to-center of supports in each direction shall not differ by more than one-third the longer span.

6.5.6.1.4 Offset of columns by a maximum of 10 percent of the span (in direction of offset) from either axis between centerlines of successive columns shall be permitted.

6.5.6.1.5 All loads shall be due to gravity only and uniformly distributed over an entire panel. The unfactored live load shall not exceed two times the unfactored dead load.

6.5.6.1.6 For a panel with beams between supports on all sides, Eq. 6.6.82 shall be satisfied for beams in the two perpendicular directions.

$$0.2 \leq \frac{\alpha_{f1} {l_2}^2}{\alpha_{f2} {l_1}^2} \leq 5.0 \qquad (6.6.82)$$

Where, α_{f1} and α_{f2} are calculated using respective stiffness parameters in accordance with the general Equation 6.6.83.

$$\alpha_f = \frac{E_{cb} I_b}{E_{cs} I_s} \qquad (6.6.83)$$

6.5.6.1.7 Moment redistribution as permitted by Sec 6.1.6 shall not be applied for slab systems designed by the direct design method. See Sec 6.5.6.7.

6.5.6.1.8 Variations from the limitations of Sec 6.5.6.1 shall be permitted if demonstrated by analysis that requirements of Sec 6.5.5.1 are satisfied.

6.5.6.2 Total factored static moment for a span

6.5.6.2.1 Total factored static moment, M_o, for a span shall be determined in a strip bounded laterally by centerline of panel on each side of centerline of supports.

6.5.6.2.2 Absolute sum of positive and average negative factored moments in each direction shall not be less than

$$M_o = \frac{q_u l_2 l_n^2}{8} \qquad (6.6.84)$$

Where, l_n is length of clear span in direction that moments are being determined.

6.5.6.2.3 Where the transverse span of panels on either side of the centerline of supports varies, l_2 in Eq. 6.6.84 shall be taken as the average of adjacent transverse spans.

6.5.6.2.4 When the span adjacent and parallel to an edge is being considered, the distance from edge to panel centerline shall be substituted for l_2 in Eq. 6.6.84.

6.5.6.2.5 Clear span l_n shall extend from face to face of columns, capitals, brackets, or walls. Value of l_n used in Eq. 6.6.84 shall not be less than $0.65l_1$. Circular or regular polygon-shaped supports shall be treated as square supports with the same area.

6.5.6.3 Negative and positive factored moments

6.5.6.3.1 Negative factored moments shall be located at face of rectangular supports. Circular or regular polygon-shaped supports shall be treated as square supports with the same area.

6.5.6.3.2 In an interior span, total static moment, M_o, shall be distributed as follows:

Negative factored moment: 0.65

Positive factored moment: 0.35

6.5.6.3.3 In an end span, total factored static moment, M_o, shall be distributed as in Table 6.6.4 below:

Table 6.6.4: Distribution of Total Factored Static Moment, M_o in an End Span

Moments	Exterior edge unrestrained	Slab with beams between all supports	Slab without beams between interior supports		Exterior edge fully restrained
			Without edge beam	With edge beam	
Interior negative factored moment	0.75	0.70	0.70	0.70	0.65
Positive factored moment	0.63	0.57	0.52	0.50	0.35
Exterior negative factored moment	0	0.16	0.26	0.30	0.65

6.5.6.3.4 Negative moment sections shall be designed to resist the larger of the two interior negative factored moments determined for spans framing into a common support unless an analysis is made to distribute the unbalanced moment in accordance with stiffnesses of adjoining elements.

6.5.6.3.5 Edge beams or edges of slab shall be proportioned to resist in torsion their share of exterior negative factored moments.

6.5.6.3.6 The gravity load moment to be transferred between slab and edge column in accordance with 6.5.5.3.1 shall be $0.3M_o$.

6.5.6.4 Factored moments in column strips

6.5.6.4.1 Column strips shall be proportioned to resist the portions in percent of interior negative factored moments as shown in Table 6.6.5.

6.5.6.4.2 Column strips shall be proportioned to resist the portions in percent of exterior negative factored moments as shown in Table 6.6.6.

Table 6.6.5: Portions of Interior Negative Moments to be resisted by Column Strip

Parameters	l_2 / l_1		
	0.5	1.0	2.0
$\left(\frac{\alpha_{f1} l_2}{l_1}\right) = 0$	75	75	75
$\left(\frac{\alpha_{f1} l_2}{l_1}\right) \geq 1$	90	75	45

Notes: Linear interpolations shall be made between values shown.
Interpolation function for % of Moment $= 75 + 30\left(\frac{\alpha_{f1} l_2}{l_1}\right)\left(1 - \frac{l_2}{l_1}\right)$

Table 6.6.6: Portions of Exterior Negative Moments to be resisted by Column Strip

Parameters		l_2 / l_1		
		0.5	1.0	2.0
$\left(\frac{\alpha_{f1} l_2}{l_1}\right) = 0$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	75	75	75
$\left(\frac{\alpha_{f1} l_2}{l_1}\right) \geq 1$	$\beta_t = 0$	100	100	100
	$\beta_t \geq 2.5$	90	75	45

Linear interpolations shall be made between values shown, where β_t is calculated in Eq. 6.6.85 and C is calculated in Eq. 6.6.86.

$$\beta_t = \frac{E_{cb} C}{2 E_{cs} I_s} \qquad (6.6.85)$$

$$C = \sum \left(1 - 0.63 \frac{x}{y}\right) \frac{x^3 y}{3} \qquad (6.6.86)$$

The constant C for T or L sections shall be permitted to be evaluated by dividing the section into separate rectangular parts, as defined in Sec 6.5.2.4, and summing the values of C for each part.

Interpolation function for % of Moment = $100 - 10\beta_t + 12\beta_t\left(\frac{\alpha_{f1}l_2}{l_1}\right)\left(1 - \frac{l_2}{l_1}\right)$

6.5.6.4.3 Where supports consist of columns or walls extending for a distance equal to or greater than $0.75l_2$ used to compute M_o, negative moments shall be considered to be uniformly distributed across l_2.

6.5.6.4.4 Column strips shall be proportioned to resist the portions in percent of positive factored moments shown in Table 6.6.7.

6.5.6.4.5 For slabs with beams between supports, the slab portion of column strips shall be proportioned to resist that portion of column strip moments not resisted by beams.

Table 6.6.7: Portions of Positive Moment to be resisted by Column Strip

Parameters	l_2/l_1		
	0.5	1.0	2.0
$\left(\frac{\alpha_{f1}l_2}{l_1}\right) = 0$	60	60	60
$\left(\frac{\alpha_{f1}l_2}{l_1}\right) \geq 1$	90	75	45

Notes: Linear interpolations shall be made between values shown.

Interpolation function for % of Moment $= 60 + 30\left(\frac{\alpha_{f1}l_2}{l_1}\right)\left(1.5 - \frac{l_2}{l_1}\right)$

6.5.6.5 Factored moments in beams

6.5.6.5.1 Beams between supports shall be proportioned to resist 85 percent of column strip moments if $\alpha_{f1}l_2/l_1$ is equal to or greater than 1.0.

6.5.6.5.2 For values of $\alpha_{f1}l_2/l_1$ between 1.0 and zero, proportion of column strip moments resisted by beams shall be obtained by linear interpolation between 85 and zero percent.

6.5.6.5.3 In addition to moments calculated for uniform loads according to Sections 6.5.6.2.2, 6.5.6.5.1, and 6.5.6.5.2, beams shall be proportioned to resist all moments caused by concentrated or linear loads applied directly to beams, including weight of projecting beam stem above or below the slab.

6.5.6.6 Factored moments in middle strips

6.5.6.6.1 That portion of negative and positive factored moments not resisted by column strips shall be proportionately assigned to corresponding half middle strips.

6.5.6.6.2 Each middle strip shall be proportioned to resist the sum of the moments assigned to its two half middle strips.

6.5.6.6.3 A middle strip adjacent to and parallel with a wall-supported edge shall be proportioned to resist twice the moment assigned to the half middle strip corresponding to the first row of interior supports.

6.5.6.7 Modification of factored moments

Modification of negative and positive factored moments by 10 percent shall be permitted provided the total static moment for a panel, M_o, in the direction considered is not less than that required by Eq. 6.6.84.

6.5.6.8 Factored shear in slab systems with beams

6.5.6.8.1 Beams with $\alpha_{f1} l_2/l_1$ equal to or greater than 1.0 shall be proportioned to resist shear caused by factored loads on tributary areas which are bounded by 45° lines drawn from the corners of the panels and the centerlines of the adjacent panels parallel to the long sides (Figure 6.6.21).

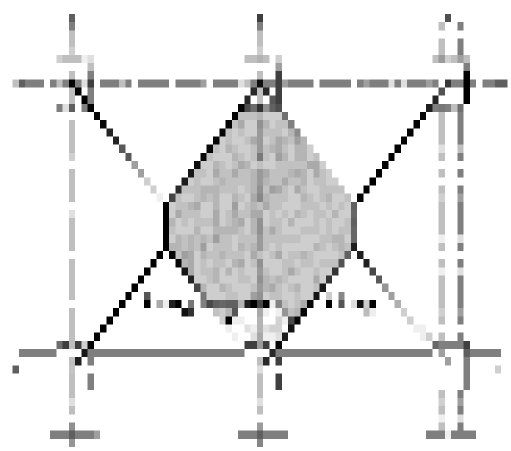


Figure 6.6.21 Tributary area for shear on an interior beam

6.5.6.8.2 In proportioning beams with $\alpha_{f1} l_2/l_1$ less than 1.0 to resist shear, linear interpolation, assuming beams carry no load at $\alpha_{f1} = 0$, shall be permitted.

6.5.6.8.3 In addition to shears calculated according to Sections 6.5.6.8.1 and 6.5.6.8.2, beams shall be proportioned to resist shears caused by factored loads applied directly on beams.

6.5.6.8.4 Computation of slab shear strength on the assumption that load is distributed to supporting beams in accordance with Sec 6.5.6.8.1 or Sec 6.5.6.8.2 shall be permitted. Resistance to total shear occurring on a panel shall be provided.

6.5.6.8.5 Shear strength shall satisfy the requirements of Sec. 6.4.

6.5.6.9 Factored moments in columns and walls

6.5.6.9.1 Columns and walls built integrally with a slab system shall resist moments caused by factored loads on the slab system.

6.5.6.9.2 At an interior support, supporting elements above and below the slab shall resist the factored moment specified by Eq. 6.6.87 in direct proportion to their stiffnesses unless a general analysis is made.

$$M_u = 0.07\left[(q_{Du} + 0.5q_{Lu})l_2 l_n^2 - q'_{Du} l'_2 (l'_n)^2\right] \qquad (6.6.87)$$

Where, q'_{Du}, l'_2, and l'_n refer to shorter span.

6.5.7 Equivalent Frame Method

6.5.7.1 Design of slab systems by the equivalent frame method shall be based on assumptions given in Sections 6.5.7.2 to 6.5.7.6, and all sections of slabs and supporting members shall be proportioned for moments and shears thus obtained.

6.5.7.1.1 Where metal column capitals are used, it shall be permitted to take account of their contributions to stiffness and resistance to moment and to shear.

6.5.7.1.2 It shall be permitted to neglect the change in length of columns and slabs due to direct stress, and deflections due to shear.

6.5.7.2 Equivalent frame

6.5.7.2.1 The structure shall be considered to be made up of equivalent frames on column lines taken longitudinally and transversely through the building (Figure 6.6.22).

6.5.7.2.2 Each frame shall consist of a row of columns or supports and slab-beam strips, bounded laterally by the centerline of panel on each side of the center line of columns or supports.

6.5.7.2.3 Columns or supports shall be assumed to be attached to slab-beam strips by torsional members (see Sec 6.5.7.5) transverse to the direction of the span for which moments are being determined and extending to bounding lateral panel centerlines on each side of a column.

6.5.7.2.4 Frames adjacent and parallel to an edge shall be bounded by that edge and the centerline of adjacent panel.

6.5.7.2.5 Analysis of each equivalent frame in its entirety shall be permitted. Alternatively, for gravity loading, a separate analysis of each floor or roof with far ends of columns considered fixed shall be permitted.

6.5.7.2.6 Where slab-beams are analyzed separately, determination of moment at a given support assuming that the slab-beam is fixed at any support two panels distant therefrom, shall be permitted, provided the slab continues beyond that point.

6.5.7.3 Slab-beams

6.5.7.3.1 Determination of the moment of inertia of slab-beams at any cross section outside of joints or column capitals using the gross area of concrete shall be permitted.

6.5.7.3.2 Variation in moment of inertia along axis of slab-beams shall be taken into account.

6.5.7.3.3 Moment of inertia of slab-beams from center of column to face of column, bracket, or capital shall be assumed equal to the moment of inertia of the slab-beam at face of column, bracket, or capital divided by the quantity $(1-c_2/l_2)^2$ where c_2 and l_2 are measured transverse to the direction of the span for which moments are being determined.

6.5.7.4 Columns

6.5.7.4.1 Determination of the moment of inertia of columns at any cross section outside of joints or column capitals using the gross area of concrete shall be permitted.

6.5.7.4.2 Variation in moment of inertia along axis of columns shall be taken into account (Figure 6.6.23).

6.5.7.4.3 Moment of inertia of columns from top to bottom of the slab-beam at a joint shall be assumed to be infinite.

6.5.7.5 Torsional members

6.5.7.5.1 Torsional members (see Sec 6.5.7.2.3) shall be assumed to have a constant cross section throughout their length consisting of the largest of (a), (b), and (c):

(a) A portion of slab having a width equal to that of the column, bracket, or capital in the direction of the span for which moments are being determined;

(b) For monolithic or fully composite construction, the portion of slab specified in (a) plus that part of the transverse beam above and below the slab;

(c) The transverse beam as defined in Sec 6.5.2.4.

6.5.7.5.2 Where beams frame into columns in the direction of the span for which moments are being determined, the torsional stiffness shall be multiplied by the ratio of the moment of inertia of the slab with such a beam to the moment of inertia of the slab without such a beam.

6.5.7.5.3 Stiffness K_t of the torsional members shall be calculated by the following expression:

$$K_t = \sum \frac{9E_{cs}C}{l_2(1-C_2/l_2)^3} \qquad (6.6.88)$$

Where, c_2 and l_2 relate to the transverse span on each side of column.

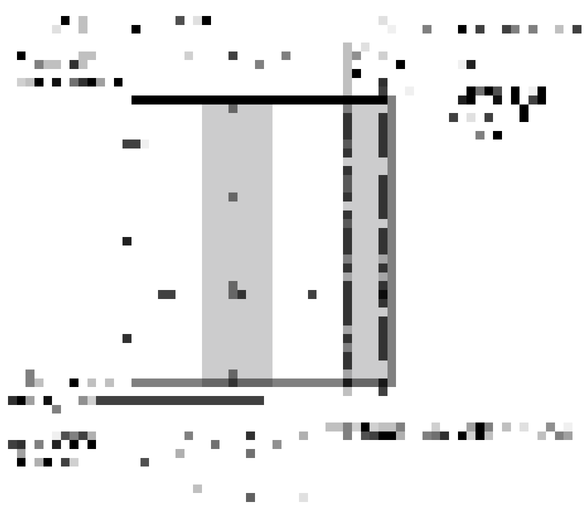


Figure 6.6.22 Definitions of equivalent frame.

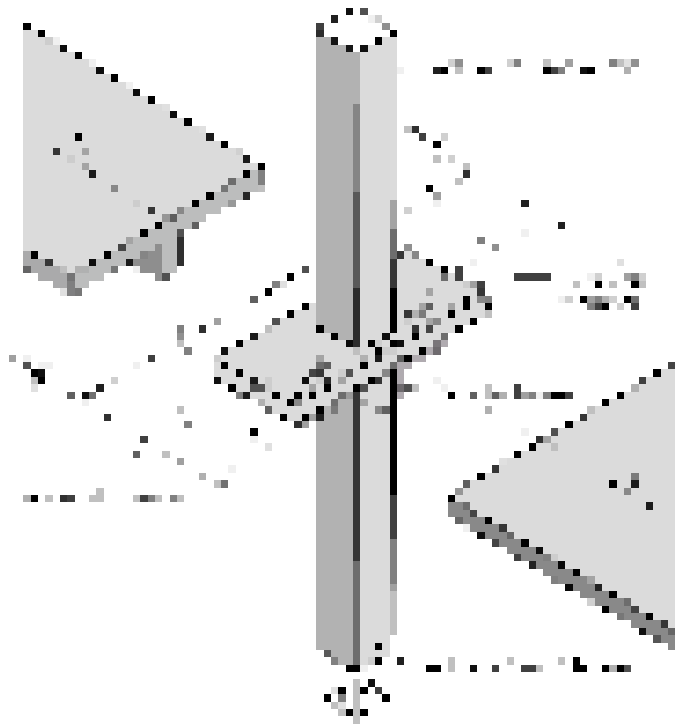


Figure 6.6.23 Equivalent column (column plus torsional members).

6.5.7.6 Arrangement of live load

6.5.7.6.1 When the loading pattern is known, the equivalent frame shall be analyzed for that load.

6.5.7.6.2 When the unfactored live load is variable but does not exceed three-quarters of the unfactored dead load, or the nature of live load is such that all panels will be loaded simultaneously, it shall be permitted to assume that maximum factored moments occur at all sections with full factored live load on entire slab system.

6.5.7.6.3 For loading conditions other than those defined in Sec 6.5.7.6.2, it shall be permitted to assume that maximum positive factored moment near mid span of a panel occurs with three-quarters of the full factored live load on the panel and on alternate panels; and it shall be permitted to assume that maximum negative factored moment in the slab at a support occurs with three-quarters of the full factored live load on adjacent panels only.

6.5.7.6.4 Factored moments shall be taken not less than those occurring with full factored live load on all panels.

6.5.7.7 Factored moments

6.5.7.7.1 At interior supports, the critical section for negative factored moment (in both column and middle strips) shall be taken at face of rectilinear supports, but not farther away than $0.175\, l_1$ from the center of a column.

6.5.7.7.2 At exterior supports with brackets or capitals, the critical section for negative factored moment in the span perpendicular to an edge shall be taken at a distance from face of supporting element not greater than one-half the projection of bracket or capital beyond face of supporting element.

6.5.7.7.3 Circular or regular polygon-shaped supports shall be treated as square supports with the same area for location of critical section for negative design moment.

6.5.7.7.4 Where slab systems within limitations of Sec 6.5.6.1 are analyzed by the equivalent frame method, it shall be permitted to reduce the resulting computed moments in such proportion that the absolute sum of the positive and average negative moments used in design need not exceed the value obtained from Eq. 6.6.84.

6.5.7.7.5 Distribution of moments at critical sections across the slab-beam strip of each frame to column strips, beams, and middle strips as provided in Sections 6.5.6.4 to 6.5.6.6 shall be permitted if the requirement of Sec 6.5.6.1.6 is satisfied.

6.5.8 Alternative Design of Two-Way Edge-Supported Slabs

6.5.8.1 General

The design method described in this Section shall be based on assumptions given in Sec 6.5.8.2 and 6.5.8.3, and all sections of slabs and supporting members shall be proportioned for moments and shears thus obtained.

6.5.8.2 Scope and limitations

6.5.8.2.1 The provisions of this section may be used as alternative to those of Sections 6.5.1 to 6.5.7 for two-way slabs supported on all four edges by walls, steel beams or monolithic concrete beams having a total depth not less than 3 times the slab thickness.

6.5.8.2.2 Panels shall be rectangular with a longer to shorter centre to centre support span ratio of not greater than 2.

6.5.8.2.3 The value of $\left(\frac{\alpha_{f1} l_2}{l_1}\right)$ shall be greater than or equal to 1.

6.5.8.3 Analysis by the Coefficient Method

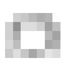
6.5.8.3.1 The negative moments and dead load and live load positive moments in the two directions shall be computed from Tables 6.6.8, 6.6.9 and 6.6.10 respectively. Shear in the slab and loads on the supporting beams shall be computed from Table 6.6.11.

Table 6.6.8: Coefficients for Negative Moments in Slabs †

$$M_{a,neg} = C_{\ a,neg} w l_a^2$$

$$M_{b,neg} = C_{\ b,neg} w l_b^2$$

Where, w = total uniform dead plus live load per unit area

Span Ratio, $m = \frac{l_a}{l_b}$	Moment Coefficient	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
										
1.00	$C_{a,neg}$		0.045		0.050	0.075	0.071		0.033	0.061
	$C_{b,neg}$		0.045	0.076	0.050			0.071	0.061	0.033
0.95	$C_{a,neg}$		0.050		0.055	0.079	0.075		0.038	0.065
	$C_{b,neg}$		0.041	0.072	0.045			0.067	0.056	0.029
0.90	$C_{a,neg}$		0.055		0.060	0.080	0.079		0.043	0.068
	$C_{b,neg}$		0.037	0.070	0.040			0.062	0.052	0.025
0.85	$C_{a,neg}$		0.060		0.066	0.082	0.083		0.049	0.072
	$C_{b,neg}$		0.031	0.065	0.034			0.057	0.046	0.021
0.80	$C_{a,neg}$		0.065		0.071	0.083	0.086		0.055	0.075
	$C_{b,neg}$		0.027	0.061	0.029			0.051	0.041	0.017

Span Ratio, $m = \frac{l_a}{l_b}$	Moment Coefficient	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
0.75	$C_{a,neg}$		0.069		0.076	0.085	0.088		0.061	0.078
	$C_{b,neg}$		0.022	0.056	0.024			0.044	0.036	0.014
0.70	$C_{a,neg}$		0.074		0.081	0.086	0.091		0.068	0.081
	$C_{b,neg}$		0.017	0.050	0.019			0.038	0.029	0.011
0.65	$C_{a,neg}$		0.077		0.085	0.087	0.093		0.074	0.083
	$C_{b,neg}$		0.014	0.043	0.015			0.031	0.024	0.008
0.60	$C_{a,neg}$		0.081		0.089	0.088	0.095		0.080	0.085
	$C_{b,neg}$		0.010	0.035	0.011			0.024	0.018	0.006
0.55	$C_{a,neg}$		0.084		0.092	0.089	0.096		0.085	0.086
	$C_{b,neg}$		0.007	0.028	0.008			0.019	0.014	0.005
0.50	$C_{a,neg}$		0.086		0.094	0.090	0.097		0.089	0.088
	$C_{b,neg}$		0.006	0.022	0.006			0.014	0.010	0.003

† A crosshatched edge indicates that the slab continues across, or is fixed at the support; an unmarked edge indicates a support at which torsional resistance is negligible.

Table 6.6.9: Coefficients for Dead Load Positive Moments in Slabs †

$M_{a,pos,dl} = C_{a,dl} w l_a^2$

$M_{b,pos,dl} = C_{b,dl} w l_b^2$

Where, w = uniform dead load per unit area

Span Ratio, $m = \frac{l_a}{l_b}$	Moment Coefficient	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
1.00	$C_{a,dl}$	0.036	0.018	0.018	0.027	0.027	0.033	0.027	0.020	0.023
	$C_{b,dl}$	0.036	0.018	0.027	0.027	0.018	0.027	0.033	0.023	0.020

Span Ratio, $m = \frac{l_a}{l_b}$	Moment Coefficient	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
										
0.95	$C_{a,dl}$	0.040	0.020	0.021	0.030	0.028	0.036	0.031	0.022	0.024
	$C_{b,dl}$	0.033	0.016	0.025	0.024	0.015	0.024	0.031	0.021	0.017
0.90	$C_{a,dl}$	0.045	0.022	0.025	0.033	0.029	0.039	0.035	0.025	0.026
	$C_{b,dl}$	0.029	0.014	0.024	0.022	0.013	0.021	0.028	0.019	0.015
0.85	$C_{a,dl}$	0.050	0.024	0.029	0.036	0.031	0.042	0.040	0.029	0.028
	$C_{b,dl}$	0.026	0.012	0.022	0.019	0.011	0.017	0.025	0.017	0.013
0.80	$C_{a,dl}$	0.056	0.026	0.034	0.039	0.032	0.045	0.045	0.032	0.029
	$C_{b,dl}$	0.023	0.011	0.020	0.016	0.009	0.015	0.022	0.015	0.010
0.75	$C_{a,dl}$	0.061	0.028	0.040	0.043	0.033	0.048	0.051	0.036	0.031
	$C_{b,dl}$	0.019	0.009	0.018	0.013	0.007	0.012	0.020	0.013	0.007
0.70	$C_{a,dl}$	0.068	0.030	0.046	0.046	0.035	0.051	0.058	0.040	0.033
	$C_{b,dl}$	0.016	0.007	0.016	0.011	0.005	0.009	0.017	0.011	0.006
0.65	$C_{a,dl}$	0.074	0.032	0.054	0.050	0.036	0.054	0.065	0.044	0.034
	$C_{b,dl}$	0.013	0.006	0.014	0.009	0.004	0.007	0.014	0.009	0.005
0.60	$C_{a,dl}$	0.081	0.034	0.062	0.053	0.037	0.056	0.073	0.048	0.036
	$C_{b,dl}$	0.010	0.004	0.011	0.007	0.003	0.006	0.012	0.007	0.004
0.55	$C_{a,dl}$	0.088	0.035	0.071	0.056	0.038	0.058	0.081	0.052	0.037
	$C_{b,dl}$	0.008	0.003	0.009	0.005	0.002	0.004	0.009	0.005	0.003
0.50	$C_{a,dl}$	0.095	0.037	0.080	0.059	0.039	0.061	0.089	0.056	0.038
	$C_{b,dl}$	0.006	0.002	0.007	0.004	0.001	0.003	0.007	0.004	0.002

† A crosshatched edge indicates that the slab continues across, or is fixed at the support; an unmarked edge indicates a support at which torsional resistance is negligible.

Table 6.6.10: Coefficients for Live Load Positive Moments in Slabs †

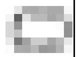
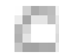
$M_{a,pos,ll} = C_{\ a,ll} w l_a^2$

$M_{b,pos,ll} = C_{\ b,ll} w l_b^2$ Where, w = uniform live load per unit area

Span Ratio, $m = \frac{l_a}{l_b}$	Moment Coefficient	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
										
1.00	$C_{a,ll}$	0.036	0.027	0.027	0.032	0.032	0.035	0.032	0.028	0.030
	$C_{b,ll}$	0.036	0.027	0.032	0.032	0.027	0.032	0.035	0.030	0.028
0.95	$C_{a,ll}$	0.040	0.030	0.031	0.035	0.034	0.038	0.036	0.031	0.032
	$C_{b,ll}$	0.033	0.025	0.029	0.029	0.024	0.029	0.032	0.027	0.025
0.90	$C_{a,ll}$	0.045	0.034	0.035	0.039	0.037	0.042	0.040	0.035	0.036
	$C_{b,ll}$	0.029	0.022	0.027	0.026	0.021	0.025	0.029	0.024	0.022
0.85	$C_{a,ll}$	0.050	0.037	0.040	0.043	0.041	0.046	0.045	0.040	0.039
	$C_{b,ll}$	0.026	0.019	0.024	0.023	0.019	0.022	0.026	0.022	0.020
0.80	$C_{a,ll}$	0.056	0.041	0.045	0.048	0.044	0.051	0.051	0.044	0.042
	$C_{b,ll}$	0.023	0.017	0.022	0.020	0.016	0.019	0.023	0.019	0.017
0.75	$C_{a,ll}$	0.061	0.045	0.051	0.052	0.047	0.055	0.056	0.049	0.046
	$C_{b,ll}$	0.019	0.014	0.019	0.016	0.013	0.016	0.020	0.016	0.013
0.70	$C_{a,ll}$	0.068	0.049	0.057	0.057	0.051	0.060	0.063	0.054	0.050
	$C_{b,ll}$	0.016	0.012	0.016	0.014	0.011	0.013	0.017	0.014	0.011
0.65	$C_{a,ll}$	0.074	0.053	0.064	0.062	0.055	0.064	0.070	0.059	0.054
	$C_{b,ll}$	0.013	0.010	0.014	0.011	0.009	0.010	0.014	0.011	0.009
0.60	$C_{a,ll}$	0.081	0.058	0.071	0.067	0.059	0.068	0.077	0.065	0.059
	$C_{b,ll}$	0.010	0.007	0.011	0.009	0.007	0.008	0.011	0.009	0.007
0.55	$C_{a,ll}$	0.088	0.062	0.080	0.072	0.063	0.073	0.085	0.070	00.063
	$C_{b,ll}$	0.008	0.006	0.009	0.007	0.005	0.006	0.009	0.007	0.006
0.50	$C_{a,ll}$	0.095	0.066	0.088	0.077	0.067	0.078	0.092	0.076	0.067
	$C_{b,ll}$	0.006	0.004	0.007	0.005	0.004	0.005	0.007	0.005	0.004

† A crosshatched edge indicates that the slab continues across, or is fixed at the support; an unmarked edge indicates a support at which torsional resistance is negligible.

Table 6.6.11: Ratio of Total Load w in l_a and l_b Directions (W_a and W_b) for Shear in Slab and Load on Supports †

Span Ratio, $m = \frac{l_a}{l_b}$	Load Ratio	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7	Case 8	Case 9
										
1.00	w_a	0.50	0.50	0.17	0.50	0.83	0.71	0.29	0.33	0.67
	w_b	0.50	0.50	0.83	0.50	0.17	0.29	0.71	0.67	0.33
0.95	w_a	0.55	0.55	0.20	0.55	0.86	0.75	0.33	0.38	0.71
	w_b	0.45	0.45	0.80	0.45	0.14	0.25	0.67	0.62	0.29
0.90	w_a	0.60	0.60	0.23	0.60	0.88	0.79	0.38	0.43	0.75
	w_b	0.40	0.40	0.77	0.40	0.12	0.21	0.62	0.57	0.25
0.85	w_a	0.66	0.66	0.28	0.66	0.90	0.83	0.43	0.49	0.79
	w_b	0.34	0.34	0.72	0.34	0.10	0.17	0.57	0.51	0.21
0.80	w_a	0.71	0.71	0.33	0.71	0.92	0.86	0.49	0.55	0.83
	w_b	0.29	0.29	0.67	0.29	0.08	0.14	0.51	0.45	0.17
0.75	w_a	0.76	0.76	0.39	0.76	0.94	0.88	0.56	0.61	0.86
	w_b	0.24	0.24	0.61	0.24	0.06	0.12	0.44	0.39	0.14
0.70	w_a	0.81	0.81	0.45	0.81	0.95	0.91	0.62	0.68	0.89
	w_b	0.19	0.19	0.55	0.19	0.05	0.09	0.38	0.32	0.11
0.65	w_a	0.85	0.85	0.53	0.85	0.96	0.93	0.69	0.74	0.92
	w_b	0.15	0.15	0.47	0.15	0.04	0.07	0.31	0.26	0.08
0.60	w_a	0.89	0.89	0.61	0.89	0.97	0.95	0.76	0.80	0.94
	w_b	0.11	0.11	0.39	0.11	0.03	0.05	0.24	0.20	0.06
0.55	w_a	0.92	0.92	0.69	0.92	0.98	0.96	0.81	0.85	0.95
	w_b	0.08	0.08	0.31	0.08	0.02	0.04	0.19	0.15	0.05
0.50	w_a	0.94	0.94	0.76	0.94	0.99	0.97	0.86	0.89	0.97
	w_b	0.06	0.06	0.24	0.06	0.01	0.03	0.14	0.11	0.03

† A crosshatched edge indicates that the slab continues across, or is fixed at the support; an unmarked edge indicates a support at which torsional resistance is negligible.

6.5.8.4 Shear on Supporting Beam

The shear requirements provided in Sec 6.5.6.8 shall be satisfied.

6.5.8.5 Deflection

Thickness of slabs supported on walls or stiff beams on all sides shall satisfy the requirements of Sec 6.2.5.3.

6.5.8.6 Reinforcement

6.5.8.6.1 Area of reinforcement in each direction shall be determined from moments at critical sections but shall not be less than that required by Sec 8.1.11 Chapter 8.

6.5.8.6.2 Spacing of reinforcement at critical sections shall not exceed two times the slab thickness, except for portions of slab area that may be of cellular or ribbed construction. In the slab over cellular spaces, reinforcement shall be provided as required by Sec 8.1.11 Chapter 8.

6.5.8.6.3 Positive moment reinforcement perpendicular to a discontinuous edge shall extend to the edge of slab and have embedment, straight or hooked, at least 150 mm in spandrel beams, columns, or walls.

6.5.8.6.4 Negative moment reinforcement perpendicular to a discontinuous edge shall be bent, hooked, or otherwise anchored, in spandrel beams, columns, or walls, and shall be developed at face of support according to provisions of Sec 8.2 Chapter 8.

6.5.8.6.5 Corner reinforcement

Corner reinforcement shall be provided at exterior corners in both bottom and top of the slab, for a distance in each direction from the corner equal to one-fifth the longer span of the corner panel as per provisions of Sec 6.5.3.6.

6.5.9 Ribbed and Hollow Slabs

6.5.9.1 General

The provisions of this section shall apply to 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;

(c) Slabs with a continuous top and bottom face but containing voids of rectangular, oval or other shape.

6.5.9.2 Analysis and design

Any method of analysis which satisfies equilibrium and compatibility requirements may be used for ribbed and hollow slabs. Approximate moments and shears in continuous one-way ribbed or hollow slabs may be obtained from Sec 6.1.4.3. For two-way slabs, the unified design approach specified in Sec 6.5 Flat Plates, Flat Slabs and Edge-supported Slabs, shall be used.

6.5.9.3 Shear

6.5.9.3.1 When burnt tile or concrete tile fillers of material having the same strength as the specified strength of concrete in the ribbed and hollow slabs are used permanently, it is permitted to include the vertical shells of fillers in contact with the ribs for shear and negative-moment strength computations, provided adequate bond between the two can be ensured.

6.5.9.4 Deflection

The recommendations for deflection with respect to solid slabs may be applied to ribbed and hollow slab. Total depth of one-way ribbed and hollow slabs shall not be less than those required by Table 6.6.1 in Sec 6.2.5.2. For other slabs the provisions of Sec 6.2.5.3 shall apply.

6.5.9.5 Size and Position of Ribs

In-situ-ribs shall be not less than 100 mm wide. They shall be spaced at centres not greater than 750 mm apart and their depth, excluding any topping, shall be not more than three and half times their width. Ribs shall be formed along each edge parallel to the span of one-way slabs.

6.5.9.6 Reinforcement

The recommendations given in Sec 8.1.6 Chapter 8 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 specified below:

(a) At least 50 percent of the total main reinforcement shall be carried through the bottom on to the bearing and anchored in accordance with Sec 8.2.8 Chapter 8.

(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 of that required in the middle of the adjoining spans and shall extend at least one-tenth of the clear span into adjoining spans.

    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 Sec 8.1.7 Chapter 8.

6.5.9.6.1 Adequate shear strength of slabs shall be provided in accordance with the requirements of Sec 6.4.10. For one-way ribbed and hollow slab construction, contribution of concrete to shear strength V_c is permitted to be 10 percent more than that specified in Sec 6.4.2. It is permitted to increase shear strength using shear reinforcement or by widening the ends of ribs.

6.6 Walls

6.6.1 Scope

6.6.1.1 Provisions of Sec. 6.6 shall apply for design of walls subjected to axial load, with or without flexure.

6.6.1.2 Cantilever retaining walls are designed according to flexural design provisions of Sec 6.3 with minimum horizontal reinforcement according to Sec 6.6.3.3.

6.6.2 General

6.6.2.1 Walls shall be designed for eccentric loads and any lateral or other loads to which they are subjected.

6.6.2.2 Walls subject to axial loads shall be designed in accordance with Sections 6.6.2, 6.6.3, and either Sec 6.6.4, Sec 6.6.5, or Sec 6.6.8.

6.6.2.3 Design for shear shall be in accordance with Sec 6.4.8.

6.6.2.4 Unless otherwise demonstrated by an analysis, the horizontal length of wall considered as effective for each concentrated load shall not exceed the smaller of the center-to-center distance between loads, and the bearing width plus four times the wall thickness.

6.6.2.5 Compression members built integrally with walls shall conform to Sec 6.3.8.2.

6.6.2.6 Walls shall be anchored to intersecting elements, such as floors and roofs; or to columns pilasters, buttresses, of intersecting walls; and to footings.

6.6.2.7 Quantity of reinforcement and limits of thickness required by Sections 6.6.3 and 6.6.5 shall be permitted to be waived where structural analysis shows adequate strength and stability.

6.6.2.8 Transfer of force to footing at base of wall shall be in accordance with Sec 6.8.8.

6.6.3 Minimum reinforcement

6.6.3.1 Minimum vertical and horizontal reinforcement shall be in accordance with Sections 6.6.3.2 and 6.6.3.3 unless a greater amount is required for shear by Sections 6.4.8.8 and 6.4.8.9.

6.6.3.2 Minimum ratio of vertical reinforcement area to gross concrete area, ρ_l, shall be:

(a) 0.0012 for deformed bars not larger than 16 mm diameter with f_y not less than 420 MPa; or

(b) 0.0015 for other deformed bars; or

(c) 0.0012 for welded wire reinforcement not larger than MW200 or MD200.

6.6.3.3 Minimum ratio of horizontal reinforcement area to gross concrete area, ρ_t, shall be:

(a) 0.0020 for deformed bars not larger than 16 mm diameter with f_y not less than 420 MPa; or

(b) 0.0025 for other deformed bars; or

(c) 0.0020 for welded wire reinforcement not larger than MW200 or MD200.

6.6.3.4 Walls more than 250 mm thick, except basement walls, shall have reinforcement for each direction placed in two layers parallel with faces of wall in accordance with the following:

(a) One layer consisting of not less than one-half and not more than two-thirds of total reinforcement required for each direction shall be placed not less than 50 mm nor more than one-third the thickness of wall from the exterior surface;

(b) The other layer, consisting of the balance of required reinforcement in that direction, shall be placed not less than 20 mm nor more than one-third the thickness of wall from the interior surface.

6.6.3.5 Vertical and horizontal reinforcement shall not be spaced farther apart than three times the wall thickness, nor farther apart than 450 mm.

6.6.3.6 Vertical reinforcement need not be enclosed by lateral ties if vertical reinforcement area is not greater than 0.01 times gross concrete area, or where vertical reinforcement is not required as compression reinforcement.

6.6.3.7 In addition to the minimum reinforcement required by Sec 6.6.3.1, not less than two 16 mm diameter bars in walls having two layers of reinforcement in both directions and one 16 mm diameter bar in walls having a single layer of reinforcement in both directions shall be provided around window, door, and similar sized openings. Such bars shall be anchored to develop f_y in tension at the corners of the openings.

6.6.4 Design of Walls as Compression Members

Except as provided in Sec 6.6.5, walls subject to axial load or combined flexure and axial load shall be designed as compression members in accordance with provisions of Sections 6.3.2, 6.3.3, 6.3.10, 6.3.11, 6.3.14, 6.6.2, and 6.6.3.

6.6.5 Empirical Method of Design

6.6.5.1 Walls of solid rectangular cross section shall be permitted to be designed by the empirical provisions of Sec 6.6.5 if the resultant of all factored loads is located within the middle third of the overall thickness of the wall and all limits of Sections 6.6.2, 6.6.3, and 6.6.5 are satisfied.

6.6.5.2 Design axial strength ϕP_n of a wall satisfying limitations of Sec 6.6.5.1 shall be computed by Eq. 6.6.89 unless designed in accordance with 6.6.4.

$$\phi P_n = 0.55\phi f_c' A_g \left[1 - \left(\frac{Kl_c}{32h}\right)^2\right] \qquad (6.6.89)$$

Where, ϕ shall correspond to compression-controlled sections in accordance with Sec 6.2.3.2.2 and effective length factor k shall be:

(a)	For walls braced top and bottom against lateral translation and	
	Restrained against rotation at one or both ends (top, bottom, or	0.8
	Unrestrained against rotation at both ends	1.0
(b)	For walls not braced against lateral translation	2.0

6.6.5.3 Minimum thickness of walls designed by empirical design method

6.6.5.3.1 Thickness of bearing walls shall not be less than 1/25 the supported height or length, whichever is shorter, nor less than 100 mm.

6.6.5.3.2 Thickness of exterior basement walls and foundation walls shall not be less than 190 mm.

6.6.6 Nonbearing Walls

6.6.6.1 Thickness of nonbearing walls shall not be less than 100 mm, nor less than 1/30 the least distance between members that provide lateral support.

6.6.7 Walls as Grade Beams

6.6.7.1 Walls designed as grade beams shall have top and bottom reinforcement as required for moment in accordance with provisions of Sections 6.3.2 to 6.3.7. Design for shear shall be in accordance with provisions of Sec. 6.4.

6.6.7.2 Portions of grade beam walls exposed above grade shall also meet requirements of Sec 6.6.3.

6.6.8 Alternative Design of Slender Walls

6.6.8.1 When flexural tension controls the out-of-plane design of a wall, the requirements of Sec 6.6.8 are considered to satisfy Sec 6.3.10.

6.6.8.2 Walls designed by the provisions of Sec 6.6.8 shall satisfy Sections 6.6.8.2.1 to 6.6.8.2.6.

6.6.8.2.1 The wall panel shall be designed as a simply supported, axially loaded member subjected to an out-of-plane uniform lateral load, with maximum moments and deflections occurring at midspan.

6.6.8.2.2 The cross section shall be constant over the height of the panel.

6.6.8.2.3 The wall shall be tension-controlled.

6.6.8.2.4 Reinforcement shall provide a design Strength

$$\phi M_n \geq M_{cr} \qquad (6.6.90)$$

Where, M_{cr} shall be obtained using the modulus of rupture, f_r, given by Eq. 6.6.91.

6.6.8.2.5 Concentrated gravity loads applied to the wall above the design flexural section shall be assumed to be distributed over a width:

(a) Equal to the bearing width, plus a width on each side that increases at a slope of 2 vertical to 1 horizontal down to the design section; but

(b) Not greater than the spacing of the concentrated loads; and

(c) Not extending beyond the edges of the wall panel.

6.6.8.2.6 Vertical stress P_u / A_g at the midheight section shall not exceed $0.06 f'_c$.

6.6.8.3 Design moment strength φM_n for combined flexure and axial loads at midheight shall be

$$\phi M_n \geq M_u \qquad (6.6.91)$$

Where,

$$M_u = M_{ua} + P_u \Delta_u \qquad (6.6.92)$$

M_{ua} is the maximum factored moment at midheight of wall due to lateral and eccentric vertical loads, not including $P\Delta$effects, and Δ_u is

$$\Delta_u = \frac{5 M_u l_c^2}{(0.75) 48 E_c I_{cr}} \qquad (6.6.93)$$

M_u shall be obtained by iteration of deflections, or by Eq. 6.6.94.

$$M_u = \frac{M_{ua}}{1 - \frac{5 P_u l_c^2}{(0.75) 48 E_c I_{cr}}} \qquad (6.6.94)$$

Where,

$$I_{cr} = \frac{E_s}{E_c}\left(A_s + \frac{P_u}{f_y}\frac{h}{2d}\right)(d-c)^2 + \frac{l_w c^3}{3} \qquad (6.6.95)$$

And, the value of E_s/E_c shall not be taken less than 6.

6.6.8.4 Maximum out-of-plane deflection, Δ_s, due to service loads, including $P\Delta$ effects, shall not exceed $l_c/150$.

If M_a, maximum moment at midheight of wall due to service lateral and eccentric vertical loads, including $P\Delta$ effects, exceeds $(2/3)M_{cr}$, Δ_s shall be calculated by Eq. 6.6.96

$$\Delta_s = (2/3)\Delta_{cr} + \frac{(M_a - (2/3)M_{cr})}{(M_n - (2/3)M_{cr})}(\Delta_n - (2/3)\Delta_{cr}) \qquad (6.6.96)$$

If M_a does not exceed $(2/3)M_{cr}$, Δ_s shall be calculated by Eq. 6.6.97

$$\Delta_s = \left(\frac{M_a}{M_{cr}}\right)\Delta_{cr} \qquad (6.6.97)$$

Where,

$$\Delta_{cr} = \frac{5M_{cr}l_c^2}{48E_cI_g} \qquad (6.6.98)$$

$$\Delta_n = \frac{5M_nl_c^2}{48E_cI_{cr}} \qquad (6.6.99)$$

I_{cr} shall be calculated by Eq. 6.6.95, and M_a shall be obtained by iteration of deflections.

6.7 Stairs

Stairs are the structural elements designed to connect different floors. The stairs shall be designed to meet the minimum load requirements. The flight arrangements, configuration and support conditions (Figure 6.6.24) shall govern the design procedure to follow.

6.7.1 Stairs Supported at Floor and Landing Level

6.7.1.1 Effective span

The effective span of stairs without stringer beams shall be taken as the following horizontal distances:

(a) Centre to centre distance of beams, where supported at top and bottom risers by beams spanning parallel with the risers,

(b) Where supported at the edge of a landing slab, which spans parallel with the risers, (Figure 6.6.25a) a distance equal to the going of the stairs plus at each end either half the width of the landing or 1.0m whichever is smaller. The going shall be measured horizontally.

(c) Where the landing spans in the same direction of the stairs (Figure 6.6.25b), the span shall be the distance centre to centre of the supporting beams or walls.

(d) Where the landing slabs, running at right angle to the direction of the flight, supported by walls or beams on three sides (Figure 6.6.25c), the effective span shall be going of the stair measured horizontally. Both positive and negative moments along the direction of the flight shall be calculated as $wl^2/8$, where w is the intensity of the total dead and live load per unit area on a horizontal plane.

Figure 6.6.24 Different forms of stairs and landing arrangements

6.7.1.2 Loading

Staircases shall be designed to support the design ultimate load according to the load combinations specified in Chapter 2, loads.

6.7.1.3 Distribution of loading

6.7.1.3.1 Where flights or landing are embedded at least 110 mm into walls 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 may be increased by 75 mm for the purpose of design (Figure 6.6.26)

In the case of stairs with open wells, where spans cross at right angles, the load on areas common to any two such spans may be taken as one half in each direction as shown in Figure 6.6.27.

6.7.1.4 Depth of section

The depth of the section shall be taken as the minimum thickness perpendicular to the soffit of the staircase.

6.7.1.5 Design

6.7.1.5.1 Strength, deflection and crack control

The recommendations given in Sections 6.1 and 6.2 for beams and one-way slabs shall apply, except for the span/depth ratio of staircases without stringer beam where the provision of Sec 6.7.1.5.2 below shall apply.

6.7.1.5.2 Permissible span/effective depth ratio for staircase without stringer beams: In case of stair flight that occupies at least 60% of the span, the ratio calculated in accordance with Sec 6.2.5.2 shall be increased by 15%.

6.7.2 Special Types of Stairs

The provisions of special types of stairs like Free Standing (Landing unsupported), Sawtooth (Slabless) and Helicoidal are provided in Appendix M.

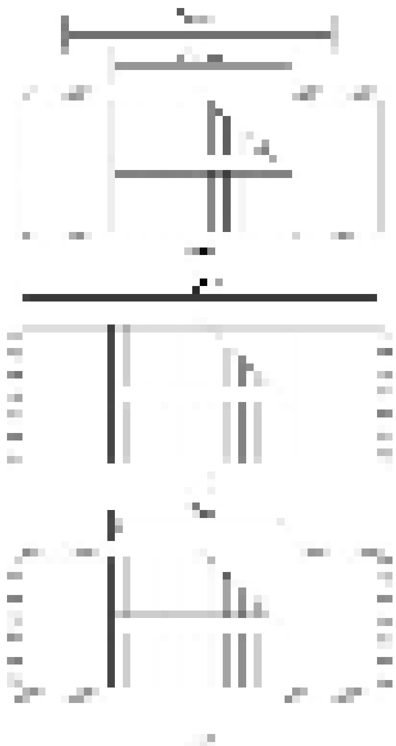


Figure 6.6.25 Effective Span for Stairs Supported at Each End by Landings

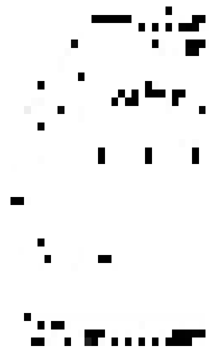


Figure 6.6.26 Loading on stairs Built in a wall

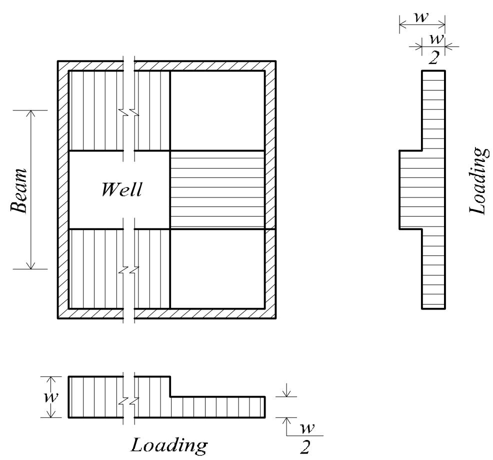


Figure 6.6.27 Loading of stairs with open wells

6.8 Footings

6.8.1 Scope

6.8.1.1 Provisions of Sec. 6.8 shall apply for design of isolated footings and, where applicable, to combined footings and mats.

6.8.1.2 Additional requirements for design of combined footings and mats are given in Sec 6.8.10.

6.8.2 Loads and Reactions

6.8.2.1 Footings shall be proportioned to resist the factored loads and induced reactions, in accordance with the appropriate design requirements of this Code and as provided in Sec. 6.8.

6.8.2.2 Base area of footing or number and arrangement of piles shall be determined from unfactored forces and moments transmitted by footing to soil or piles and permissible soil pressure or permissible pile capacity determined using principles of soil mechanics.

6.8.2.3 For footings on piles, computations for moments and shears shall be permitted to be based on the assumption that the reaction from any pile is concentrated at pile center.

6.8.3 Equivalent Square Shapes for Circular or Regular Polygon-Shaped Columns or Pedestals Supported By Footings

For location of critical sections for moment, shear, and development of reinforcement in footings, it shall be permitted to treat circular or regular polygon-shaped concrete columns or pedestals as square members with the same area.

6.8.4 Moment in Footings

6.8.4.1 External moment on any section of a footing shall be determined by passing a vertical plane through the footing, and computing the moment of the forces acting over entire area of footing on one side of that vertical plane.

6.8.4.2 Maximum factored moment, M_u, for an isolated footing shall be computed as prescribed in Sec 6.8.4.1 at critical sections located as follows:

(a) At face of column, pedestal, or wall, for footings supporting a concrete column, pedestal, or wall;

(b) Halfway between middle and edge of wall, for footings supporting a masonry wall;

(c) Halfway between face of column and edge of steel base plate, for footings supporting a column with steel base plate.

6.8.4.3 In one-way footings and two-way square footings, reinforcement shall be distributed uniformly across entire width of footing.

6.8.4.4 In two-way rectangular footings, reinforcement shall be distributed in accordance with Sections 6.8.4.4.1 and 6.8.4.4.2.

6.8.4.4.1 Reinforcement in long direction shall be distributed uniformly across entire width of footing.

6.8.4.4.2 For reinforcement in short direction, a portion of the total reinforcement, $\gamma_s A_s$, shall be distributed uniformly over a band width (centered on centerline of column or pedestal) equal to the length of short side of footing. Remainder of reinforcement required in short direction$(1 - \gamma_s)A_s$, shall be distributed uniformly outside center band width of footing.

$$\gamma_s = \frac{2}{(\beta+1)} \qquad (6.6.100)$$

Where, β is ratio of long to short sides of footing.

6.8.5 Shear in Footings

6.8.5.1 Shear strength of footings supported on soil or rock shall be in accordance with Sec 6.4.10.

6.8.5.2 Location of critical section for shear in accordance with Sec. 6.4 shall be measured from face of column, pedestal, or wall, for footings supporting a column, pedestal, or wall. For footings supporting a column or pedestal with steel base plates, the critical section shall be measured from location defined in Sec 6.8.4.2(c).

6.8.5.3 Where the distance between axis of any pile to the axis of the column is more than two times the distance between the top of the pile cap and the top of the pile, the pile cap shall satisfy Sections 6.4.10 and 6.8.5.4. Other pile caps shall satisfy either Appendix I, or both Sections 6.4.10 and 6.8.5.4. If Appendix I is used, the effective concrete compression strength of the struts, f_{ce}, shall be determined using Sec I.3.2.2(b).

6.8.5.4 Computation of shear on any section through a footing supported on piles (Figure 6.6.28) shall be in accordance with Sections 6.8.5.4.1, 6.8.5.4.2, and 6.8.5.4.3.

6.8.5.4.1 Entire reaction from any pile with its center located $\frac{d_{pile}}{2}$ or more outside the section shall be considered as producing shear on that section.

6.8.5.4.2 Reaction from any pile with its center located $\frac{d_{pile}}{2}$ or more inside the section shall be considered as producing no shear on that section.

6.8.5.4.3 For intermediate positions of pile center, the portion of the pile reaction to be considered as producing shear on the section shall be based on straight-line interpolation between full value at $\frac{d_{pile}}{2}$ outside the section and zero value at $\frac{d_{pile}}{2}$ inside the section.

Figure 6.6.28 Modified critical perimeter for shear with over-lapping critical perimeters.

6.8.6 Development of Reinforcement in Footings

6.8.6.1 Development of reinforcement in footings shall be in accordance with Sec. 8.2.

6.8.6.2 Calculated tension or compression in reinforcement at each section shall be developed on each side of that section by embedment length, hook (tension only) or mechanical device, or a combination thereof.

6.8.6.3 Critical sections for development of reinforcement shall be assumed at the same locations as defined in 6.8.4.2 for maximum factored moment, and at all other vertical planes where changes of section or reinforcement occur. See also 8.2.7.6.

6.8.7 Minimum Footing Depth

Depth of footing above bottom reinforcement shall not be less than 150 mm for footings on soil, nor less than 300 mm for footings on piles.

6.8.8 Force Transfer at Base of Column, Wall, or Reinforced Pedestal

6.8.8.1 Forces and moments at base of column, wall, or pedestal shall be transferred to supporting pedestal or footing by bearing on concrete and by reinforcement, dowels, and mechanical connectors.

6.8.8.1.1 Bearing stress on concrete at contact surface between supported and supporting member shall not exceed concrete bearing strength for either surface as given by Sec 6.3.14.

6.8.8.1.2 Reinforcement, dowels, or mechanical connectors between supported and supporting members shall be adequate to transfer:

(a) All compressive force that exceeds concrete bearing strength of either member;

(b) Any computed tensile force across interface.

In addition, reinforcement, dowels, or mechanical connectors shall satisfy Sec 6.8.8.2 or Sec 6.8.8.3.

6.8.8.1.3 If calculated moments are transferred to supporting pedestal or footing, then reinforcement, dowels, or mechanical connectors shall be adequate to satisfy Sec 8.2.15.

6.8.8.1.4 Lateral forces shall be transferred to supporting pedestal or footing in accordance with shear-friction provisions of Sec 6.4.5, or by other appropriate means.

6.8.8.2 In cast-in-place construction, reinforcement required to satisfy Sec 6.8.8.1 shall be provided either by extending longitudinal bars into supporting pedestal or footing, or by dowels.

6.8.8.2.1 For cast-in-place columns and pedestals, area of reinforcement across interface shall be not less than $0.005A_g$, where A_g is the gross area of the supported member.

6.8.8.2.2 For cast-in-place walls, area of reinforcement across interface shall be not less than minimum vertical reinforcement given in Sec 6.6.3.2.

6.8.8.2.3 At footings, it shall be permitted to lap splice 43 mm diameter and 57 mm diameter longitudinal bars, in compression only, with dowels to provide reinforcement required to satisfy Sec 6.8.8.1. Dowels shall not be larger than 36 mm diameter bar and shall extend into supported member a distance not less than the larger of $l_{dc'}$ of 43 mm diameter or 57 mm diameter bars and compression lap splice length of the dowels, whichever is greater, and into the footing a distance not less than l_{dc} of the dowels.

6.8.8.2.4 If a pinned or rocker connection is provided in cast-in-place construction, connection shall conform to the provisions of Sections 6.8.8.1 and 6.8.8.3.

6.8.8.3 In precast construction, anchor bolts or suitable mechanical connectors shall be permitted for satisfying 6.8.8.1. Anchor bolts shall be designed in accordance with Appendix K.

6.8.8.3.1 Connection between precast columns or pedestals and supporting members shall meet the requirements of Sec 6.10.5.1.3(a).

6.8.8.3.2 Connection between precast walls and supporting members shall meet the requirements of Sec 6.10.5.1.3(b) and (c).

6.8.8.3.3 Anchor bolts and mechanical connections shall be designed to reach their design strength before anchorage failure or failure of surrounding concrete. Anchor bolts shall be designed in accordance with Appendix K.

6.8.9 Stepped or Sloped Footings

6.8.9.1 In sloped or stepped footings, angle of slope or depth and location of steps shall be such that design requirements are satisfied at every section. (See also Sec 8.2.7.6.)

6.8.9.2 Sloped or stepped footings designed as a unit shall be constructed to ensure action as a unit.

6.8.10 Combined Footings and Mats

6.8.10.1 Footings supporting more than one column, pedestal, or wall (combined footings or mats) shall be proportioned to resist the factored loads and induced reactions, in accordance with appropriate design requirements of the Code.

6.8.10.2 The direct design method of Sec. 6.5 shall not be used for design of combined footings and mats.

6.8.10.3 Distribution of soil pressure under combined footings and mats shall be consistent with properties of the soil and the structure and with established principles of soil mechanics.

6.8.10.4 Minimum reinforcing steel in mat foundations shall meet the requirements of Sec. 8.1.11.2 in each principal direction. Maximum spacing shall not exceed 450 mm.

6.9 Folded Plates and Shells

6.9.1 Scope and Definitions

6.9.1.1 Provisions of Sec. 6.9 shall apply to thin shell and folded plate concrete structures, including ribs and edge members.

6.9.1.2 All provisions of this Code not specifically excluded, and not in conflict with provisions of Sec. 6.9, shall apply to thin-shell structures.

6.9.1.3 Thin shells

Three-dimensional spatial structures made up of one or more curved slabs or folded plates whose thicknesses are small compared to their other dimensions. Thin shells are characterized by their three-dimensional load-carrying behavior, which is determined by the geometry of their forms, by the manner in which they are supported, and by the nature of the applied load.

6.9.1.4 Folded plates

A class of shell structure formed by joining flat, thin slabs along their edges to create a three-dimensional spatial structure.

6.9.1.5 Ribbed shells

Spatial structures with material placed primarily along certain preferred rib lines, with the area between the ribs filled with thin slabs or left open.

6.9.1.6 Auxiliary members

Ribs or edge beams that serve to strengthen, stiffen, or support the shell; usually, auxiliary members act jointly with the shell.

6.9.1.7 Elastic analysis

An analysis of deformations and internal forces based on equilibrium, compatibility of strains, and assumed elastic behavior, and representing to a suitable approximation the three-dimensional action of the shell together with its auxiliary members.

6.9.1.8 Inelastic analysis

An analysis of deformations and internal forces based on equilibrium, nonlinear stress-strain relations for concrete and reinforcement, consideration of cracking and time-dependent effects, and compatibility of strains. The analysis shall represent to a suitable approximation three-dimensional action of the shell together with its auxiliary members.

6.9.1.9 Experimental analysis

An analysis procedure based on the measurement of deformations or strains, or both, of the structure or its model; experimental analysis is based on either elastic or inelastic behavior.

6.9.2 Analysis and Design

6.9.2.1 Elastic behavior shall be an accepted basis for determining internal forces and displacements of thin shells. This behavior shall be permitted to be established by computations based on an analysis of the uncracked concrete structure in which the material is assumed linearly elastic, homogeneous, and isotropic. Poisson's ratio of concrete shall be permitted to be taken equal to zero.

6.9.2.2 Inelastic analyses shall be permitted to be used where it can be shown that such methods provide a safe basis for design.

6.9.2.3 Equilibrium checks of internal resistances and external loads shall be made to ensure consistency of results.

6.9.2.4 Experimental or numerical analysis procedures shall be permitted where it can be shown that such procedures provide a safe basis for design.

6.9.2.5 Approximate methods of analysis shall be permitted where it can be shown that such methods provide a safe basis for design.

6.9.2.6 The thickness of a shell and its reinforcement shall be proportioned for the required strength and serviceability, using either the strength design method of Sec 6.1.2.1 or the design method of Sec 6.1.2.2.

6.9.2.7 Shell instability shall be investigated and shown by design to be precluded.

6.9.2.8 Auxiliary members shall be designed according to the applicable provisions of the Code. It shall be permitted to assume that a portion of the shell equal to the flange width, as specified in Sec 6.1.13, acts with the auxiliary member. In such portions of the shell, the reinforcement perpendicular to the auxiliary member shall be at least equal to that required for the flange of a T-beam by Sec 6.1.13.5.

6.9.2.9 Strength design of shell slabs for membrane and bending forces shall be based on the distribution of stresses and strains as determined from either an elastic or an inelastic analysis.

6.9.2.10 In a region where membrane cracking is predicted, the nominal compressive strength parallel to the cracks shall be taken as $0.4f_c'$.

6.9.3 Design Strength of Materials

6.9.3.1 Specified compressive strength of concrete f_c' at 28 days shall not be less than 21 MPa.

6.9.3.2 Specified yield strength of reinforcement f_y shall not exceed 420 MPa.

6.9.4 Shell Reinforcement

6.9.4.1 Shell reinforcement shall be provided to resist tensile stresses from internal membrane forces, to resist tension from bending and twisting moments, to limit shrinkage and temperature crack width and spacing, and as reinforcement at shell boundaries, load attachments, and shell openings.

6.9.4.2 Tensile reinforcement shall be provided in two or more directions and shall be proportioned such that its resistance in any direction equals or exceeds the component of internal forces in that direction. Alternatively, reinforcement for the membrane forces in the slab shall be calculated as the reinforcement required to resist axial tensile forces plus the tensile force due to shear-friction required to transfer shear across any cross section of the membrane. The assumed coefficient of friction, μ, shall not exceed that specified in Sec 6.4.5.4.3.

6.9.4.3 The area of shell reinforcement at any section as measured in two orthogonal directions shall not be less than the slab shrinkage or temperature reinforcement required by Sec 8.1.11.

6.9.4.4 Reinforcement for shear and bending moments about axes in the plane of the shell slab shall be calculated in accordance with Sections 6.3, 6.4 and 6.5.

6.9.4.5 The area of shell tension reinforcement shall be limited so that the reinforcement will yield before either crushing of concrete in compression or shell buckling can take place.

6.9.4.6 In regions of high tension, membrane reinforcement shall, if practical, be placed in the general directions of the principal tensile membrane forces. Where this is not practical, it shall be permitted to place membrane reinforcement in two or more component directions.

6.9.4.7 If the direction of reinforcement varies more than 10^{o} from the direction of principal tensile membrane force, the amount of reinforcement shall be reviewed in relation to cracking at service loads.

6.9.4.8 Where the magnitude of the principal tensile membrane stress within the shell varies greatly over the area of the shell surface, reinforcement resisting the total tension shall be permitted to be concentrated in the regions of largest tensile stress where it can be shown that this provides a safe basis for design. However, the ratio of shell reinforcement in any portion of the tensile zone shall be not less than 0.0035 based on the overall thickness of the shell.

6.9.4.9 Reinforcement required to resist shell bending moments shall be proportioned with due regard to the simultaneous action of membrane axial forces at the same location. Where shell reinforcement is required in only one face to resist bending moments, equal amounts shall be placed near both surfaces of the shell even though a reversal of bending moments is not indicated by the analysis.

6.9.4.10 Shell reinforcement in any direction shall not be spaced farther apart than 450 mm nor farther apart than five times the shell thickness. Where the principal membrane tensile stress on the gross concrete area due to factored loads exceeds $0.33\phi\lambda\sqrt{f_c'}$, reinforcement shall not be spaced farther apart than three times the shell thickness.

6.9.4.11 Shell reinforcement at the junction of the shell and supporting members or edge members shall be anchored in or extended through such members in accordance with the requirements of Sec. 8.2, except that the minimum development length shall be $1.2l_d$ but not less than 450 mm.

6.9.4.12 Splice lengths of shell reinforcement shall be governed by the provisions of Sec. 8.2, except that the minimum splice length of tension bars shall be 1.2 times the value required by Sec. 8.2 but not less than 450 mm. The number of splices in principal tensile reinforcement shall be kept to a practical minimum. Where splices are necessary they shall be staggered at least l_d with not more than one-third of the reinforcement spliced at any section.

6.9.5 Construction

6.9.5.1 When removal of formwork is based on a specific modulus of elasticity of concrete because of stability or deflection considerations, the value of the modulus of elasticity, E_c, used shall be determined from flexural tests of field-cured beam specimens. The number of test specimens, the dimensions of test beam specimens, and test procedures shall be specified by the Engineer.

6.9.5.2 Contract documents shall specify the tolerances for the shape of the shell. If construction results in deviations from the shape greater than the specified tolerances, an analysis of the effect of the deviations shall be made and any required remedial actions shall be taken to ensure safe behavior.

6.10 Precast Concrete

6.10.1 Scope

6.10.1.1 All provisions of this Code, not specifically excluded and not in conflict with the provisions of Sec 6.10, shall apply to structures incorporating precast concrete structural members.

6.10.2 General

6.10.2.1 Design of precast members and connections shall include loading and restraint conditions from initial fabrication to end use in the structure, including form removal, storage, transportation, and erection.

6.10.2.2 When precast members are incorporated into a structural system, the forces and deformations occurring in and adjacent to connections shall be included in the design.

6.10.2.3 Tolerances for both precast members and interfacing members shall be specified. Design of precast members and connections shall include the effects of these tolerances.

6.10.2.4 In addition to the requirements for drawings and specifications in Sec 1.9.3 of Chapter 1, the following (a) and (b) shall be included in either the contract documents or shop drawings:

(a) Details of reinforcement, inserts and lifting devices required to resist temporary loads from handling, storage, transportation, and erection;

(b) Required concrete strength at stated ages or stages of construction.

6.10.3 Distribution of Forces in Members

6.10.3.1 Distribution of forces that are perpendicular to the plane of members shall be established by analysis or by test.

6.10.3.2 Where the system behavior requires in-plane forces to be transferred between the members of a precast floor or wall system, Sections 6.10.3.2.1 and 6.10.3.2.2 shall apply.

6.10.3.2.1 In-plane force paths shall be continuous through both connections and members.

6.10.3.2.2 Where tension forces occur, a continuous path of steel or steel reinforcement shall be provided.

6.10.4 Member Design

6.10.4.1 In one-way precast floor and roof slabs and in one-way precast, prestressed wall panels, all not wider than 3.7 m, and where members are not mechanically connected to cause restraint in the transverse direction, the shrinkage and temperature reinforcement requirements of Sec. 8.1.11 in the direction normal to the flexural reinforcement shall be permitted to be waived. This waiver shall not apply to members that require reinforcement to resist transverse flexural stresses.

6.10.4.2 For precast, non prestressed walls the reinforcement shall be designed in accordance with the provisions of Sec 6.3 or Sec 6.6, except that the area of horizontal and vertical reinforcement each shall be not less than $0.001A_g$, where A_g is the gross cross-sectional area of the wall panel. Spacing of reinforcement shall not exceed 5 times the wall thickness nor 750 mm for interior walls nor 450 mm for exterior walls.

6.10.5 Structural Integrity

6.10.5.1 Except where the provisions of Sec 6.10.5.2 govern, the minimum provisions of Sec 6.10.5.1.1 to 6.10.5.1.4 for structural integrity shall apply to all precast concrete structures.

6.10.5.1.1 Longitudinal and transverse ties required by Sec 8.1.12.3 shall connect members to a lateral load-resisting system.

6.10.5.1.2 Where precast elements form floor or roof diaphragms, the connections between diaphragm and those members being laterally supported shall have a nominal tensile strength capable of resisting not less than 4.4 kN per linear m.

6.10.5.1.3 Vertical tension tie requirements of Sec 8.1.12.3 shall apply to all vertical structural members, except cladding, and shall be achieved by providing connections at horizontal joints in accordance with (a) through (c):

(a) Precast columns shall have a nominal strength in tension not less than $1.4A_g$, in N. For columns with a larger cross section than required by consideration of loading, a reduced effective area A_g(in mm^2), based on cross-section required but not less than one-half the total area, shall be permitted;

(b) Precast wall panels shall have a minimum of two ties per panel, with a nominal tensile strength not less than 44 kN per tie;

(c) When design forces result in no tension at the base, the ties required by Sec 6.10.5.1.3(b) shall be permitted to be anchored into an appropriately reinforced concrete floor slab-on-ground.

6.10.5.1.4 Connection details that rely solely on friction caused by gravity loads shall not be used.

6.10.5.2 For precast concrete bearing wall structures three or more stories in height, the minimum provisions of Sections 6.10.5.2.1 to 6.10.5.2.5 shall apply (Figure 6.6.29).

6.10.5.2.1 Longitudinal and transverse ties shall be provided in floor and roof systems to provide a nominal strength of 22 kN per meter of width or length. Ties shall be provided over interior wall supports and between members and exterior walls. Ties shall be positioned in or within 600 mm of the plane of floor or roof system.

6.10.5.2.2 Longitudinal ties parallel to floor or roof slab spans shall be spaced not more than 3 m on centers. Provisions shall be made to transfer forces around openings.

6.10.5.2.3 Transverse ties perpendicular to floor or roof slab spans shall be spaced not greater than the bearing wall spacing.

6.10.5.2.4 Ties around the perimeter of each floor and roof, within 1.2 m of the edge, shall provide a nominal strength in tension not less than 71 kN.

6.10.5.2.5 Vertical tension ties shall be provided in all walls and shall be continuous over the height of the building. They shall provide a nominal tensile strength not less than 44 kN per horizontal meter of wall. Not less than two ties shall be provided for each precast panel.

Figure 6.6.29 Typical arrangement of tensile ties in large panel structures.

6.10.6 Connection and Bearing Design

6.10.6.1 Forces shall be permitted to be transferred between members by grouted joints, shear keys, mechanical connectors, reinforcing steel connections, reinforced topping, or a combination of these means.

6.10.6.1.1 The adequacy of connections to transfer forces between members shall be determined by analysis or by test. Where shear is the primary result of imposed loading, it shall be permitted to use the provisions of Sec 6.4.5 as applicable.

6.10.6.1.2 When designing a connection using materials with different structural properties, their relative stiffnesses, strengths, and ductilities shall be considered.

6.10.6.2 Bearing for precast floor and roof members on simple supports shall satisfy Sections 6.10.6.2.1 and 6.10.6.2.2.

6.10.6.2.1 The allowable bearing stress at the contact surface between supported and supporting members and between any intermediate bearing elements shall not exceed the bearing strength for either surface or the bearing element, or both. Concrete bearing strength shall be as given in Sec 6.3.14.

6.10.6.2.2 Unless shown by test or analysis that performance will not be impaired, (a) and (b) shall be met (Figure 6.6.30):

(a) Each member and its supporting system shall have design dimensions selected so that, after consideration of tolerances, the distance from the edge of the support to the end of the precast member in the direction of the span is at least $l_n/180$, but not less than:

For solid or hollow-core slabs	50 mm
For beams or stemmed members	75 mm

(b) Bearing pads at unarmored edges shall be set back a minimum of 13 mm from the face of the support, or at least the chamfer dimension at chamfered edges.

6.10.6.2.3 The requirements of Sec 8.2.8.1 shall not apply to the positive bending moment reinforcement for statically determinate precast members, but at least one-third of such reinforcement shall extend to the center of the bearing length, taking into account permitted tolerances in Sections 8.1.5.2c and 6.10.2.3.

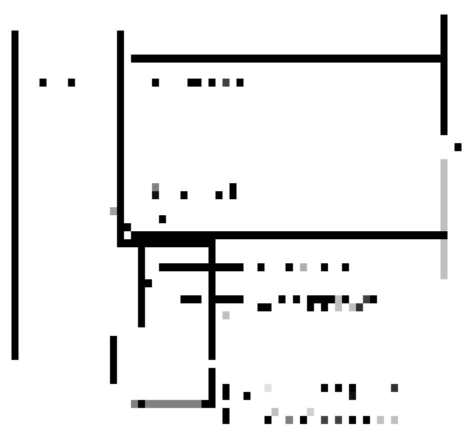


Figure 6.6.30 Bearing length on support

6.10.7 Items Embedded after Concrete Placement

6.10.7.1 When approved by the designer, embedded items (such as dowels or inserts) that either protrude from the concrete or remain exposed for inspection shall be permitted to be embedded while the concrete is in a plastic state provided that Sections 6.10.7.1.1, 6.10.7.1.2, and 6.10.7.1.3 are met.

6.10.7.1.1 Embedded items are not required to be hooked or tied to reinforcement within the concrete.

6.10.7.1.2 Embedded items are maintained in the correct position while the concrete remains plastic.

6.10.7.1.3 The concrete is properly consolidated around the embedded item.

6.10.8 Marking and Identification

6.10.8.1 Each precast member shall be marked to indicate its location and orientation in the structure and date of manufacture.

6.10.8.2 Identification marks shall correspond to placing drawings.

6.10.9 Handling

6.10.9.1 Member design shall consider forces and distortions during curing, stripping, storage, transportation, and erection so that precast members are not overstressed or otherwise damaged.

6.10.9.2 During erection, precast members and structures shall be adequately supported and braced to ensure proper alignment and structural integrity until permanent connections are completed.

6.10.10 Evaluation of Strength of Precast Construction

6.10.10.1 A precast element to be made composite with cast-in-place concrete shall be permitted to be tested in flexure as a precast element alone in accordance with Sections 6.10.10.1.1 and 6.10.10.1.2.

6.10.10.1.1Test loads shall be applied only when calculations indicate the isolated precast element will not be critical in compression or buckling.

6.10.10.1.2The test load shall be that load which, when applied to the precast member alone, induces the same total force in the tension reinforcement as would be induced by loading the composite member with the test load required by Sec 6.11.3.2.

6.10.10.2 The provisions of Sec 6.11.5 shall be the basis for acceptance or rejection of the precast element.

6.11 Evaluation of Strength of Existing Structures

6.11.1 Strength Evaluation - General

6.11.1.1 If there is doubt that a part or all of a structure meets the safety requirements of this Code, a strength evaluation shall be carried out as required by the Engineer.

6.11.1.2 If the effect of the strength deficiency is well understood and if it is feasible to measure the dimensions and material properties required for analysis, analytical evaluations of strength based on those measurements shall suffice. Required data shall be determined in accordance with Sec 6.11.2.

6.11.1.3 If the effect of the strength deficiency is not well understood or if it is not feasible to establish the required dimensions and material properties by measurement, a load test shall be required if the structure is to remain in service.

6.11.1.4 If the doubt about safety of a part or all of a structure involves deterioration, and if the observed response during the load test satisfies the acceptance criteria, the structure or part of the structure shall be permitted to remain in service for a specified time period. If deemed necessary by the Engineer, periodic reevaluations shall be conducted.

6.11.2 Determination of Material Properties and Required Dimensions

6.11.2.1 Dimensions of the structural elements shall be established at critical sections.

6.11.2.2 Locations and sizes of the reinforcing bars, welded wire reinforcement, or tendons shall be determined by measurement. It shall be permitted to base reinforcement locations on available drawings if spot checks are made confirming the information on the drawings.

6.11.2.3 If required, concrete strength shall be based on results of cylinder tests from the original construction or tests of cores removed from the part of the structure where the strength is in question. For strength evaluation of an existing structure, cylinder or core test data shall be used to estimate an equivalent f_c'. The method for obtaining and testing cores shall be in accordance with ASTM C42M.

6.11.2.4 If required, reinforcement or prestressing steel strength shall be based on tensile tests of representative samples of the material in the structure in question.

6.11.2.5 If the required dimensions and material properties are determined through measurements and testing, and if calculations can be made in accordance with Sec 6.11.1.2, it shall be permitted to increase φ from those specified in 6.2.3, but ϕ shall not be more than:

Tension-controlled sections, as defined in 6.3.3.4	1.0
Compression-controlled sections, as defined in Sec 6.3.3.3:	
Members with spiral reinforcement conforming to Sec 6.3.9.3	0.9
Other reinforced members	0.8
Shear and/or torsion	0.8
Bearing on concrete	0.8

6.11.3 Load Test Procedure

6.11.3.1 Load arrangement

The number and arrangement of spans or panels loaded shall be selected to maximize the deflection and stresses in the critical regions of the structural elements of which strength is in doubt. More than one test load arrangement shall be used if a single arrangement will not simultaneously result in maximum values of the effects (such as deflection, rotation, or stress) necessary to demonstrate the adequacy of the structure.

6.11.3.2 Load intensity

The total test load (including dead load already in place) shall not be less than the larger of (a), (b), and (c):

(a) $1.15D + 1.5L + 0.4(L_r or P)$

(b) $1.15D + 0.9L + 1.5(L_r or P)$

(c) $1.3D$

The load factor on the live load L in (b) shall be permitted to be reduced to 0.45 except for garages, areas occupied as places of public assembly, and all areas where, L is greater than 4.8 kN/m^2. It shall be permitted to reduce L in accordance with the provisions of this Code.

6.11.3.3 A load test shall not be made until that portion of the structure to be subjected to load is at least 56 days old. If the owner of the structure, the contractor, and all involved parties agree, it shall be permitted to make the test at an earlier age.

6.11.4 Loading Criteria

6.11.4.1 The initial value for all applicable response measurements (such as deflection, rotation, strain, slip, crack widths) shall be obtained not more than 1 hour before application of the first load increment. Measurements shall be made at locations where maximum response is expected. Additional measurements shall be made if required.

6.11.4.2 Test load shall be applied in not less than four approximately equal increments.

6.11.4.3 Uniform test load shall be applied in a manner to ensure uniform distribution of the load transmitted to the structure or portion of the structure being tested. Arching of the applied load shall be avoided.

6.11.4.4 A set of response measurements shall be made after each load increment is applied and after the total load has been applied on the structure for at least 24 hours.

6.11.4.5 Total test load shall be removed immediately after all response measurements defined in Sec 6.11.4.4 are made.

6.11.4.6 A set of final response measurements shall be made 24 hours after the test load is removed.

6.11.5 Acceptance Criteria

6.11.5.1 The portion of the structure tested shall show no evidence of failure. Spalling and crushing of compressed concrete shall be considered an indication of failure.

6.11.5.2 Measured deflections shall satisfy either Eq. (6.6.101) or (6.6.102):

$$\Delta_1 \leq \frac{l_t^2}{20{,}000h} \quad (6.6.101)$$

$$\Delta_r \leq \frac{\Delta_1}{4} \quad (6.6.102)$$

If the measured maximum and residual deflections, Δ_1 and Δ_r, do not satisfy Eq. (6.6.101) or (6.6.102), it shall be permitted to repeat the load test.

The repeat test shall be conducted not earlier than 72 hours after removal of the first test load. The portion of the structure tested in the repeat test shall be considered acceptable if deflection recovery Δ_r satisfies the condition:

$$\Delta_r \leq \frac{\Delta_2}{5} \quad (6.6.103)$$

Where, Δ_2 is the maximum deflection measured during the second test relative to the position of the structure at the beginning of the second test.

6.11.5.3 Structural members tested shall not have cracks indicating the imminence of shear failure.

6.11.5.4 In regions of structural members without transverse reinforcement, appearance of structural cracks inclined to the longitudinal axis and having a horizontal projection longer than the depth of the member at midpoint of the crack shall be evaluated.

6.11.5.5 In regions of anchorage and lap splices, the appearance along the line of reinforcement of a series of short inclined cracks or horizontal cracks shall be evaluated.

6.11.6 Provision for Lower Load Rating

If the structure under investigation does not satisfy conditions or criteria of Sec 6.11.1.2, Sec 6.11.5.2, or Sec 6.11.5.3, the structure shall be permitted for use at a lower load rating based on the results of the load test or analysis, if approved by the Engineer.

6.11.7 Safety

6.11.7.1 Load tests shall be conducted in such a way as to provide for safety of life and structure during test.

6.11.7.2 Safety measures shall not interfere with load test procedures or affect results.

6.12 Composite Concrete Flexural Members

6.12.1 Scope

6.12.1.1 Provisions of Sec 6.12 shall apply for design of composite concrete flexural members defined as precast concrete, cast-in-place concrete elements, or both, constructed in separate placements but so interconnected that all elements respond to loads as a unit.

6.12.1.2 All provisions of the Code shall apply to composite concrete flexural members, except as specifically modified in Sec 6.12.

6.12.2 General

6.12.2.1 The use of an entire composite member or portions thereof for resisting shear and moment shall be permitted.

6.12.2.2 Individual elements shall be investigated for all critical stages of loading.

6.12.2.3 If the specified strength, unit weight, or other properties of the various elements are different, properties of the individual elements or the most critical values shall be used in design.

6.12.2.4 In strength computations of composite members, no distinction shall be made between shored and unshored members.

6.12.2.5 All elements shall be designed to support all loads introduced prior to full development of design strength of composite members.

6.12.2.6 Reinforcement shall be provided as required to minimize cracking and to prevent separation of individual elements of composite members.

6.12.2.7 Composite members shall meet requirements for control of deflections in accordance with Sec 6.2.5.4.

6.12.3 Shoring

When used, shoring shall not be removed until supported elements have developed design properties required to support all loads and limit deflections and cracking at time of shoring removal.

6.12.4 Vertical Shear Strength

6.12.4.1 Where an entire composite member is assumed to resist vertical shear, design shall be in accordance with requirements of Sec 6.4 as for a monolithically cast member of the same cross-sectional shape.

6.12.4.2 Shear reinforcement shall be fully anchored into interconnected elements in accordance with Sec 8.2.10.

6.12.4.3 Extended and anchored shear reinforcement shall be permitted to be included as ties for horizontal shear.

6.12.5 Horizontal Shear Strength

6.12.5.1 In a composite member, full transfer of horizontal shear forces shall be ensured at contact surfaces of interconnected elements.

6.12.5.2 For the provisions of Sec 6.12.5, d shall be taken as the distance from extreme compression fiber for entire composite section to centroid of longitudinal tension reinforcement, if any.

6.12.5.3 Unless calculated in accordance with Sec 6.12.5.4, design of cross sections subject to horizontal shear shall be based on

$$V_u \leq \phi V_{nh} \tag{6.6.104}$$

Where, V_{nh} is nominal horizontal shear strength in accordance with Sections 6.12.5.3.1 to 6.12.5.3.4.

6.12.5.3.1 Where contact surfaces are clean, free of laitance, and intentionally roughened, V_{nh} shall not be taken greater than $0.55 b_v d$.

6.12.5.3.2 Where minimum ties are provided in accordance with Sec 6.12.6, and contact surfaces are clean and free of laitance, but not intentionally roughened, V_{nh} shall not be taken greater than $0.55 b_v d$.

6.12.5.3.3 Where ties are provided in accordance with Sec 6.12.6, and contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately 6 mm, V_{nh} shall be taken equal to, $(1.8+0.6\rho_v f_v)\lambda b_v d$, but not greater than $3.5 b_v d$. Values for λ in Sec 6.4.5.4.3 shall apply and ρ_v is $A_v/(b_v s)$.

6.12.5.3.4 Where V_u at section considered exceeds $\phi(3.5 b_v d)$, design for horizontal shear shall be in accordance with Sec 6.4.5.4.

6.12.5.4 As an alternative to Sec 6.12.5.3, horizontal shear shall be permitted to be determined by computing the actual change in compressive or tensile force in any segment, and provisions shall be made to transfer that force as horizontal shear to the supporting element. The factored horizontal shear force V_u shall not exceed horizontal shear strength ϕV_{nh} as given in Sections 6.12.5.3.1 to 6.12.5.3.4, where area of contact surface shall be substituted for $b_v d$.

6.12.5.4.1 Where ties provided to resist horizontal shear are designed to satisfy Sec 6.12.5.4, the tie area to tie spacing ratio along the member shall approximately reflect the distribution of shear forces in the member.

6.12.5.5 Where tension exists across any contact surface between interconnected elements, shear transfer by contact shall be permitted only when minimum ties are provided in accordance with Sec 6.12.6.

6.12.6 Ties for Horizontal Shear

6.12.6.1 Where ties are provided to transfer horizontal shear, tie area shall not be less than that required by Sec 6.4.3.5.3, and tie spacing shall not exceed four times the least dimension of supported element, nor exceed 600 mm.

6.12.6.2 Ties for horizontal shear shall consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire reinforcement.

6.12.6.3 All ties shall be fully anchored into interconnected elements in accordance with Sec 8.2.10.

6.13 List of Related Appendices

Appendix I Strut-and-Tie Models

Appendix J Working Stress Design Method for Reinforced Concrete Structures

Appendix K Anchoring to Concrete

Appendix L Information on Steel Reinforcement

Appendix M Special Types of Stairs

PART VI
Chapter 7
Masonry Structures

7.1 Introduction

7.1.1 Scope

This Chapter of the Code covers the design, construction and quality control of masonry structures.

7.1.2 Definitions

For the purpose of this Chapter, the following definitions shall be applicable.

ACTUAL DIMENSIONS	The measured dimensions of a designated item; such as a designated masonry unit or wall used in the structures. The actual dimension shall not vary from the specified dimension by more than the amount allowed in the appropriate standard mentioned in Sec 2.2.4 Chapter 2 Part 5.
BED BLOCK	A block bedded on a wall, column or pier to disperse a concentrated load on a masonry element.
BED JOINT	A horizontal mortar joint upon which masonry units are placed.
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.
BOND BEAM	A horizontal grouted element within masonry in which reinforcement is embedded.
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 and conforming to the requirement of Sec 4.3.3(c) (ii).
CAVITY WALL	A wall comprising two limbs each built-up as single or multi-wythe units and separated by a 50-115 mm wide cavity. The limbs are tied together by metal ties or bonding units for structural integrity.

CELL	A void space having a gross cross-sectional area greater than 1000 mm^2.
COLLAR JOINT	The vertical, longitudinal, mortar or grouted joints.
COLUMN	An isolated vertical load bearing member the width of which does not exceed three times the thickness.
CROSS JOINT	A vertical joint normal to the face of the wall.
CROSS-SECTIONAL AREA OF MASONRY UNIT	Net cross-sectional area of masonry unit is the gross cross-sectional area minus the area of cellular space.
CURTAIN WALL	A non-load bearing self-supporting wall subject to transverse lateral loads, and laterally supported by vertical or horizontal structural member where necessary.
FACED WALL	A wall in which facing and backing of two different materials are bonded together to ensure common action under load.
GROUT	A mixture of cementitious materials and aggregate to which water is added such that the mixture will flow without segregation of the constituents.
GROUTED HOLLOW-UNIT MASONRY	That form of grouted masonry construction in which certain designated cells of hollow units are continuously filled with grout.
GROUTED MULTI-WYTHE MASONRY	That form of grouted masonry construction in which the space between the wythes is solidly or periodically filled with grout.
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.
JAMB	Side of an opening in wall.
HEAD JOINT	The mortar joint having a vertical transverse plane.
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.
LIMB	Inner or outer portion of a cavity wall.
LOAD BEARING WALL	A wall designed to carry an imposed vertical load in addition to its own weight, together with any lateral load.

MASONRY	An assemblage of masonry units properly bonded together with mortar.
MASONRY UNIT	Individual units, such as brick, tile, stone or concrete block, which are bonded together with mortar to form a masonry element such as walls, columns, piers, buttress, etc.
NOMINAL DIMENSIONS	Specified dimensions plus the thickness of the joint with which the unit is laid.
PANEL WALL	An exterior non-load bearing wall in framed structure, supported at each storey but subject to lateral loads.
PARTITION WALL	An interior non-load bearing wall, one storey or part storey in height.
PIER	A projection from either or both sides of a wall forming an integral part of the wall and conforming to the requirement of Sec 7.4.3.3 of this Chapter.
PILASTER	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 bounded into a limb of cavity wall, the thickness obtained by treating that limb as an independent wall.
PRISM	An assemblage of masonry units bonded by mortar with or without grout used as a test specimen for determining properties of masonry.
REINFORCED MASONRY	The masonry construction, in which reinforcement acting in conjunction with the masonry is used to resist forces and is designed in accordance with Sec 7.6 of this Chapter.
SHEAR WALL	A load bearing wall designed to carry horizontal forces acting in its own plane with or without vertical imposed loads.
SOLID UNIT	A masonry unit whose net cross-sectional area in any plane parallel to the bearing surface is 75 percent or more of the gross cross-sectional area in the same plane.
SPECIFIED DIMENSIONS	The dimensions specified for the manufacture or construction of masonry, masonry units, joints or any other components of a structure. Unless otherwise stated, all calculations shall be made using or based on specified dimensions.

STACK BOND	A bond in bearing and nonbearing walls, except veneered walls, in which less than 75 percent of the units in any transverse vertical plane lap the ends of the units below a distance less than one-half the height of the unit, or less than one-fourth the length of the unit.
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.
WALL JOINT	A vertical joint parallel to the face of the wall.
WALL TIE	A metal fastener which connects wythes of masonry to each other or to other materials.
WYTHE	Portion of a wall which is one masonry unit in thickness.

7.1.3 Symbols and Notation

The following units shall be generally implicit in this Chapter for the corresponding quantities:

Lengths	mm
Areas	mm^2
Moment of inertia	mm^4
Force	N
Moment, torsion	N mm
Stress, strength	N/mm^2

A_b = Cross-sectional area of anchor bolt

A_e = Effective area of masonry

A_g = Gross area of wall

A_{mv} = Net area of masonry section bounded by wall thickness and length of section in the direction of shear force considered

A_p = Area of tension (pullout) cone of an embedded anchor bolt projected into the surface of masonry

A_s = Effective cross-sectional area of reinforcement in a flexural member

A_v = Area of steel required for shear reinforcement perpendicular to the longitudinal reinforcement

A'_s	=	Effective cross-sectional area of compression reinforcement in a flexural member
B_t	=	Allowable tension force on anchor bolt
B_v	=	Computed shear force on anchor bolt
C_d	=	Masonry shear strength coefficient
E_m	=	Modulus of elasticity of masonry
E_s	=	Modulus of elasticity of steel
F	=	Loads due to weight and pressure of fluids or related moments and forces
F_a	=	Allowable average axial compressive stress for centroidally applied axial load only
F_b	=	Allowable flexural compressive stress if members were carrying bending load only
F_{br}	=	Allowable bearing stress
F_s	=	Allowable stress in reinforcement
F_{sc}	=	Allowable compressive stress in column reinforcement
F_t	=	Allowable flexural tensile stress in masonry
F_v	=	Allowable shear stress in masonry
G	=	Shear modulus of masonry
H	=	Actual height between lateral supports
H'	=	Height of opening
I	=	Moment of inertia about the neutral axis of the cross-sectional area
I_g, I_{cr}	=	Gross, cracked moment of inertia of the wall cross-section
L	=	Actual length of wall
M	=	Design moment
M_c	=	Moment capacity of the compression steel in a flexural member about the centroid of the tensile force
M_{cr}	=	Cracking moment strength of the masonry wall
M_m	=	The moment of the compressive force in the masonry about the centroid of the tensile force in the reinforcement

M_n	=	Nominal moment strength of the masonry wall
M_s	=	The moment of the tensile force in the reinforcement about the centroid of the compressive force in the masonry
M_{ser}	=	Service moment at the mid-height of the panel, including P-Delta effects
M_u	=	Factored moment
P	=	Design axial load
P_a	=	Allowable centroidal axial load for reinforced masonry columns
P_b	=	Nominal balanced design axial strength
P_f	=	Load from tributary floor or roof area
P_o	=	Nominal axial load strength with bending
P_u	=	Factored axial load
P_{uf}	=	Factored load from tributary floor or roof loads
P_{uw}	=	Factored weight of the wall tributary to the section under consideration
P_w	=	Weight of the wall tributary to the section under consideration
S	=	Section modulus
V	=	Total design shear force
V_m	=	Nominal shear strength provided by masonry
V_n	=	Nominal shear strength
V_s	=	Nominal shear strength provided by shear reinforcement
a	=	Depth of equivalent rectangular stress block for strength design
b	=	Effective width of rectangular member or width of flange for T and I section
b_t	=	Computed tension force on anchor bolt
b_v	=	Allowable shear force on anchor bolt
b_w	=	Width of web in T and I member
c	=	Distance from the neutral axis to extreme fibre
d	=	Distance from the compression face of a flexural member to the centroid of longitudinal tensile reinforcement

d_b	=	Diameter of the reinforcing bar, diameter of bolt
e	=	Eccentricity of P_u
e_{mu}	=	Maximum usable compressive strain of masonry
f_a	=	Computed axial compressive stress due to design axial load
f_b	=	Computed flexural stress in the extreme fibre due to design bending load only
f_{md}	=	Computed compressive stress in masonry due to dead load only
f_r	=	Modulus of rupture
f_s	=	Computed stress in reinforcement due to design load
f_y	=	Tensile yield stress of reinforcement
f_v	=	Computed shear stress due to design load
f'_m	=	Specified compressive strength of masonry at the age of 28 days
h	=	Height of wall between points of support
h'	=	Effective height of a wall or column
j	=	Ratio or distance between centroid of flexural compressive force and centroid of tensile forces to depth, d
k	=	Ratio of depth of the compression zone in flexural member to depth, d; stiffening coefficient
l	=	Length of a wall or segment
l_b	=	Embedment depth of anchor bolt
l_{be}	=	Anchor bolt edge distance, the least length measured from the edge of masonry to the surface of the anchor bolt
l_d	=	Required development length of reinforcement
n	=	Modular ratio $= E_s/E_m$
r_b	=	Ratio of the area of bars cut off to the total area of bars at the section
s	=	Spacing of stirrups or bent bars in a direction parallel to that of the main reinforcement
t	=	Effective thickness of a wythe, wall or column

u = Bond stress per unit of surface area of bar

Δ_u = Horizontal deflection at mid-height under factored load; P-Delta effects shall be included in deflection calculation

Σ_o = Sum of the perimeters of all the longitudinal reinforcement

ρ = Steel ratio = A_s/bd

ρ_n = Ratio of distributed shear reinforcement on a plane perpendicular to the plane of A_{mv}

ϕ = Strength reduction factor.

7.2 Materials

7.2.1 General

All materials used in masonry construction shall conform to the requirements specified in Part 5 of this Code. If no requirements are specified for a material, quality shall be based on generally accepted good practice, subject to the approval of the building official.

7.2.2 Masonry Units

The following types of masonry units which conform to the standards mentioned in Sec 2.2.4 of Part 5 may be used in masonry construction:

(a) Common building clay bricks

(b) Burnt clay hollow bricks

(c) Burnt clay facing bricks

(d) Hollow concrete blocks

Other types of masonry units conforming to Sec 2.2.4 of Part 5 may also be used.

7.2.3 Mortar and Grout

Mortar and grout for masonry construction shall conform to the requirements specified in Part 5 of this Code. Mix proportions and compressive strength of some commonly used mortars are given in Table 6.7.1.

7.3 Allowable Stresses

7.3.1 General

Stresses in masonry shall not exceed the values given in this Section. All allowable stresses for working stress design may be increased one third when considering wind or earthquake forces either acting alone or combined with vertical loads. No increase shall be allowed for vertical loads acting alone.

7.3.2 Specified Compressive Strength of Masonry, f'_m

The allowable stresses for masonry construction shall be based on the value of f'_m as determined by Sec 7.3.3 below.

Table 6.7.1: Mix Proportion and Strength of Commonly used Mortars

Grade of Mortar	Mix Proportion by Volume [1, 2]		Minimum Compressive Strength at 28 days, N/mm^2
	Cement	Sand	
M1	1	3	10
M2		4	7.5
M3		5	5
M4		6	3
M5		7	2
M6		8	1

1 Sand and cement shall be measured in loose volume and sand shall be well graded with a minimum F.M. of 1.20

2 Lime to a maximum of one fourth (1/4) part by volume of cement may be used to increase workability.

7.3.3 Compliance with f'_m

Compliance with the requirements for the specified compressive strength of masonry, f'_m shall be in accordance with the following:

7.3.3.1 Masonry Prism Testing: The compressive strength of masonry based on tests at 28 days in accordance with "Standard Test Method for Compressive Strength of Masonry Prisms", (ASTM E447) for each set of prisms shall equal or exceed f'_m. Verification by masonry prism testing shall meet the following :

(a) Testing Prior to Construction: A set of five masonry prisms shall be built and tested in accordance with ASTM E447 prior to the start of construction. Materials used for prisms shall be same as used in the project. Prisms shall be constructed under the observation of the engineer or an approved agency and tested by an approved agency.

(b) Testing During Construction: When full allowable stresses are used in design, a set of three prisms shall be built and tested during construction in accordance with (ASTM E447) for each 500 square meters of wall area, but not less than one set of three masonry prisms for any project. No testing during construction shall be required when 50% of the allowable stresses are used in design.

7.3.4 Quality Control

Quality control shall include, but not be limited to assure that:

(a) Masonry units, reinforcement, cement, lime, aggregate and all other materials meet the requirements of the applicable standard of quality and that they are properly stored and prepared for use.

(b) Mortar and grout are properly mixed using specified proportions of ingredients. The method of measuring materials for mortar and grout shall be such that proportions of materials are controlled.

(c) Construction details, procedures and workmanship are in accordance with the plans and specification.

(d) Placement, splices and bar diameters are in accordance with the provisions of this Chapter and the plans and specifications.

7.3.5 Allowable Stresses in Masonry

When the quality control provisions specified in Sec 7.3.4 above do not include requirements for special inspection, the allowable design stresses in this Section shall be reduced by 50 percent.

(a) Axial Compressive Stress

(i) Unreinforced masonry walls, columns and reinforced masonry wall

$$F_a = \frac{f'_m}{5}\left[1 - \left(\frac{h'}{42t}\right)^3\right] \tag{6.7.1}$$

(ii) Reinforced masonry columns

$$F_a = \left(\frac{f'_m}{5} + \frac{A_s}{1.5A_g}F_{sc}\right)\left[1 - \left(\frac{h'}{42t}\right)^3\right] \tag{6.7.2}$$

(b) Compressive Stress in Flexural

$$F_b = 0.33 f'_m \leq 10 \text{ N/mm}^2 \tag{6.7.3}$$

(c) Tensile Stress of Walls in Flexure

The allowable tensile stress for walls in flexure of masonry structures without tensile reinforcement using mortar Type M_1 or M_2 shall not exceed the values specified in Tables 6.7.2 and 6.7.3. For Types M_3 and M_4 mortar, the values shall be reduced by 25 percent.

No tension is allowed across head joints in stack bond masonry. Values for tension normal to head joints are for running bond. These values shall not be used for horizontal flexural members such as beams, girders or lintels.

Table 6.7.2: Flexural Tension, Ft

Masonry	Normal to Bed Joints N/mm^2	Normal to Head Joints N/mm^2
Solid Units	0.20	0.40
Hollow Units	0.12	0.25

Table 6.7.3: Tension Normal to Head Joints, Ft

Masonry	Clay Units N/mm^2	Concrete Units N/mm^2
Solid Units	0.35	0.40
Hollow Units	0.22	0.25

(d) Reinforcing Bond Stress, u

Plain Bars: 0.30 N/mm^2

Deformed Bars: 1.0 N/mm^2

(e) Shear Stress for Flexural Members, F_v

(i) When no shear reinforcement is used

$$F_v = 0.083\sqrt{f_m'} \leq 0.25 \text{ N/mm}^2 \qquad (6.7.4)$$

(ii) When shear reinforcement is designed to take entire shear force

$$F_v = 0.25\sqrt{f_m'} \leq 0.75 \text{ N/mm}^2 \qquad (6.7.5)$$

(f) Shear Stress for Shear Walls, F_v

(i) Unreinforced masonry

For clay units:

$$F_v = 0.025\sqrt{f_m'} \leq 0.40 \text{ N/mm}^2 \qquad (6.7.6)$$

For concrete units:

M_1 or M_2 Mortar: 0.20 N/mm²

M_3 Mortar: 0.12 N/mm²

(ii) The allowable shear stress for reinforced masonry shear walls shall be according to Table 6.7.4.

Table 6.7.4: Allowable Shear Stress for Reinforced Masonry Shear Walls, F_v

Masonry Wall	M/Vd	F_v, N/mm²	Maximum Allowable N/mm²
Masonry taking all shear	< 1	$\frac{1}{36}\left(4-\frac{M}{Vd}\right)\sqrt{f'_m}$	$\left(0.4-0.2\frac{M}{Vd}\right)$
	≥ 1	$0.083\sqrt{f'_m}$	0.17
Reinforcement taking all shear	< 1	$\frac{1}{24}\left(4-\frac{M}{Vd}\right)\sqrt{f'_m}$	$\left(0.6-0.2\frac{\mathrm{M}}{\mathrm{Vd}}\right)$
	≥ 1	$0.125\sqrt{f'_m}$	0.37

7.3.6 Allowable Stresses in Reinforcement

(a) Tensile Stress

(i) Deformed bars,

$$F_s = 0.5f_y \quad \leq 165 \ \text{N/mm}^2 \tag{6.7.7}$$

(ii) Ties, anchors and plain bars,

$$F_s = 0.4f_y \quad \leq 135 \ \text{N/mm}^2 \tag{6.7.8}$$

(b) Compressive Stress

(i) Deformed bars in columns and shear walls,

$$F_{sc} = 0.4f_y \quad \leq 165 \ \text{N/mm}^2 \tag{6.7.9}$$

(ii) Deformed bars in flexural members

$$F_{sc} = 0.5f_y \quad \leq 165 \ \text{N/mm}^2 \tag{6.7.10}$$

7.3.7 Combined Compressive Stress

Members subject to combined axial and flexural stresses shall be designed in accordance with accepted principles of mechanics or in accordance with the following formula:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \tag{6.7.11}$$

7.3.8 Modulus of Elasticity

The modulus of elasticity of masonry shall be determined by the secant method. The slope of the line connecting the points $0.05f'_m$ and $0.33f'_m$ on the stress-strain curve shall be taken as the modulus of elasticity of masonry. If required, actual values shall be established by tests. These values are not to be reduced by 50 per cent as specified in Sec 7.3.5(a).

(a) Modulus of Elasticity for Masonry

$$E_m = 750f'_m \leq 15{,}000 \text{ N/mm}^2 \qquad (6.7.12)$$

(b) Modulus of Elasticity for Steel

$$E_s = 2{,}00{,}000 \text{ N/mm}^2 \qquad (6.7.13)$$

(c) Shear Modulus of Masonry

$$G = 0.4E_m \text{ N/mm}^2 \qquad (6.7.14)$$

7.3.9 Shear and Tension on Embedded Anchor Bolts

7.3.9.1 Allowable loads and placement requirements for anchor bolts shall be in accordance with the following:

(a) Bent bar anchor bolts shall have a hook with a 90° bend with an inside diameter of $3d_b$ plus an extension of $1.5d_b$ at the free end.

(b) Headed anchor bolts shall have a standard bolt head.

(c) Plate anchor bolts shall have a plate welded to the shank to provide anchorage equivalent to headed anchor bolts.

7.3.9.2 The effective embedment length, l_b for bent bar anchors shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the bent end minus one anchor bolt diameter. For plate or headed anchor bolts l_b shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the plate or head of the anchorage. All bolts shall be grouted in place with at least 25 mm of grout between the bolt and the masonry except that 6 mm diameter bolts may be placed in bed joints which are at least twice as thick as the diameter of the bolt.

7.3.9.3 Allowable shear force

Allowable loads in shear shall be according to Table 6.7.5 or lesser of the value obtained from the following formulae:

$$B_v = 1070(f'_m A_b)^{1/4} \qquad (6.7.15)$$

$$B_v = 0.12A_b f_y \qquad (6.7.16)$$

When the distance l be is less than $12d_b$, the value of B_v in Eq. 6.7.15 shall be reduced to zero at a distance l_{be} equal to 40 mm. Where adjacent anchors are spaced closer than $8d_b$, the allowable shear of the adjacent anchors determined by Eq. 6.7.15 shall be reduced by interpolation to 0.75 times the allowable shear value at a centre to centre spacing of $4d_b$.

7.3.9.4 Allowable tension

Allowable tension shall be the lesser value selected from Table 6.7.6 and Table 6.7.7 or shall be determined from lesser of the values obtained from the following formulae:

$$B_t = 0.04A_p\sqrt{f'_m} \qquad (6.7.17)$$

$$B_t = 0.2A_b f_y \qquad (6.7.18)$$

The area A_p shall be the lesser of the area obtained from Equations 6.7.17 and 6.7.18 and where the projected areas of adjacent anchor bolts overlap, A_p of each anchor bolt shall be reduced by 50 percent of the overlapping area.

$$A_p = \pi l_b^2 \qquad (6.7.19)$$

$$A_p = \pi l_{be}^2 \qquad (6.7.20)$$

Table 6.7.5: Allowable Shear, B_v for Embedded Anchor Bolts for Masonry, kN*

	Bent Bar Anchor Bolt Diameter, mm						
f'_m N/mm^2	10	12	16	20	22	25	28
10	2.0	3.7	5.9	7.9	8.5	9.1	9.6
12	2.0	3.7	5.9	8.2	8.3	9.5	10.1
13	2.0	3.7	5.9	8.5	9.2	9.8	10.4
17	2.0	3.7	5.9	8.5	9.7	10.3	11.0
20	2.0	3.7	5.9	8.5	10.1	10.8	11.5
27	2.0	3.7	5.9	8.5	10.9	11.6	12.3

* Values are for bolts of at least ASTM A307 quality. Bolts shall be those specified in Sec 4.3.9.1.

Table 6.7.6: Allowable Tension, B_t for Embedded Anchor Bolts for Masonry, kN[1, 2]

	Embedment Length l_b, or Edge Distance, l_{be} mm						
f'_m N/mm²	**50**	**75**	**100**	**125**	**150**	**200**	**250**
10	1.0	2.4	4.3	6.7	9.7	17.3	27.0
12	1.2	2.6	4.7	7.4	10.6	18.9	29.6
13	1.2	2.8	5.0	7.8	11.2	20.0	31.2
17	1.3	3.1	5.6	8.7	12.6	22.4	35.0
20	1.5	3.4	6.7	9.5	13.8	24.5	38.2
27	1.7	3.9	7.0	11.0	15.9	28.3	44.1

1 The allowable tension values are based on compressive strength of masonry assemblages. Where yield strength of anchor bolt steel governs, the allowable tension is given in Table 6.7.7.

2 Values are for bolts of at least ASTM A307 quality. Bolts shall be those specified in Sec 7.3.9.1.

Table 6.7.7: Allowable Tension, B_t for Embedded Anchor Bolts for Masonry, kN[1]

Bent Bar Anchor Bolt Diameter, mm							
6	10	12	16	20	22	25	28
1.5	3.5	6.2	9.8	14.1	19.2	25.1	31.8

1 Values are for bolts of at least ASTM A307 quality. Bolts shall be those specified in Sec 7.3.9.

7.3.9.5 Combined shear and tension

Anchor bolts subjected to combined shear and tension shall be designed in accordance with the formula given below:

$$\frac{b_t}{B_t} + \frac{b_v}{B_v} \leq 1.00 \qquad (6.7.21)$$

7.3.9.6 Minimum edge distance, l_{be}

The minimum value of l_{be} measured from the edge of the masonry parallel to the anchor bolt to the surface of the anchor bolt shall be 40 mm.

7.3.9.7 Minimum embedment depth, l_b

The minimum embedment depth l_b shall be $4d_b$ but not less than 50 mm.

7.3.9.8 Minimum spacing between bolts

The minimum centre to centre spacing between anchors shall be $4d_b$.

7.3.10 Load Test

For load test, the member shall be subject to a superimposed load equal to twice the design live load plus one-half of the dead load. This load shall be maintained for a period of 24 hours. If, during the test or upon removal of the load, the member shows evidence of failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or where possible, a lower rating shall be established. A flexural member shall be considered to have passed the test if the maximum deflection at the end of the 24 hour period neither exceeds $0.005l$ nor $0.00025\ l^2/t$ and the beam and slabs show a recovery of at least 75 percent of the observed deflection within 24 hours after removal of the load.

7.3.11 Reuse of Masonry Units

Masonry units may be reused when clean, unbroken and conforms to the requirements of Part 5. All structural properties of masonry of reclaimed units, especially adhesion bond, shall be determined by approved test. The allowable working stress shall not exceed 50 percent of that permitted for new masonry units of the same properties.

7.4 Basic Design Requirements

7.4.1 General

Masonry structures shall be designed according to the provisions of this Section. The required design strengths of masonry materials and any special requirements shall be specified in the plan submitted for approval.

7.4.2 Design Considerations

7.4.2.1 Masonry structures shall be designed based on working stress and linear stress-strain distribution. Requirements for working stress design of unreinforced and reinforced masonry structures are provided in Sections 4.5 and 4.6 respectively. In lieu of the working stress design method, slender walls and shear walls may be designed by the strength design method specified in Sec 7.7.

The structure shall be proportioned such that eccentricity of loading on the members is as small as possible. Eccentric loading shall preferably be avoided by providing:

(a) adequate bearing of floor/roof on the walls

(b) adequate stiffness in slabs, and

(c) fixity at the supports.

7.4.2.2 Effective height

(a) Wall: The effective height of a wall shall be taken as the clear height between the lateral supports at top and bottom in a direction normal to the axis considered. For members not supported at the top normal to the axis considered, the effective height is twice the height of the member above the support. Effective height less than the clear height may be used if justified.

(b) Column: Effective height of the column 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 at the top normal to the axis considered.

(c) Opening in Wall: 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 obtained as follows:

(i) When wall has full restraint at the top, effective height for the direction perpendicular to the plane of wall equals $0.75H$ plus $0.25H'$, where H is the distance between supports and H' is the height of the taller opening; and effective height for the direction parallel to the wall equals H.

(ii) When wall has partial restraint at the top and bottom, effective height for the direction perpendicular to the plane of wall equals H when height of neither opening exceeds $0.5H$ and it is equal to $2H$ when height of any opening exceeds$0.5H$; and effective height for the direction parallel to the plane of the wall equals $2H$.

7.4.2.3 Effective length

Effective length of a wall for different support conditions shall be as given in Table 6.7.8.

7.4.2.4 Effective thickness

The effective thickness of walls and columns for use in the calculation of slenderness ratio, shall be defined as follows:

(a) Solid Walls: The effective thickness of solid walls, faced walls or grouted walls shall be the specified thickness of the wall.

(b) Solid Walls with Raked Mortar Joints: The effective thickness of solid walls with raked mortar joints shall be the minimum thickness measured at the joint.

(c) Cavity Walls: When both limbs of a cavity wall are axially loaded, each limb shall be considered independently and the effective thickness of each limb shall be determined as in (a) or (b) above. If one of the limbs is axially loaded, the effective thickness of the cavity wall shall be taken as the square root of the sum of the squares of the effective thicknesses of the limbs.

(d) Walls Stiffened by Pilasters: When solid or cavity walls are stiffened by pilasters at intervals, the effective thickness to be used for the calculation of h'/t ratio shall be determined as follows:

(i) Solid Walls: For stiffened solid walls the effective thickness shall be the specified thickness multiplied by the stiffening coefficient, k, values of which are given below:

l_p/w_p	Stiffening Coefficient, k*		
	$t_p/t_w = 1$	$t_p/t_w = 2$	$t_p/t_w = 3$
6	1.0	1.4	2.0
8	1.0	1.3	1.7
10	1.0	1.2	1.4
15	1.0	1.1	1.2
20 or more	1.0	1.0	1.0

* Linear interpolation is permitted for obtaining intermediate values of k

Where, l_p = centre to centre spacing of pilasters

t_p = thickness of pilaster including the wall

t_w = specified thickness of main wall

w_p = width of pilaster in the direction of wall

(ii) Cavity Walls: When one or both limbs of a cavity wall are adequately bonded into pilasters at intervals, the effective thickness of each limb shall be determined separately as in (a), (b) or d above and the effective thickness of the stiffened cavity wall shall be determined in accordance with (c) above.

Where slenderness ratio of the wall is based on the effective length, the effective thickness shall be the same as that without pilasters.

(e) Columns: The effective thickness for rectangular columns in the direction considered is the actual thickness provided in that direction. The effective thickness for nonrectangular columns is the thickness of a square column with the same moment of inertia about its axis as that about the axis considered in the actual column.

Table 6.7.8: Effective Length of Walls

Support Condition	Effective Length
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. OR, Where a wall is continuous and is supported by pier/buttresses conforming to Sec 7.4.3.3 (c).	$0.8L$
Where a wall is supported by cross wall at one end and continuous with cross wall at other end. OR, Where a wall is supported by pier/buttresses at one end and continuous with pier/buttresses at other end conforming to Sec 7.4.3.3 (c).	$0.9L$
Where a wall is supported at each end by cross wall. OR, Where a wall is supported at each end by pier/buttresses conforming to Sec 7.4.3.3 (c).	$1.0L$
Where a wall is free at one end and continuous with a cross wall at the other end. OR, Where a wall is free at one end and continuous with a pier/buttresses at the other end conforming to Sec 7.4.3.3 (c).	$1.5L$
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/buttresses conforming to Sec 7.4.3.3 (c).	$2.0L$

7.4.2.5 Slenderness ratio

(a) Walls: For a wall, slenderness ratio shall be the ratio of effective height to effective thickness or effective length to effective thickness whichever less is. In case of a load bearing wall, slenderness ratio shall not exceed 20.

(b) Column: For a column, slenderness ratio shall be taken to be the greater of the ratio of effective heights to the respective effective thickness in the two principal directions. Slenderness ratio for a load bearing column shall not exceed 12.

7.4.2.6 Effective area

The effective cross-sectional area shall be based on the minimum bedded area of the hollow units, or the gross area of solid units plus any grouted area. If hollow units are used perpendicular to the direction of stress, the effective area shall be lesser of the minimum bedded area or the minimum cross-sectional area. If bed joints are raked, the effective area shall be correspondingly reduced. Effective areas for cavity walls shall be that of the loaded wythes.

7.4.2.7 Flexural resistance of cavity walls

For computing the flexural resistance, lateral loads perpendicular to the plane of the wall shall be distributed to the wythes according to their respective flexural rigidities.

7.4.2.8 Effective width of intersecting walls

Where a shear wall is anchored to an intersecting wall or walls, the width of the overhanging flange formed by the intersected walls on either side of the shear wall shall not exceed 6 times the thickness of the intersected wall. Limits of the effective flange may be waived if justified. Only the effective area of the wall parallel to the shear forces may be assumed to carry horizontal shear.

7.4.3 Supports

7.4.3.1 Vertical support

Structural members providing vertical support of masonry shall provide a bearing surface on which the initial bed joint shall not be less than 6 mm or more than 25 mm and shall be of noncombustible materials, except where masonry is a nonstructural decorative feature or wearing surface.

7.4.3.2 Vertical deflection

Elements supporting masonry shall be designed so that their vertical deflection does not exceed 1/600 of the clear span under total loads. Lintels shall be supported on each end such that allowable stresses in the supporting masonry are not exceeded. The minimum bearing length shall be 100 mm.

7.4.3.3 Lateral support

(a) Lateral support of masonry may be provided by cross walls, columns, piers, counter forts or buttresses when spanning horizontally or by floors, beams or roofs when spanning vertically.

(b) Lateral supports for a masonry element such as load bearing wall or column shall be provided to

  (i) limit the slenderness of a masonry element so as to prevent or reduce possibility of buckling of the member due to vertical loads; and

  (ii) resist the horizontal components of forces so as to ensure stability of a structure against overturning.

(c) From consideration of slenderness (i.e. requirement b(i) above), masonry elements may be considered to be laterally supported if

  (i) in case of a wall, where slenderness ratio is based on effective height, 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 100 mm;

(ii) 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 125 mm, whichever is more and average length equal to or more than one-fifth of the height of the wall, is built at right angle to the wall and properly bonded;

(iii) in case of a column, an RC or timber beam/RS joist/roof truss, is supported on the column. In this case, the column will not be considered to be laterally supported in the direction at right angle to it; and

(iv) in case of a column, an RC beam forming a part of beam and slab construction, is supported on the column, and the slab adequately bears on stiffening walls. This construction will provide lateral support to the column, in the direction of both horizontal axes.

7.4.4 Stability

A wall or column subject to vertical and lateral loads may be considered to provide adequate lateral support from consideration of stability, if the construction providing the support is capable of resisting the following forces:

(a) Simple static reactions at the point of lateral support to all the lateral loads; plus

(b) A lateral load equal to 2.5% of the total vertical load that the wall or column is designated to carry at the point of lateral support.

7.4.4.1 In case of load bearing buildings up to five storeys, stability requirements may be considered to have been satisfied if the following conditions are met.

(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 spacing as given in Table 6.7.9 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 the height of the opening.

(c) Floors and roof either bear on cross walls or are properly anchored to those walls such that all lateral loads are safely transmitted to those walls and through them to the foundation.

(d) Cross walls are built jointly with the bearing walls and jointly mortared, or interconnected by toothing.

Cross walls may be anchored to walls to be supported by ties of noncorrosive metal of minimum section 6 x 35 mm and length 60 mm with ends bent at least 50 mm, maximum vertical spacing of ties being 1.2 m.

Table 6.7.9: Thickness and Spacing of Stiffening Walls

Thickness of Load Bearing Wall to be Stiffened (mm)	Height of Storey not to Exceed (m)	Stiffening Wall *		
		Thickness not less than		Maximum spacing (m)
		1 to 3 storeys (mm)	4 and 5 storeys (mm)	
100	3.2	100	-	4.5
200	3.2	100	200	6.0
300	3.4	100	200	8.0
above 300	5.0	100	200	8.0

* Storey height and maximum spacing as given are centre to centre dimensions.

7.4.4.2 In case of walls exceeding 8 m in length, safety and adequacy of lateral supports shall always be checked by structural analysis.

7.4.4.3 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 by the cross walls and structural analysis for stability may be dispensed with.

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7.4.4.6 In case of external walls of basement and plinth, stability requirements of Sec 7.4.4 may be considered to be satisfied 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 Type M_3 or better,

(b) Clear height of ceiling in basement does not exceed 2.6 m,

(c) 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,

(d) Minimum thickness of basement walls is in accordance with Table 6.7.10.

In case there is surcharge on basement walls from adjoining buildings, thickness of basement walls shall be based on structural analysis.

Table 6.7.10: Minimum Thickness of Basement Wall

Minimum Nominal Thickness of Basement Wall (mm)	Height of the Ground above Basement Floor Level	
	Wall Loading (Permanent Load)	
	Less than 50 kN/m	More than 50 kN/m
375	2.0 m	2.5 m
250	1.4 m	1.8 m

7.4.4.7 Free standing wall

Free standing walls, subject to wind pressure or seismic forces shall be designed on the basis of permissible tensile stress in masonry or stability consideration. However in Seismic Zones 1 and 2, free standing walls may be proportioned without making any design calculations with the help of Table 6.7.11 provided the mortar used is of type not leaner than M_3. For parapet wall see Sec 7.4.9.4.

7.4.5 Structural Continuity

Intersecting structural elements intended to act as a unit shall be anchored together to resist the design forces. Walls shall be anchored together to all floors, roofs or other elements which provide lateral support for the wall. Where floors or roofs are designed to transmit horizontal forces to walls, the anchorages to the walls shall be designed to resist the horizontal forces.

Table 6.7.11: Height to Thickness Ratio of Free Standing Wall

Design Wind Pressure, N/m^2	Height to Thickness Ratio
Up to 300	10
600	7
900	5
1100	4

Note: Height is to be taken from 150 mm below ground level or top of footing/ foundation block, whichever is higher, and up to the top edge of the wall.

7.4.5.1 Multi-wythe Walls

All wythes shall be bonded by grout or tied together by corrosion resistant wall ties or joint reinforcement as follows:

(a) Wall Ties in Cavity Wall Construction: Wall ties shall be of sufficient length to engage all wythes. The portion of the wall ties within the wythe shall be completely embedded in mortar or grout. The ends of the wall ties shall be bent to 90 degree angles with an extension not less than 50 mm long. Wall ties not completely embedded in mortar or grout between wythes shall be a single piece with each end engaged in each wythe.

There shall be at least one 6 mm diameter wall tie for each 0.45 m^2 of wall area. For cavity walls in which the width of the cavity is greater than 75 mm, but not more than 115 mm, at least one 6 mm diameter wall tie for each 0.3 m^2 of wall area shall be provided.

Ties in alternate courses shall be staggered. The vertical distance between ties shall not exceed 600 mm. The horizontal distance between ties shall not exceed 900 mm. Additional ties spaced not more than 900 mm apart shall be provided around and within 300 mm of the opening. Wall ties of different size and spacing may be used if they provide equivalent strength between wythes.

(b) Wall Ties for Grouted Multi-wythe Construction: The two wythes shall be bonded together with at least 6 mm diameter steel wall ties for each 0.20 m^2 of area. Wall ties of different size and spacing may be used if they provide equivalent strength between wythes.

(c) Joint Reinforcement: Prefabricated joint reinforcement for masonry walls shall have a minimum of one cross wire of at least 3 mm diameter steel for each 0.2 m^2 of wall area. The vertical spacing of the joint reinforcement shall not exceed 400 mm. The longitudinal wires shall be thoroughly embedded in the bed joint mortar. The joint reinforcement shall engage all wythes.

Where the space between tied wythes is filled with grout or mortar, the allowable stresses and other provisions for masonry bonded walls shall apply. Where the space is not filled, tied walls shall conform to the allowable stress, lateral support, thickness (excluding cavity), height and tie requirements of cavity walls.

7.4.6 Joint Reinforcement and Protection of Ties

The minimum mortar cover between ties or joint reinforcement and any exposed face shall be 15 mm. The thickness of grout or mortar between masonry units and joint reinforcement shall not be less than 6 mm, except that smaller diameter reinforcement or bolts may be placed in bed joints which are at least twice as thick as the diameter of the reinforcement.

7.4.7 Pipes and Conduits

Pipe or conduit shall not be embedded in any masonry so as to reduce the capacity to less than that necessary for required stability or required fire protection, except the following:

(a) Rigid electrical conduit may be embedded in structural masonry when their location has been detailed on the approved plan.

(b) Any pipe or conduit may pass vertically or horizontally through any masonry by means of a sleeve at least large enough to pass any hub or coupling on the pipeline. Such sleeves shall not be placed closer than three diameters, centre to centre, nor shall they unduly impair the strength of construction.

(c) Placement of pipes or conduits in unfilled cores of hollow unit masonry shall not be considered as embedment.

7.4.8 Loads and Load Combination

7.4.8.1 Design loads

All design loads and other forces to be taken for the design of masonry structures shall conform to Chapter 2, Loads.

7.4.8.2 Load dispersion

The angle of dispersion of vertical load on walls shall be taken as not more than 30^{o} from the vertical.

7.4.8.3 Distribution of concentrated vertical loads in walls

The length of wall, laid up in running bond, which may be considered capable of working at the maximum allowable compressive stresses to resist vertical concentrated loads, shall not exceed the centre to centre distance between such loads, nor the width of bearing area plus four times the wall thickness. Concentrated vertical loads shall not be assumed distributed across continuous vertical mortar or control joints unless elements designed to distribute the concentrated vertical loads are employed.

7.4.8.4 Loads on non-bearing wall

Masonry walls used as interior partition or as exterior surfaces of building which do not carry vertical loads imposed by other elements of the building shall be designed to carry their own weight plus any superimposed finish and lateral forces. Bonding or anchorage of nonbearing walls shall be adequate to support the walls and to transfer lateral forces to the supporting structures.

7.4.8.5 Load combinations

Load combination for design of masonry structures shall conform to requirements of Sec 2.7 Chapter 2 Part 6.

7.4.9 Minimum Design Dimensions

7.4.9.1 Minimum thickness of load bearing walls

The nominal thickness of masonry bearing walls in building shall not be less than 250 mm.

Exception:

Stiffened solid masonry bearing walls in one-storey buildings may have a minimum effective thickness of 165 mm when not over 3 m in height, provided that when gable construction is used an additional 1.5 m height may be permitted at the peak of the gable.

7.4.9.2 Variation in thickness

When a change in thickness due to minimum thickness requirements occurs between floor levels, the greater thickness shall be carried up to the higher floor level.

7.4.9.3 Decrease in thickness

When walls of masonry of hollow units or masonry bonded hollow walls are decreased in thickness, a course or courses of solid masonry shall be constructed between the walls below and the thinner wall above, or special units or construction shall be used to transmit the loads from wythes to the walls below.

7.4.9.4 Parapet wall

Parapet walls shall be at least 200 mm thick and height shall not exceed 4 times the thickness. The parapet wall shall not be thinner than the wall below.

7.5 Design of Unreinforced Masonry

7.5.1 General

The requirements of this Section are applicable to unreinforced masonry in addition to the requirements of Sec 7.4.

7.5.2 Design of Members Subjected to Axial Compression

The stresses due to compressive forces applied at the centroid of any load bearing wall, column and pilaster may be computed by Eq. 6.7.22 below assuming uniform distribution over the effective area.

$$f_a = \frac{P}{A_e} \qquad (6.7.22)$$

7.5.3 Design of Members Subjected to Combined Bending and Axial Compression

(a) Compressive stresses due to combined bending and axial load shall satisfy the requirements of Sec 7.3.5.

(b) Resultant tensile stress due to combined bending and axial load shall not exceed the allowable flexural tensile stress, F_t as specified in Sec 7.3.

7.5.4 Design of Members Subjected to Flexure

Stresses due to flexure calculated by Eq. 6.7.23 below shall not exceed the values given in Sec 7.3.5.

$$f_b = \frac{Mc}{I} \qquad (6.7.23)$$

7.5.5 Design of Members Subjected to Shear

Shear calculations in flexural members and shear walls shall be based on Eq. 6.7.24 below.

$$f_v = \frac{V}{A_e} \qquad (6.7.24)$$

7.5.6 Design of Arches

Geometrical form and the cross-sectional dimensions of masonry arch shall be selected such that the line of thrust at any section of the arch is kept within the middle third of the section of the arch rib. The elastic theory of arches shall be permitted for the analysis of unreinforced masonry arches. All supports of arches shall be capable of developing the required horizontal thrust without suffering unacceptable displacements. Every arch must be designed to resist the stresses due to the following loads:

(a) Gravity loads :

(i) Dead loads shall be placed in conformity with their actual distribution.

(ii) Live loads shall be positioned to cover entire span or part of the span as necessary to produce the maximum stresses at the crown, springing and all other sections of the arch rib.

(b) Loads due to temperature change.

(c) Shrinkage load due to setting and hardening.

(d) Shortening of arch rib under thrust caused by loads.

7.5.7 Footings and Corbels

The slope of footing and corbelling (measured from the horizontal to the face of the corbelled surface) shall not be less than 60 degrees.

The maximum horizontal projection of corbelling from the plane of the wall shall be such that stress at any section does not exceed the allowable value.

7.6 Design of Reinforced Masonry

7.6.1 General

The requirements of this Section are in addition to those specified in Sec 7.4 and are applicable to reinforced masonry. Plain bars larger than 6 mm in diameter shall not be used.

7.6.1.1 Assumptions

The following assumptions shall be applicable for this Section.

(a) Masonry carries no tensile stress.

(b) Reinforcement is completely surrounded by and bonded to masonry material so that they work together as a homogeneous material within the range of working stresses.

7.6.2 Design of Members Subjected to Axial Compression

Stresses due to compressive forces applied at the centroid of load bearing wall, column and pilaster may be computed assuming uniform distribution over the effective area. Stress shall be calculated from Eq. 6.7.25 below:

$$f_a = \frac{p}{A_e} \qquad (6.7.25)$$

7.6.3 Design of Members Subjected to Combined Bending and Axial Compression

Stress due to combined bending and axial loads shall satisfy the requirements of Sec 7.3.5. Columns and walls subjected to bending with or without axial loads shall meet all applicable requirements for flexural design.

The design of walls with an (h//t) ratio larger than 30 shall be based on forces and moments determined from analysis of structure. Such analysis shall take into account influence of axial loads and variable moment of inertia on member stiffness and fixed end moments, effect of deflections on moments and forces, and the effects of duration of loads.

7.6.4 Design of Members Subjected to Shear Force

Shearing stresses in flexural members and shear walls shall be computed by

$$f_v = \frac{V}{bjd} \qquad (6.7.26)$$

When the computed shear stress exceeds the allowable value, web reinforcement shall be provided and designed to carry the total shear force. Both vertical and horizontal shear stresses shall be considered. The area required for shear reinforcement placed perpendicular to the longitudinal reinforcement shall be computed by Eq. 6.7.27 below:

$$A_v = \frac{sV}{F_s d} \qquad (6.7.27)$$

Spacing of vertical shear reinforcement shall not exceed d/2, nor 600 mm. Inclined shear reinforcement shall have a maximum spacing of $0.375d(1 + cot\alpha)$, but not greater than 600 mm, where α is the acute angle between inclined bar and the horizontal.

7.6.5 Design of Members Subjected to Flexural Stress

7.6.5.1 Rectangular elements

Rectangular flexural elements shall be designed in accordance with the following equations or other methods based on the simplified assumptions.

(a) Compressive stress in the masonry:

$$f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) \qquad (6.7.28)$$

(b) Tensile stress in the longitudinal reinforcement:

$$f_s = \frac{M}{A_s jd} \tag{6.7.29}$$

(c) Design coefficients :

$$k = [(np)^2 + 2np]^{1/2} - np \tag{6.7.30}$$

Or,

$$k = \frac{1}{1+\frac{f_s}{nf_b}} \tag{6.7.31}$$

$$j = 1 - \frac{k}{3} \tag{6.7.32}$$

7.6.5.2 Nonrectangular sections

Flexural elements of nonrectangular cross-section shall be designed in accordance with the assumptions given in Sec 7.4.2.1 and 7.6.1.1.

7.6.5.3 Lateral support

The clear distance between lateral supports of a beam shall not exceed 32 times the least depth of compression area.

7.6.5.4 Effective width

In computing flexural stresses in walls where reinforcement occurs, the effective width assumed for running bond masonry shall not exceed 6 times the nominal wall thickness or the centre to centre distance between reinforcement. Where stack bond is used, the effective width shall not exceed 3 times the nominal wall thickness or the centre to centre distance between reinforcement or the length of one unit, unless grouted solid using open-ended joints.

7.6.5.5 Bond

In flexural members in which tensile reinforcement is parallel to the compressive face, the bond stress shall be computed by the formula:

$$u = \frac{V}{\sum_o jd} \tag{6.7.33}$$

7.6.6 Reinforcement Requirements and Details

7.6.6.1 Column reinforcement

(a) Vertical Reinforcement: The area of vertical reinforcement shall not be less than 0.005 A_e and not more than 0.04A_e. At least four 10 mm bars shall be provided.

(b) Lateral Ties: All longitudinal bars for columns shall be enclosed by lateral ties. Lateral support shall be provided to the longitudinal bars by the corner of a complete tie having an included angle of not more than 135 degrees or by a hook at the end of a tie. The corner bars shall have such support provided by a complete tie enclosing the longitudinal bars. Alternate longitudinal bars shall have such lateral support provided by ties and no bar shall be farther than 150 mm from such a laterally supported bar.

Lateral ties and longitudinal bars shall be placed not less than 40 mm and not more than 125 mm, from the surface of the column. Lateral ties may be against the longitudinal bars or placed in the horizontal bed joint if the requirements of Sec 4.4.6 are met. Spacing of ties shall not be more than 16 times longitudinal bar diameter, 48 times tie bar diameter or the least dimension of the column but not more than 450 mm.

Ties shall be at least 6 mm in diameter for 22 mm diameter or smaller longitudinal bars and 10 mm in diameter for larger longitudinal bars. Ties less than 10 mm in diameter may be used for longitudinal bars larger than 22 mm in diameter, provided the total cross-sectional area of such smaller ties crossing a longitudinal plane is equal to that of the larger ties at their required spacing.

(c) Anchor Bolt Ties: Additional ties shall be provided around anchor bolts which are set in the top of the column. Such ties shall engage at least four bolts or, alternatively at least four vertical column bars or a combination of bolts and bars totaling four in number. Such ties shall be located within the top 125 mm of the column and shall provide a total of 250 square millimeters or more in cross-sectional area. The upper most ties shall be within 50 mm of the top of the column.

7.6.6.2 Maximum reinforcement size

The maximum size of reinforcing bars shall be 35 mm. Maximum steel area in cell shall be 6 percent of the cell area without splices and 12 percent of cell area with splices.

7.6.6.3 Spacing of longitudinal reinforcement

The clear distance between parallel bars, except in columns, shall not be less than the nominal diameter of the bars or 25 mm, except that bars in a splice may be in contact. This clear distance requirement applies to the clear distance between a contact splice and adjacent splices or bars. The minimum clear distance between parallel bars in columns shall be two and one-half times the bar diameter.

The clear distance between the surface of a bar and any surface of a masonry unit shall not be less than 6 mm for fine grout and 12 mm for coarse grout. Cross webs of hollow units may be used as support for horizontal reinforcement.

All reinforcing bars, except joint reinforcing, shall be completely embedded in mortar or grout and have a minimum cover, including the masonry unit, as specified below:

(a) 20 mm when not exposed to weather

(b) 40 mm when exposed to weather

(c) 50 mm when exposed to soil

7.6.6.4 Anchorage of Flexural Reinforcement

(a) The tension or compression in any bar at any section must be developed on each side of that section by the required development length. The development length of the bar may be achieved by a combination of an embedment length, anchorage or, for tension only, hooks.

The required development length for deformed bars or deformed wires shall be calculated by:

For bar in tension,

$$l_d = 0.29 d_b f_s \tag{6.7.34}$$

For bar in compression,

$$l_d = 0.22 d_b f_s \tag{6.7.35}$$

Development length for plain bars shall be 2.0 times the length calculated by Eq. 6.7.34.

(b) Except at supports, or at the free end of cantilevers, every reinforcing bar shall be extended beyond the point at which it is no longer needed to resist tensile stress for a distance equal to 12 bar diameters or the depth of the flexural member, whichever is greater. No flexural bars shall be terminated in a tensile zone unless one of the following conditions is satisfied:

(i) The shear is not over one-half of that permitted, including allowance for shear reinforcement, if any.

(ii) Additional shear reinforcement in excess of that required is provided each way from the cutoff a distance equal to the depth of the beam. The shear reinforcement spacing shall not exceed $d/8r_b$, where r_b is the ratio of the area of bars cutoff to the total area of bars at the section.

(iii) The continuing bars provide double the area required for flexure at that point or double the perimeter required for reinforcing bond.

(c) At least one third of the total reinforcement provided for negative moment at the support shall be extended beyond the extreme position of the point of inflection a distance sufficient to develop one half the allowable stress in the bar, one sixteenth of the clear span, or the depth d of the member, whichever is greater.

(d) Tensile reinforcement of negative moment in any span of a continuous restrained or cantilever beam, or in any member of a rigid frame, shall be adequately anchored by reinforcing bond, hooks or mechanical anchors in or through the supporting member.

(e) At least one third of the required positive moment reinforcement in simple beams or at the freely supported end of continuous beams shall extend along the same face of the beam into the support at least 150 mm. At least one fourth of the required positive moment reinforcement at the continuous end of continuous beams shall extend along the same face of the beam into the support at least 150 mm.

(f) Compression reinforcement in flexural members shall be anchored by ties or stirrups not less than 6 mm in diameter, spaced not farther apart than 16 bar diameters or 48 tie diameters whichever is smaller. Such ties or stirrups shall be used throughout the distance where compression steel is required.

(g) In regions of moment where the design tensile stresses in the steel are greater than 80 percent of the allowable steel tensile stress (F_s), the lap length of splices shall be increased not less than 50 percent of the minimum required length. Other equivalent means of stress transfer to accomplish the same 50 percent increase may be used.

7.6.6.5 Anchorage of shear reinforcement

(a) Single separate bars used as shear reinforcement shall be anchored at each end by one of the following methods:

(i) Hooking tightly around the longitudinal reinforcement through 180 degrees.

(ii) Embedment above or below the mid-depth of the beam on the compression side a distance sufficient to develop the stress in the bar for plane or deformed bars.

(iii) By a standard hook (see Sec 7.6.6.6) considered as developing 50 N/mm^2, plus embedment sufficient to develop the remainder of the stress to which the bars are subject. The effective embedded length shall not be assumed to exceed the distance between the mid-depth of the beam and the tangent of the hook.

(b) The ends of bars forming single U or multiple U stirrups shall be anchored by one of the methods specified above or shall be bent through an angle of at least 90 degrees tightly around a longitudinal reinforcing bar not less in diameter than the stirrup bar, and shall project beyond the bend at least 12 diameters of the stirrup.

(c) The loops or closed ends of single U or multiple U stirrups shall be anchored by bending around the longitudinal reinforcement through an angle of at least 90 degrees and project beyond the end of the bend at least 12 diameters of the stirrup.

7.6.6.6 Hooks

(a) The term "standard hook" shall mean one of the following:

(i) A 180 degree turn plus an extension of at least 4 bar diameters but not less than 65 mm at the free end of the bar.

(ii) 90 degree turn plus an extension of at least 12 bar diameters at the free end of the bar.

(iii) For stirrup and tie anchorage only either a 90 degree or a 135 degree turn, plus an extension of at least 6 bar diameters but not less than 65 mm at the free end of the bar.

(b) The diameter of bend measured on the inside of the bar other than stirrups and ties, shall not be less than that set forth in Table 6.7.12.

Table 6.7.12: Minimum Diameter of Bend

Bar Diameter	Minimum Diameter of Bend
6 to 25 mm	6 bar diameters
8 to 35 mm	8 bar diameters

(c) Inside diameter of bend for 12 mm diameter or smaller stirrups and ties shall not be less than 4 bar diameters. Inside diameter of bend for 16 mm diameter or larger stirrups and ties shall not be less than that given in Table 6.7.12.

(d) Hooks shall not be permitted in the tension portion of any beam, except at the ends of simple or cantilever beams or at the freely supported ends of continuous or restrained beams.

(e) Hooks shall not be assumed to carry a load which would produce a tensile stress in the bar greater than 50 N/mm^2.

(f) Hooks shall not be considered effective in adding to the compressive resistance of bars.

(g) Any mechanical device capable of developing the strength of the bar without damage to the masonry may be used in lieu of a hook. Data must be presented to show the adequacy of such devices.

7.6.6.7 Splices

The amount of lap of lapped splices shall be sufficient to transfer the allowable stress of the reinforcement as in Sec 7.6.6.4. In no case shall the length of the lapped splice be less than 30 bar diameters for compression and 40 bar diameters for tension.

Welded or mechanical connections shall develop 125 percent of the specified yield strength of the bar in tension, except for connections of compression bars in columns that are not part of the seismic system and are not subject to flexure, where the compressive strength only need be developed.

When adjacent splices in grouted masonry are separated by 75 mm or less, the lap length shall be increased by 30 percent or the splice may be staggered at least 24 bar diameters with no increase in lap length.

7.7 Strength Design of Slender Walls and Shear Walls

7.7.1 Design of Slender Walls

In lieu of the procedure set forth in Sec 7.6, the procedures prescribed in this Section, which consider the slenderness of walls by representing effects of axial forces and deflection in calculation of moments, may be used when the vertical load stress at the location of maximum moment computed by Eq. 6.7.36 does not exceed $0.04f'_m$. The value of f'_m shall not exceed 40 N/mm^2.

$$\frac{P_w+P_f}{A_g} \leq 0.04f'_m \qquad (6.7.36)$$

Slender masonry walls shall have a minimum nominal thickness of 150 mm.

7.7.1.1 Slender wall design procedure

(a) Maximum Reinforcement: The reinforcement ratio shall not exceed $0.5\rho_b$, where ρ_b is the balanced steel ratio.

(b) Moment and Deflection Calculation: All moments and deflections of slender walls shall be calculated based on simple support conditions at top and bottom. For other support and fixity conditions, moments and deflections shall be calculated using established principles of mechanics.

7.7.1.2 Strength design

(a) Loads: Factored loads shall be determined in accordance with Chapter 2, Loads.

(b) Required Moment: Required moment and axial force shall be determined at the mid-height of the wall and shall be used for design. The factored moment, M_u, at the mid-height of the wall shall be determined by Eq. 6.7.37.

$$M_u = \frac{w_u h^2}{8} + P_u \frac{e}{2} + \left(P_{uw} + P_{uf}\right)\Delta_u \qquad (6.7.37)$$

Where,

Δ_u = horizontal deflection at mid-height under factored load; P - Delta effects shall be included in deflection calculation.

e = eccentricity of P_u

P_u = axial load at mid-height of wall, including tributary wall weight.

$= P_{uw} + P_{uf}$

(c) Design Strength: Design strength in flexure is the nominal moment strength, M_n multiplied by the strength reduction factor, ϕ and shall equal or exceed the factored moment, M_u

$$M_u \leq \phi M_n \qquad (6.7.38)$$

Where,

M_n = nominal moment strength

$= A_{se} f_y (d - a/2)$

A_{se} = effective area of steel

$= \frac{A_s f_y + P_u}{f_y}$ and

a = depth of stress block due to factored loads.

$= \frac{A_s f_y + P_u}{0.85 f'_m b}$

The strength reduction factor ϕ for flexure shall be 0.80.

(d) Design Assumptions: The following are the design assumptions for calculation of nominal strength.

(i) Nominal strength of singly reinforced masonry wall cross-sections subject to combined flexure and axial load shall be based on applicable conditions of equilibrium and compatibility of strains.

(ii) Strain in reinforcement and masonry walls shall be assumed directly proportional to the distance from the neutral axis.

(iii) Maximum usable strain at extreme masonry compression fibre shall be assumed equal to 0.003.

(iv) Stress in reinforcement below specified yield strength f_y shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y stress in reinforcement shall be considered independent of strain and equal to f_y.

(v) Tensile strength of masonry walls shall be neglected in flexural calculations of strength, except for deflection calculation.

(vi) Relationship between masonry compressive stress and masonry strain may be assumed to be rectangular as defined by the following:

- Masonry stress of $0.85f'_m$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross-section and a straight line located parallel to the neutral axis at a distance $a = 0.85c$ from the fibre of maximum compressive strain.
- Distance c from fibre of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis.

7.7.1.3 Deflection calculation

The mid-height deflection, Δ_s under service lateral and vertical loads (without load factors) shall be limited to:

$$\Delta_s = 0.007h \qquad (6.7.39)$$

The mid-height deflection shall be computed by:

When, $M_{ser} \leq M_{cr}$

$$\Delta_s = \frac{5M_s h^2}{48E_m I_g} \qquad (6.7.40)$$

When, $M_{cr} < M_{ser} < M_n$

$$\Delta_s = \frac{5M_{cr}h^2}{48E_m I_g} + 5\frac{(M_{ser} - M_{cr})h^2}{48E_m I_{cr}} \qquad (6.7.41)$$

The cracking moment strength of the wall M_{cr} shall be determined by:

$$M_{cr} = Sf_y \quad (6.7.42)$$

The modulus of rupture, f_r shall be determined form Table 6.7.13.

Table 6.7.13: Values of the Modulus of Rupture, f_r

Type of Masonry	Fully Grouted	Partially Grouted
Solid Masonry	$0.17\sqrt{f'_m} \le 0.65$ N/mm^2	Not allowed
Hollow Unit Masonry	$0.33\sqrt{f'_m} \le 1.2$ N/mm^2	$0.21\sqrt{f'_m} \le 0.65$ N/mm^2

7.7.2 Design of Shear Walls

Based on ultimate strength design, the procedures described below may be used as an alternative to the procedure specified in Sec 7.6 for the design of reinforced hollow unit masonry shear walls. Provisions for quality control during construction of the shear wall are specified in Sec 7.3.4

7.7.2.1 Required strength

The required strength to resist different combinations of loads shall be determined in accordance with Sec 2.7.3.1 Chapter 2 of this Part.

7.7.2.2 Design strength

Shear walls shall be proportioned such that the design strength exceeds the required strength. Design strength in terms of axial force, shear force and moment provided by the shear wall shall be computed as the nominal strength multiplied by the strength reduction factor ϕ. Strength reduction factor ϕ shall be as follows:

(a) For axial load and axial load with flexure $\phi = 0.65$

(b) For members with f_y less than 410 N/mm^2 and with symmetrical reinforcement, ϕ may be increased linearly to 0.85 as ϕP_n decreases from $0.10 f'_m A_e$ or $0.25 P_b$ to zero.

For solid grouted walls P_b may be calculated using:

$$P_b = 0.85 f'_m b a_b \quad (6.7.43a)$$

Where,

$$a_b = 0.85 \left[e_{mu} / \left(e_{mu} + f_y/E_s\right)\right] d \quad (6.7.43b)$$

(c) For shear $\phi = 0.60$. The shear strength reduction factor may be increased to 0.80 for any shear wall when its nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength for the factored load combination.

7.7.2.3 Design Assumptions for Nominal Strength

(a) Nominal strength of shear wall cross-sections shall be based on assumptions specified in Sec 7.7.1.2(d).

(b) The maximum usable strain e_{mu}, at the extreme masonry compression fibre shall not exceed 0.003.

(c) f'_m shall not be less than 7 N/mm^2 or greater than 20 N/mm^2.

7.7.2.4 Axial Strength

The nominal axial strength of shear walls supporting axial loads only shall be calculated by Eq 6.7.44.

$$P_o = 0.85 f'_m (A_e - A_s) + f_y A_s \tag{6.7.44}$$

The shear wall shall be designed for the axial strength P_u, such that

$$P_u \leq \emptyset (0.80) P_0 \tag{6.7.45}$$

7.7.2.5 Shear strength

(a) The nominal shear strength shall be determined by the provisions as specified in (b) or (c) below. The maximum nominal shear strength values are given in Table 6.7.14.

Table 6.7.14: Maximum Nominal Shear Strength Values

$\frac{M^*}{Vd}$	$\frac{V_n}{A_e\sqrt{f'_m}}$
≤ 0.25	72.0
≥ 1.00	48.0

* M is the maximum bending moment that occurs simultaneously with the shear load V at the section under consideration. Interpolation may be by straight line for M/Vd values between 0.25 and 1.00.

(b) The nominal shear strength of shear walls except for shear walls specified in (c) below shall be determined by Eq. 6.7.46.

$$V_n = V_m + V_s \qquad (6.7.46)$$

Where,

$$V_m = 0.083 C_d A_{mv} \sqrt{f'_m} \qquad (6.7.47)$$

The value of C_d in Eq. 6.7.47 is given as:

$$C_d = 2.4 \text{ for } \frac{M}{Vd} \leq 0.25 \qquad (6.7.48a)$$

$$C_d = 1.2 \text{ for } \frac{M}{Vd} \geq 1.0 \qquad (6.7.48b)$$

$$V_s = A_{mv} \rho_n f_y \qquad (6.7.48c)$$

(c) For a shear wall whose nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength, two shear regions exist.

(i) For all cross-sections within the region defined by the base of the shear wall and a plane at a distance L_w above the base of the shear wall, the nominal shear strength shall be determined by Eq. 6.7.49

$$V_n = A_{mv} \rho_n f_y \qquad (6.7.49)$$

The required shear strength for this region shall be calculated at a distance $\frac{L_w}{2}$ above the base of the shear wall but not to exceed one-half storey height.

(ii) For the other region, the nominal shear strength of the shear wall shall be determined by Eq. 6.7.46.

7.7.2.6 Reinforcement

Reinforcement shall be in accordance with the following:

(a) Minimum reinforcement shall be provided in accordance with Sec 7.8.5.1 for all seismic areas using this method of analysis.

(b) When the shear wall failure mode is in flexure, the nominal flexural strength of the shear wall shall be at least 1.8 times the cracking moment strength of a fully grouted wall or 3.0 times the cracking moment strength of a partially grouted wall as obtained from Eq. 6.7.42.

(c) All continuous reinforcement shall be anchored or spliced in accordance with Sec 7.6.6.4 with $f_s = 0.5 f_y$.

(d) Vertical reinforcement shall not be less than 50 percent of the horizontal reinforcement.

(e) Spacing of horizontal reinforcement within the region defined in Sec 7.7.2.5(c) shall not exceed three times the nominal wall thickness or 600 mm, whichever is smaller.

7.7.2.7 Boundary member

Boundary members shall be as follows:

(a) The need for boundary members at boundaries of shear wall shall be determined using the provisions set forth in (b) or (c) below.

(b) Boundary members shall be provided when the failure mode is flexure and the maximum extreme fibre stress exceeds $0.2f'_m$. The boundary members may be discontinued where the calculated compressive stresses are less than $0.15f'_m$. Stresses may be calculated for the factored forces using a linearly elastic model and gross section properties.

(c) When the failure mode is flexure, boundary member shall be provided to confine all vertical reinforcement whose corresponding masonry compressive stress exceeds $0.4f'_m$. The minimum length of the boundary member shall be 3 times the thickness of the wall.

(d) Boundary members shall be confined with minimum of 10 mm diameter bars at a maximum of 200 mm spacing or equivalent within the grouted core and within the region defined by the base of the shear wall and a plane at a distance L_w above the base of the shear wall.

7.8 Earthquake Resistant Design

7.8.1 General

All masonry structures constructed in the Seismic Zones 2, 3 and 4 shown in Figure 6.2.13 shall be designed in accordance with the provisions of this Section.

7.8.2 Loads

Seismic forces on masonry structures shall be determined in accordance with the provisions of Sec 2.5 Chapter 2 of this Part.

7.8.3 Materials

(a) Well burnt clay bricks and concrete hollow blocks having a crushing strength not less than 12 N/mm^2 shall be used.

(b) Mortar not leaner than M_3 shall be used for masonry constructions.

7.8.4 Provisions for Seismic Zone 2 and 3

7.8.4.1 Wall Reinforcement

Vertical reinforcement of at least 12 mm diameter shall be provided continuously from support to support at each corner, at each side of each opening, at the ends of walls and at a maximum spacing of 1.2 m horizontally throughout the wall. Horizontal reinforcement not less than 12 mm diameter shall be provided:

(a) at the bottom and top of wall openings and shall extend at least 40 bar diameters, with a minimum of 600 mm, past the opening,

(b) continuously at structurally connected roof and floor levels and at the top of walls,

(c) at the bottom of the wall or in the top of the foundations when dowelled to the wall,

(d) at maximum spacing of 3.0 m unless uniformly distributed joint reinforcement is provided. Reinforcement at the top and bottom of openings when continuous in the wall may be used in determining the maximum spacing specified in item (a) above.

7.8.4.2 Stack bond

Where stack bond is used, the minimum horizontal reinforcement ratio shall be $0.0007bt$. This ratio shall be satisfied by uniformly distributed joint reinforcement or by horizontal reinforcement spaced not more than 1.2 m and fully embedded in grout or mortar.

7.8.4.3 Columns

Columns shall be reinforced as specified in Sec 7.6.6.1.

7.8.5 Provisions for Seismic Zone 4

All masonry structures built in Seismic Zone 4 shall be designed and constructed in accordance with requirements for Seismic Zone 2 and with the following additional requirements and limitations.

Reinforced hollow unit stack bond construction which is part of the seismic resisting system shall use open-end units so that all head joints are made solid, shall use bond beam units to facilitate the flow of grout and shall be grouted solid.

7.8.5.1 Wall reinforcement

Reinforced masonry walls shall be reinforced with both vertical and horizontal reinforcement. The sum of the areas of horizontal and vertical reinforcement shall be at least 0.002 times the gross cross-sectional area of the wall and the area of reinforcement in either direction shall not be less than 0.0007 times the gross cross-sectional area of the wall. The spacing of reinforcement shall not exceed 1.20 m. The diameter of reinforcing bar shall not be less than 10 mm except that joint reinforcement may be considered as part of all of the requirements for minimum reinforcement. Reinforcement shall be continuous around wall corners and through intersections. Only reinforcement which is continuous in the wall or element shall be considered in computing the minimum area of reinforcement. Reinforcement with splices conforming to Sec 7.6.6.7 shall be considered as continuous reinforcement.

7.8.5.2 Column reinforcement

The spacing of column ties shall be not more than 225 mm for the full height of columns stressed by tensile or compressive axial overturning forces due to the seismic loads, and 225 mm for the tops and bottoms of all other columns for a distance of one sixth of the clear column height, but not less than 450 mm or maximum column dimension. Tie spacing for the remaining column height shall be not more than 16 bar diameters, 48 tie diameters or the least column dimension, but not more than 450 mm.

7.8.5.3 Stack bond

Where stack bond is used, the minimum horizontal reinforcement ratio shall be $0.0015bt$. If open-end units are used and grouted solid, the minimum horizontal reinforcement ratio shall be $0.0007bt$.

7.8.5.4 Minimum dimension

(a) Bearing Walls: The nominal thickness of reinforced masonry bearing walls shall be not less than 150 mm except that nominal 100 mm thick load bearing reinforced hollow clay unit masonry walls may be used, provided net area unit strength exceeds 55 N/mm^2, units are laid in running bond, bar sizes do not exceed 12 mm with no more than two bars or one splice in a cell, and joints are flush cut, concave or a protruding V section.

(b) Columns: The least nominal dimension of a reinforced masonry column shall be 375 mm except that if the allowable stresses are reduced to 50 percent of the values given in Sec 7.3, the minimum nominal dimension shall be 250 mm.

7.8.5.5 Shear wall

(a) When calculating shear or diagonal tension stresses, shear walls which resist seismic forces shall be designed to resist 1.5 times the forces specified in Chapter 2, Loads.

(b) The portion of the reinforcement required to resist shear shall be uniformly distributed and shall be joint reinforcing, deformed bars, or a combination thereof. The maximum spacing of reinforcement in each direction shall be not less than the smaller of one-half the length or height of the element or more than 1.20 m.

Joint reinforcement used in exterior walls and considered in the determination of the shear strength of the member shall conform to the requirement "Joint Reinforcement for Masonry" (UBC Standard No. 24-15) or "Standard Specification for Steel Wire, Plain, for Concrete Reinforcement", (ASTM, A82).

Reinforcement required to resist in-plane shear shall be terminated with a standard hook or with an extension of proper embedment length beyond the reinforcing at the end of the wall section. The hook or extension may be turned up, down or horizontally. Provisions shall be made not to obstruct grout placement. Wall reinforcement terminating in columns or beams shall be fully anchored into these elements.

(c) Multi-wythe grouted masonry shear walls shall be designed with consideration of the adhesion bond strength between the grout and masonry units. When bond strengths are not known from previous tests, the bond strength shall be determined by test.

7.8.5.6 Hook

The standard hook for tie anchorage shall have a minimum turn of 135 degrees plus an extension of at least 6 bar diameters, but not less than 100 mm at the free end of the bar. Where the ties are placed in the horizontal bed joints, the hook shall consist of a 90 degree bend having a radius of not less than 4 tie diameters plus an extension of 32 tie diameters.

7.8.5.7 Mortar joints between masonry and concrete

Concrete abutting structural masonry such as at starter courses or at wall intersections not designed as true separation joints shall be roughened to a full amplitude of 1.5 mm and shall be bonded to the masonry as per the requirements of this Section as if it were masonry.

7.8.6 Additional Requirements

7.8.6.1 Opening in bearing walls

(a) Tops of all openings in a storey shall preferably be at the same level so that a continuous band could be provided over them, including the lintels throughout the building.

(b) The total width of the openings shall not be more than half of the length of the walls between the adjacent cross walls, except as provided in (f) below.

(c) The opening shall preferably be located away from the corner by a clear distance of at least one-eighth of the height of the opening for Seismic Zones 2 and 3, and one-fourth of the height for Seismic Zone 4.

(d) The horizontal distance between two openings shall not be less than one-fourth of the height of the shorter opening for Seismic Zones 2 and 3, and one-half of the height for Seismic Zone 4.

(e) The vertical distance between openings one above the other shall be not less than 600 mm.

(f) Where openings do not comply with the requirements of (b) and (c) above, they shall be strengthened in accordance with Sec 7.8.6.5.

(g) If a window or ventilator is to be projected out, the projection shall be in reinforced masonry or concrete and well anchored.

(h) If the height of an opening is approximately full height of a wall, dividing the wall into two portions, these portions of the wall shall be reinforced with horizontal reinforcement of 6 mm diameter bars at not more than 600 mm intervals, one on inner and one on outer face, properly tied to vertical steel at jambs and corners or junctions of walls where used.

(i) The use of arches to span over the openings is a source of weakness and shall be avoided unless steel ties are provided.

7.8.6.2 Strengthening arrangements

All masonry buildings shall be strengthened by the methods specified in Table 6.7.15.

Table 6.7.15: Strengthening of Masonry Buildings for Earthquake

Seismic Zones	No. of Storey	Strengthening Arrangements to be Provided.
1	Up to 4	a) Masonry mortar shall not be leaner than M_3
2, 3	Up to 2 with pitched roof	a) Masonry mortar shall not be leaner than M_3 b) By lintel and roof band (Sec 7.8.6.3) c) By vertical reinforcement at corners and junctions of walls (Sec 7.8.6.4) d) Bracing in plan at tie level for pitched roof*
	3 to 4	a) Masonry mortar shall not be leaner than M_3 b) By lintel and roof band (Sec 7.8.6.3) c) By vertical reinforcement at corners and junctions of walls (Sec 7.8.6.4) d) Vertical reinforcement at jambs of openings (Sec 7.8.6.5) e) Bracing in plan at tie level for pitched roof*

Seismic Zones	No. of Storey	Strengthening Arrangements to be Provided.
4	Up to 4	a) Masonry mortar shall not be leaner than M_3 b) By lintel and roof band (Sec 7.8.6.3) c) By vertical reinforcement at corners and junctions of walls (Sec 7.8.6.4) d) Vertical reinforcement at jambs of openings (Sec 7.8.6.5) e) Bracing in plan at tie level for pitched roof*

- At tie level all the trusses and the gable end shall be provided with diagonal bracing in plan so as to transmit the lateral shear due to earthquake force to the gable walls acting as shear walls at the ends.

7.8.6.3 Bands

Roof band need not be provided underneath reinforced concrete or brickwork slabs resting on bearing walls, provided that the slabs are continuous over parts between crumple sections, if any, and cover the width of end walls fully.

The band shall be made of reinforced concrete with f_c' not less than 20 N/mm^2 or reinforced brickwork in cement mortar not leaner than 1: 4. The bands shall be to the full width of the wall and not less than 75 mm in depth and shall be reinforced as indicated in Table 6.7.16. In case of reinforced brickwork, the thickness of joints containing steel bars shall be increased so as to have a minimum mortar cover of 6 mm around the bar. In bands of reinforced brickwork, the area of steel provided shall be equal to that specified above for reinforced concrete bands.

Table 6.7.16: Band Reinforcement

Seismic Zones	Plain Mild Steel Bars	High Strength Deformed Bars	Links
2, 3	2 - 12 mm dia, one on each face of the wall with suitable cover	2 - 10 mm dia, one on each face of the wall with suitable cover	6 mm dia, 150 mm c/c
4	2 - 16 mm dia, one on each face of the wall with suitable cover	2 - 12 mm dia, one on each face of the wall with suitable cover	6 mm dia, 150 mm c/c

7.8.6.4 Strengthening of corner and junctions

Vertical steel at corners and junctions of walls which are up to one and a half bricks thick shall be provided either with mild steel or high strength deformed bars as specified in Table 6.7.17. For thicker walls, reinforcement shall be increased proportionately. The 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 and passing through the lintel bands in all storeys. Bars in different storeys may be welded or suitably lapped.

(a) Typical details of vertical steel in brickwork and hollow block at corners, T-junctions and jambs of opening are shown in Figures 6.7.1 and 6.7.2.

(b) Details of vertical reinforcement given in Table 6.7.17 are applicable to brick masonry and hollow block masonry.

7.8.6.5 Strengthening of jambs of openings

Openings in bearing walls shall be strengthened, where necessary, by providing reinforced concrete members or reinforcing the brickwork around them as shown in Figure 6.7.3.

7.8.6.6 Walls adjoining structural framing

Where walls are dependent on the structural frame for lateral support they shall be anchored to the structural members with metal ties or keyed to the structural members. Horizontal ties shall consist of 6 mm diameter U-bars spaced at a maximum of 450 mm on centre and embedded at least 250 mm into the masonry and properly tied to the vertical steel of the same member.

Table 6.7.17: Vertical Reinforcement for Brick and Hollow Block Masonry

No. of Storeys	Storeys	Diameter of Single Bar or Equivalent Area of Plain Mild Steel Bar to be Provided (mm)		Diameter of Single Bar or Equivalent Area of High Strength Deformed Bar to be Provided (mm)	
		Zone 2 and 3	Zone 4	Zone 2 and 3	Zone 4
1	-	nil	12	nil	10
2	Top	nil	12	nil	10
	Bottom	nil	16	nil	12
3	Top	12	12	10	10
	Middle	12	16	10	12
	Bottom	16	16	12	12
4	Top	12	12	10	10
	Third	12	16	10	12
	Second	16	20	12	16
	Bottom	16	25	12	20

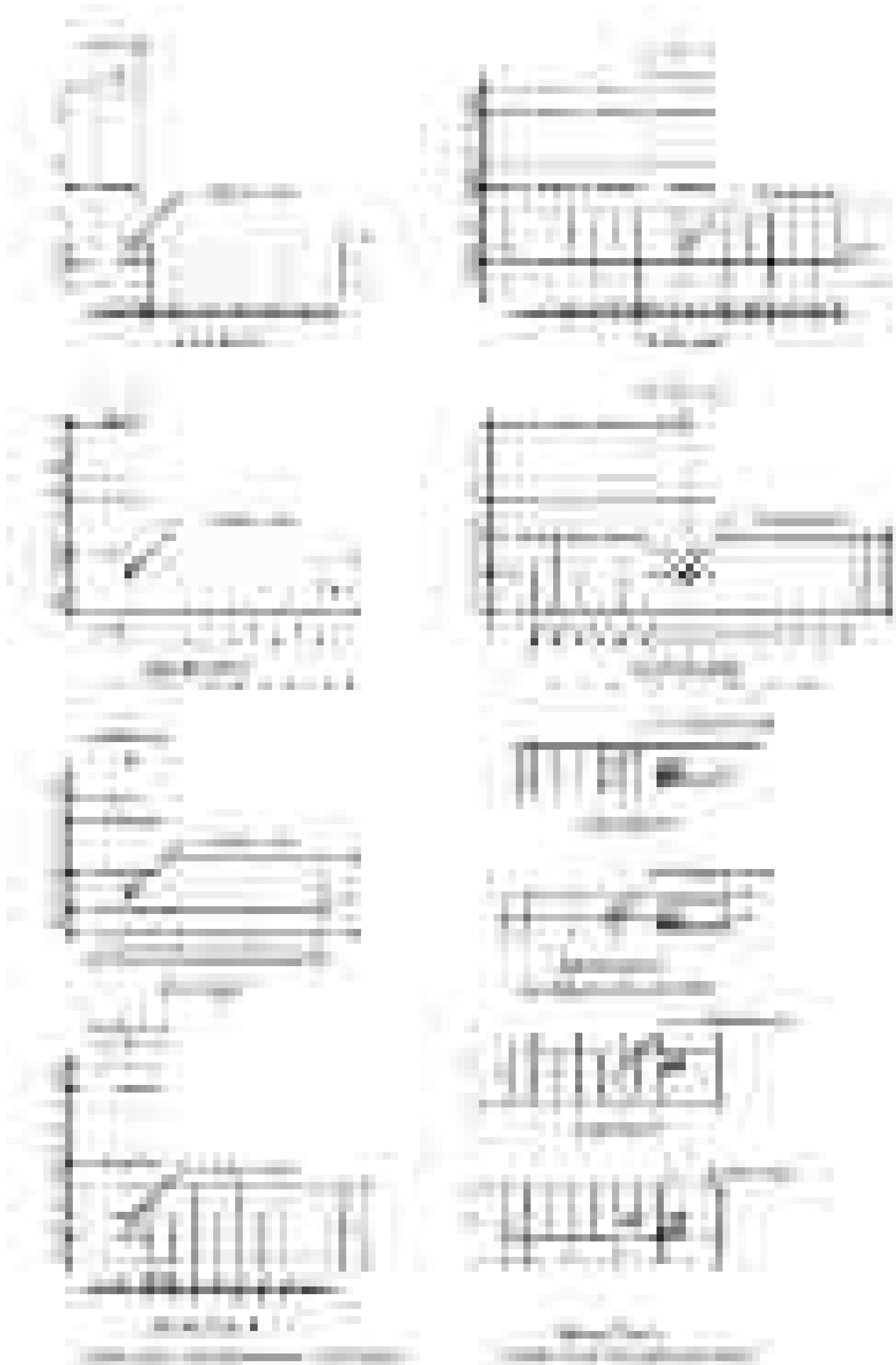


Figure 6.7.1 Typical details of vertical reinforcement in brick masonry

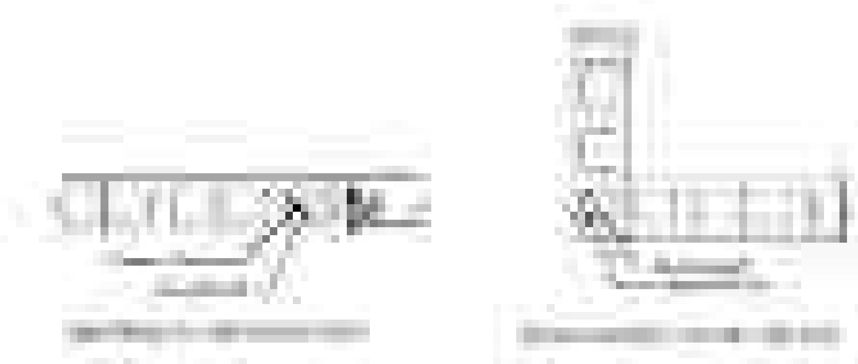


Figure 6.7.2 Typical details of vertical reinforcement in hollow block masonry

Figure 6.7.3 Minimum reinforcement in walls and around openings in Seismic Zones 2, 3 and 4

7.9 Provisions For High Wind Regions

7.9.1 General

The provisions of this Section shall apply to masonry structures located at regions where the basic wind speed is greater than 200 km/h.

7.9.2 Materials

Materials for masonry structures shall generally comply with the provisions of Part 5; however, there are some special requirements for masonry construction in high wind regions, which are given below:

(a) Burnt clay bricks shall have a compressive strength not less than 15 N/mm^2,

(b) Grout shall have a minimum compressive strength of 12.5 N/mm^2,

(c) Mortar for exterior walls and interior shear walls shall be type M_1 or M_2,

(d) Unburnt clay masonry units shall not be used.

7.9.3 Construction Requirements

Masonry construction shall comply with the provisions of Sec 7.10.

7.9.4 Foundation

Footings shall have a thickness of not less than 375 mm and shall be extended 450 mm below the undisturbed ground surface. Foundation stem wall shall have the same width and reinforcement as the wall it supports.

7.9.5 Drainage

Walls retaining more than 1 m of earth and enclosing interior spaces or floors below grade shall have minimum 100 mm diameter footing drain. A slope of 1:50 away from the building shall be provided around the building.

7.9.6 Wall Construction

7.9.6.1 Minimum thickness of different types of wall shall be as given in Table 6.7.18.

7.9.6.2 All walls shall be laterally supported at the top and bottom. The maximum unsupported height of bearing walls or other masonry walls shall be 3.5 m. Gable end walls may be 4.5 m high at their peak.

7.9.6.3 The span of lintels over openings shall not exceed 3.5 m. All lintels shall be reinforced and the reinforcement bars shall extend not less than 600 mm beyond the edge of opening and into lintel supports.

7.9.6.4 Walls shall be adequately reinforced.

7.9.6.5 Anchors between walls and floors or roofs shall be embedded in grouted cells or cavities and shall conform to Sec 7.9.7 below.

Table 6.7.18: Minimum thickness of Walls in High Wind Region

Type of Wall	Minimum Thickness (mm)
Unreinforced grouted brick wall	250
Reinforced exterior bearing wall	200
Unreinforced hollow and solid masonry wall	200
Interior nonbearing wall	150

7.9.7 Floor and Roof Systems

Floors and roofs of all masonry structures shall be adequately anchored with the wall it supports to resist lateral and uplift forces due to wind specified in Sec 2.4 of this Part.

7.9.8 Lateral Force Resistance

7.9.8.1 Strapping, approved framing anchors and mechanical fasteners, bond beams and vertical reinforcement shall be installed to provide a continuous tie from the roof to foundation system as shown in Figure 6.7.4. In addition, roof and floor systems, masonry shear walls, or masonry or wood cross walls shall be provided for lateral stability.

7.9.8.2 Floor and roof diaphragms shall be properly connected to masonry walls. Gable and sloped roof members not supported at the ridge shall be tied by the ceiling joist or equivalent lateral ties located as close to where the roof members bear on the wall as practically possible and not at more than 1.2 m on centers. Collar ties shall not be used for these lateral ties.

7.9.8.3 Masonry walls shall be provided around all sides of floor and roof systems in accordance with Figure 6.7.5. The cumulative length of exterior masonry walls along each side of the floor or roof systems shall be at least 20 percent of the parallel dimension. Required elements shall be without openings and shall not be less that 1.25 m in width.

Interior cross walls at right angles to bearing walls shall be provided when the length of the building perpendicular to the span of the floor of roof framing exceeds twice the distance between shear walls or 10 m, whichever is greater.

7.9.8.4 When required interior cross wall shall be at least 1.8 m long and reinforced with 2 mm wire joint reinforcement spaced not more than 400 mm on centre.

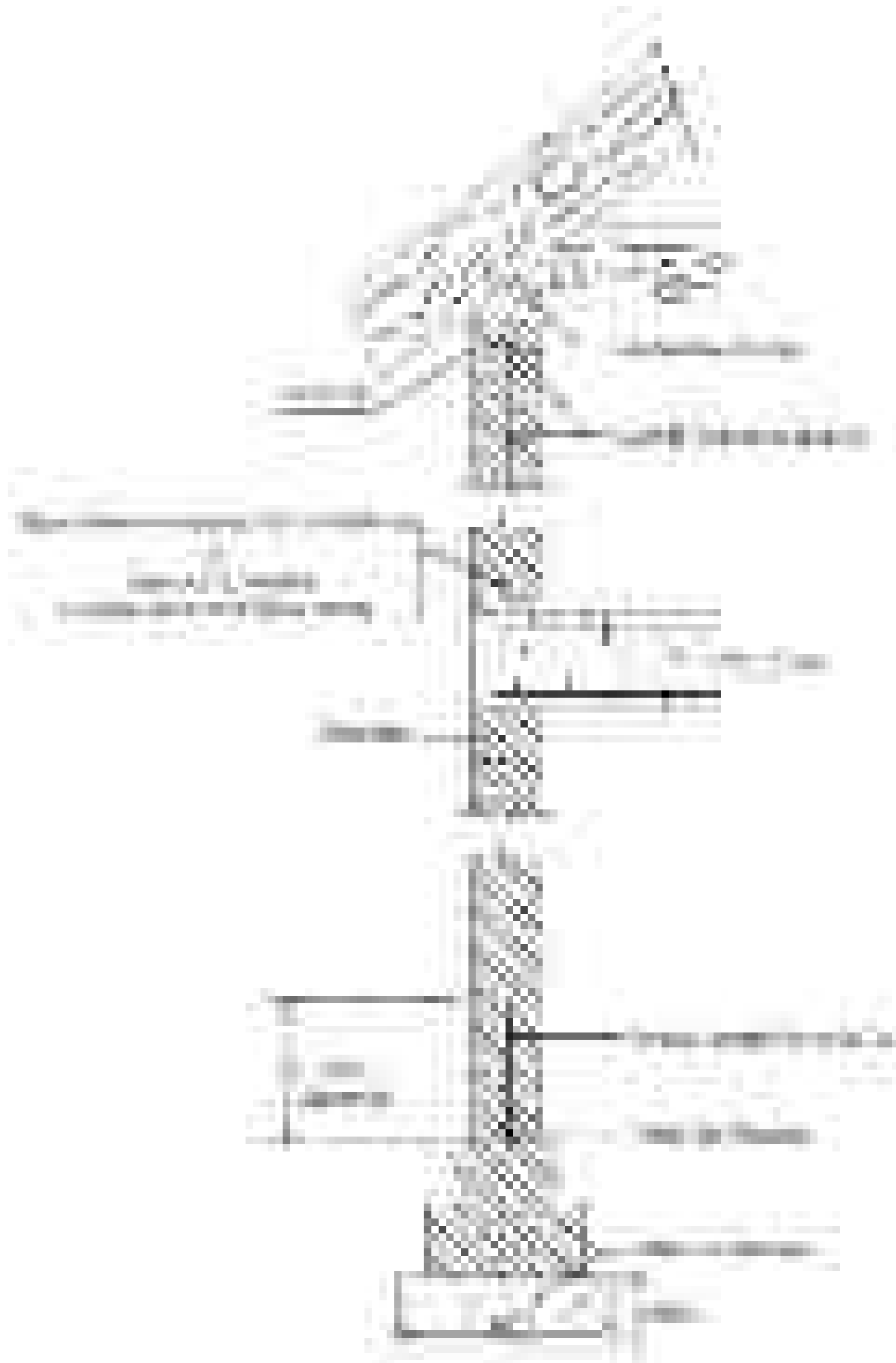


Figure 6.7.4 Continuous tie from roof to foundation of masonry structure

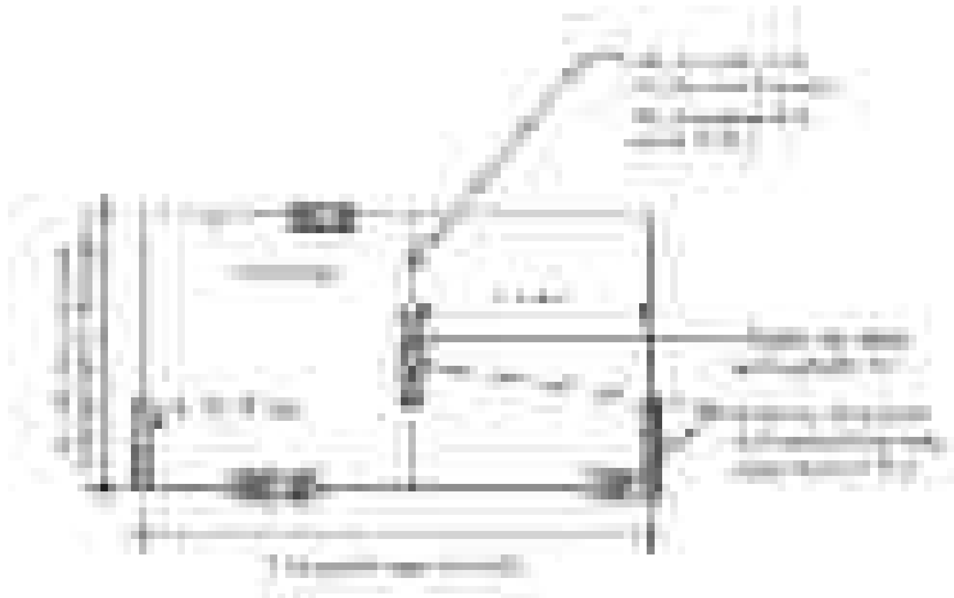


Figure 6.7.5 Masonry walls required in high wind regions

7.10 Construction

7.10.1 General

Masonry shall be constructed according to the provisions of this Section.

7.10.2 Storage and Preparation of Construction Materials

Storage, handling and preparation at the site shall conform to the following:

(a) Masonry materials shall be stored in such a way that at the time of use the materials are clean and structurally suitable for the intended use.

(b) All metal reinforcement shall be free from loose rust and other coatings that inhibit reinforcing bond.

(c) Burnt clay units shall have a rate of absorption per minute not exceeding 1 litre/m^2 at the time of lying. In the absorption test the surface of the unit shall be held 3 mm below the surface of the water.

(d) Burnt clay units shall be thoroughly wetted before placing. Concrete masonry units shall not be wetted unless otherwise approved.

(e) Materials shall be stored in such a manner that deterioration or intrusion of foreign materials is prevented and at the time of mixing the material conforms to the applicable requirements.

(f) The method of measuring materials for mortar and grout shall be such that proportions of the materials can be easily controlled.

(g) Mortar or grout mixed at the job site shall be mixed for a period of time not less than 3 minutes or more than 10 minutes in a mechanical mixer with the amount of water required to provide the desired workability. Hand mixing of small amounts of mortar is permitted. Mortar may be retempered. Mortar or grout which has hardened or stiffened due to hydration of the cement shall not be used, but under no case shall mortar be used two and one-half hours, nor grout used one and one-half hours, after the initial mixing water has been added to the dry ingredients at the job site.

7.10.3 Placing Masonry Units

(a) The mortar shall be sufficiently plastic and units shall be placed with sufficient pressure to extrude mortar from the joint and produce a tight joint. Deep furrowing which produces voids shall not be used.

The initial bed joint thickness shall not be less than 5 mm or more than 25 mm; subsequent bed joints shall be not less than 5 mm or more than 15 mm in thickness.

(b) All surfaces in contact with mortar or grout shall be clean and free of deleterious materials.

(c) Solid masonry units shall have full head and bed joints.

(d) All head and bed joints shall be filled solidly with mortar for a distance from the face of the unit not less than the thickness of the shell. Head joints of open-end units with beveled ends need not be mortared. The beveled ends shall form a grout key which permits grout within 16 mm of the face of the unit. The units shall be tightly butted to prevent leakage of grout.

7.10.4 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 3m height.

(b) Deviation in verticality in total height of any wall of a building more than one storey in height shall not exceed 12 mm.

(c) Deviation from position shown on plan of any brickwork shall not exceed 12 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.

7.10.5 Reinforcement Placing

Reinforcing details shall conform to the requirements of Sec 7.6.6. Metal reinforcement shall be located in accordance with the plans and specifications. Reinforcement shall be secured against displacement prior to grouting by wire positioners or other suitable devices at intervals not exceeding 20 bar diameters.

Tolerances for the placement of steel in walls and flexural elements shall be ± 12 mm for $d \leq 200$ mm, ± 25 mm for 200 mm $\leq d \leq 600$ mm and $\pm$ 30 mm for $d >$ 600 mm. Tolerance for longitudinal location of reinforcement shall be $\pm$ 50 mm.

7.10.6 Grouted Masonry

Grouted masonry shall be constructed in such a manner that all elements act together as a structural element.

Space to be filled with grout shall be clean and shall not contain any foreign materials. Grout materials and water content shall be controlled to provide adequate workability and shall be mixed thoroughly. The grouting of any section of wall shall be completed in one day with no interruptions greater than one hour.

Size and height limitations of the grout space or cell shall not be less than those shown in Table 6.7.19. Higher grout pours or smaller cavity widths or cell size than shown in Table 6.7.19 may be used when approved, if it can be demonstrated that grout spaces are properly filled.

Cleanouts are required for all grout pours over 1.5 m in height. When required, cleanouts shall be provided in the bottom course at every vertical bar but shall not be spaced more than 800 mm on centre for solidly grouted masonry. When cleanouts are required, they shall be sealed after inspection and before grouting. When cleanouts are not provided, special provisions must be made to keep the bottom and sides of the grout spaces, as well as the minimum total clear area as required by Table 6.7.19, clean and clear prior to grouting.

Table 6.7.19: Grouting Limitations

Grout Type	Grout pour Maximum Height (m)	Minimum Dimensions of the Total Clear Areas within Grout Spaces and Cells	
		Multi-wythe Masonry (mm)	Hollow Unit Masonry (mm)
Fine	0.30	20	40×50
	1.50	40	40×50
	2.40	40	40×75
	3.65	40	45×75
	7.30	50	75×75
Coarse	0.30	40	40×75
	1.50	50	60×75
	2.40	50	75×75
	3.65	60	75×75
	7.30	75	75×100

7.10.7 Chases, Recesses and Holes

(a) Chases, recesses and holes may be permitted in masonry provided either they are considered in the structural design or they are not cut into walls made of hollow or perforated units, or vertical chases are planned instead of horizontal chases.

(b) Depth of vertical and horizontal chases in load bearing walls shall not exceed one-third and one-sixth of the wall thickness respectively.

(c) Vertical chases shall not be closer than 2 m in any stretch of wall and shall not be located within 350 mm of an opening or within 230 mm of a cross wall that serves as stiffening wall for stability. Width of a vertical chase shall not exceed the thickness of wall in which it occurs.

(d) Horizontal chases shall be located in the upper or lower middle third height of wall at a distance not less than 600 mm from lateral support. No horizontal chase shall exceed one metre 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.

(e) Lintel shall not be used to support masonry directly above a recess or a hole wider than 300 mm. 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 adequate length of masonry on the sides of openings to resist the horizontal thrust.

(f) Recesses and holes in masonry shall be kept at the time of construction so as to avoid subsequent cutting. If cutting is necessary, it shall be done using sharp tools without causing heavy impact and damage to the surrounding areas.

(g) No chase, recess or hole shall be provided in half-brick load bearing wall, excepting the minimum number of holes needed for scaffolding.

7.11 Confined Masonry

7.11.1 General

Confined masonry construction consists of masonry walls (made either of clay brick or concrete block units) and horizontal and vertical RC confining members built on all four sides of a masonry wall panel. Vertical members, called tie-columns or practical columns, resemble columns in RC frame construction except that they tend to be of far smaller cross-section. Horizontal elements, called tie-beams, resemble beams in RC frame construction. To emphasize that confining elements are not beams and columns, alternative terms horizontal ties and vertical ties could be used instead of tie-beams and tie-columns.

The confining members are effective in

(a) Enhancing the stability and integrity of masonry walls for in-plane and out-of-plane earthquake loads (confining members can effectively contain damaged masonry walls),

(b) Enhancing the strength (resistance) of masonry walls under lateral earthquake loads, and

(c) Reducing the brittleness of masonry walls under earthquake loads and hence improving their earthquake performance.

The structural components of a confined masonry building are (see Figure 6.7.6):

(a) Masonry walls – transmit the gravity load from the slab(s) above down to the foundation. The walls act as bracing panels, which resist horizontal earthquake forces. The walls must be confined by concrete tie-beams and tie-columns to ensure satisfactory earthquake performance.

(b) Confining elements (tie-columns and tie-beams) - provide restraint to masonry walls and protect them from complete disintegration even in major earthquakes.

These elements resist gravity loads and have important role in ensuring vertical stability of a building in an earthquake.

(a) Floor and roof slabs - transmit both gravity and lateral loads to the walls. In an earthquake, slabs behave like horizontal beams and are called diaphragms.

(b) Plinth band - transmits the load from the walls down to the foundation. It also protects the ground floor walls from excessive settlement in soft soil conditions.

(c) Foundation - transmits the loads from the structure to the ground.

The design of confined masonry members shall be based on similar assumptions to those set out for unreinforced and for reinforced masonry members. Confined masonry shall be constructed according to the provisions of this Section.

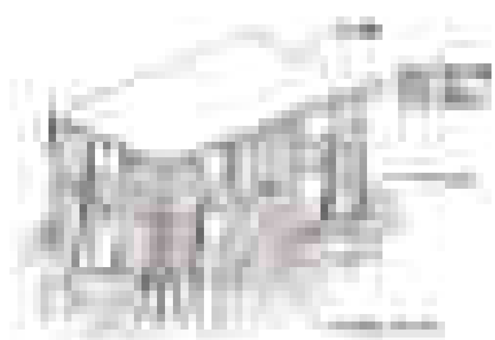


Figure 6.7.6 Typical confined masonry building

7.11.2 Difference of Confined Masonry from RC Frame Construction

The appearance of a finished confined masonry construction and a RC frame construction with masonry in fills may look alike, however these two construction systems are substantially different. The main differences are related to the construction sequence, as well as to the manner in which these structures resist gravity and lateral loads. These differences are summarized in Table 6.7.20 and are illustrated by diagrams in Figure 6.7.7.

In confined masonry construction, confining elements are not designed to act as a moment-resisting frame; as a result, detailing of reinforcement is simple. In general, confining elements have smaller cross-sectional dimensions than the corresponding beams and columns in a RC frame building. It should be noted that the most important difference between the confined masonry walls and infill walls is that infill walls are not load-bearing walls, while the walls in a confined masonry building are.

A transition from RC frame to confined masonry construction in most cases leads to savings related to concrete cost, since confining elements are smaller in size than the corresponding RC frame members.

Table 6.7.20: Comparison between confined masonry and RC frame construction

Component	Confined masonry construction	RC frame construction
Gravity and lateral load-resisting system	Masonry walls are the main load bearing elements and are expected to resist both gravity and lateral loads. Confining elements (tie-beams and tie-columns) are significantly smaller in size than RC beams and columns.	RC frames resist both gravity and lateral loads through their relatively large beams, columns, and their connections. Masonry in fills are not load-bearing walls.
Foundation construction	Strip footing beneath the wall and the RC plinth band	Isolated footing beneath each column
Superstructure construction sequence	1. Masonry walls are constructed first. 2. Subsequently, tie-columns are cast in place. 3. Finally, tie-beams are constructed on top of the walls, simultaneously with the floor/roof slab construction.	1. The frame is constructed first. 2. Masonry walls are constructed at a later stage and are not bonded to the frame members; these walls are nonstructural, that is, non-load bearing walls.

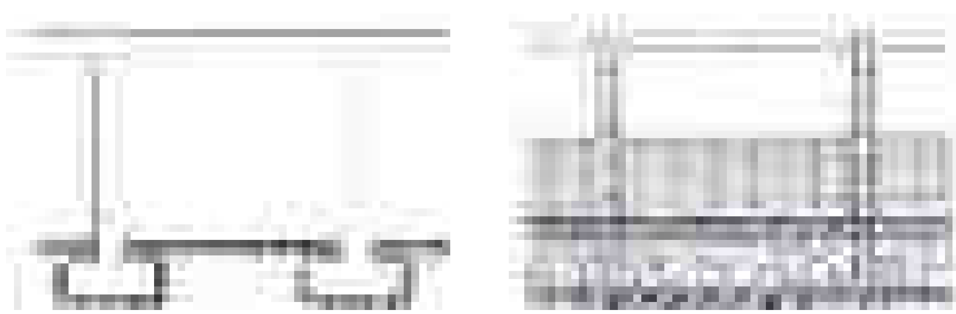
7.11.3 Mechanism of Resisting Earthquake Effects

A confined masonry building subjected to earthquake ground shaking can be modeled as a vertical truss, as shown in Figure 6.7.8. Masonry walls act as diagonal struts subjected to compression, while reinforced concrete confining members act in tension and/or compression, depending on the direction of lateral earthquake forces. This model is appropriate before the cracking in the walls takes place. Subsequently, the cracking is concentrated at the ground floor level and significant lateral deformations take place. Under severe earthquake ground shaking, the collapse of confined masonry buildings may take place due to soft storey effect similar to the one observed in RC frames with masonry in fills, as shown in Figure 6.7.8. The following failure modes are characteristic of confined masonry walls:

(a) Shear failure mode, and; (b) Flexural failure mode.

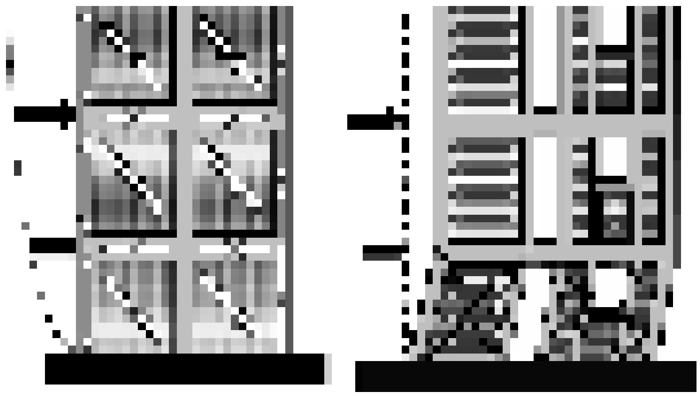
Note that, in confined masonry structures, shear failure mode develops due to in-plane seismic loads (acting along in the plane of the wall), whereas flexural failure mode may develop either due to in-plane or out-of-plane loads (acting perpendicular to the wall plane).

Shear failure mode is characterized by distributed diagonal cracking in the wall. These cracks propagate into the tie-columns at higher load levels, as shown in Figure 6.7.9. Initially, a masonry wall panel resists the effects of lateral earthquake loads by itself while the confining elements (tie-columns) do not play a significant role. However, once the cracking takes place, the wall pushes the tie-columns sideways. At that stage, vertical reinforcement in tie-columns becomes engaged in resisting tension and compression stresses. Damage in the tie-columns at the ultimate load level is concentrated at the top and the bottom of the panel. These locations, characterized by extensive crushing of concrete and yielding of steel reinforcement, are called plastic hinges (Figure 6.7.10). Note that the term plastic hinge has a different meaning in the context of confined masonry components than that referred to in relation to RC beams and columns, where these hinges form due to flexure and axial loads. In confined masonry construction, tie-beams and tie-columns resist axial loads. Shear failure can lead to severe damage in the masonry wall and the top and bottom of the tie-columns.



(a) (b)

Figure 6.7.7 (a) RC frame construction; (b) Confined masonry construction



(a) (b)

Figure 6.7.8 Confined masonry building: (a) Vertical truss model; (b) Collapse at the ground floor level

Figure 6.7.9 Shear failure of confined masonry walls

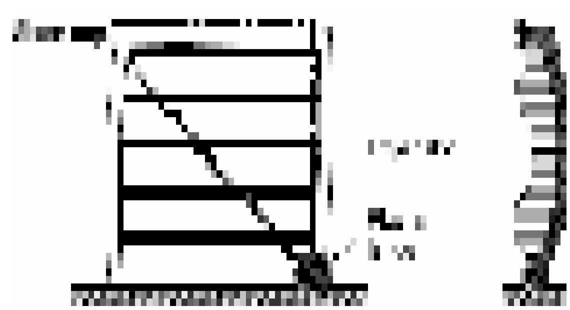


Figure 6.7.10 Plastic hinge developed in a confined masonry wall

Flexural failure caused by in-plane lateral loads is characterized by horizontal cracking in the mortar bed joints on the tension side of the wall, as shown in Figure 6.7.11. Extensive horizontal cracking, which usually takes place in tie-columns, as well as shear cracking can be observed.

Irrespective of the failure mechanism, tie-columns resist the major portion of gravity load when masonry walls suffer severe damage (this is due to their high axial stiffness and load resistance). The failure of a tie- column usually takes place when cracks propagate from the masonry wall into the tie-column and shear it off. Subsequently, the vertical stability of the entire wall is compromised. Vertical strains in the confined masonry walls decrease at an increased damage level, thereby indicating that a major portion of the gravity load is resisted by tie-columns. This finding confirms the notion that tie-columns have a critical role in resisting the gravity load in damaged confined masonry buildings and ensuring their vertical stability.

7.11.4 Key Factors Influencing Seismic Resistance

7.11.4.1 Wall density

Wall density is believed to be one of the key parameters influencing the seismic performance of confined masonry buildings. It can be determined as the transverse area of walls in each principal direction divided by the total floor area of the building.

7.11.4.2 Masonry units and mortar

The lateral load resistance of confined masonry walls strongly depends on the strength of the masonry units and the mortar used. The walls built using low-strength bricks or ungrouted hollow block units had the lowest strength while the ones built using grouted or solid units had the largest strength. However, the use of grouted and solid units results in an increase both in wall mass and seismic loads. Also, the weaker the mortar the lower the masonry strength (due to the unit-mortar interaction, the masonry strength is always lower than the unit strength). There is no significant difference in strength between unreinforced and confined masonry wall specimens with the same geometry and material properties.

7.11.4.3 Tie-columns

Tie-columns significantly influence the ductility and stability of cracked confined masonry walls. The provision of closely spaced transverse reinforcement (ties) at the top and bottom ends of tie-columns results in improved wall stability and ductility in the post-cracking stage.

7.11.4.4 Horizontal wall reinforcement

Horizontal reinforcement has a beneficial effect on wall ductility. Specimens with horizontal reinforcement showed a more uniform distribution of inclined shear cracks than the unreinforced specimens. Horizontal rebars should be anchored into the tie-columns; the anchorage should be provided with 90^0 hooks at the far end of the tie-column (Figure 6.7.12). The hooks should be embedded in the concrete within the tie-column (note that the tie-column reinforcement was omitted from the figure). The bar diameter should be larger than 3.5 mm and less than ¾ the joint thickness.

7.11.4.5 Openings

When the opening area is less than approximately 10 percent of the total wall area, the wall lateral load resistance is not significantly reduced as compared to a solid wall (i.e. wall without openings). The walls with larger openings develop diagonal cracks (same as solid walls), except that the cracks are formed in the piers between the openings; thus, diagonal struts form in the piers, as shown in Figure 6.7.13.

7.11.5 Verification of Members

7.11.5.1 In the verification of confined masonry members subjected to bending and/or axial loading, the assumptions for reinforced masonry members should be adopted. In determining the design value of the moment of resistance of a section a rectangular stress distribution may be assumed, based on the strength of the masonry, only. Reinforcement in compression should also be ignored.

7.11.5.2 In the verification of confined masonry members subjected to shear loading the shear resistance of the member should be taken as the sum of the shear resistance of the masonry and of the concrete of the confining elements. In calculating the shear resistance of the masonry the rules for unreinforced masonry walls subjected to shear loading should be used, considering the length of the masonry element. Reinforcement of confining elements should not be taken into account.

7.11.5.3 In the verification of confined masonry members subjected to lateral loading, the assumptions set out for unreinforced and reinforced masonry walls should be used. The contribution of the reinforcement of the confining elements should be considered.

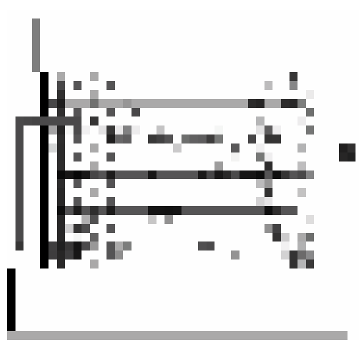


Figure 6.7.11 Flexural failure of confined masonry walls

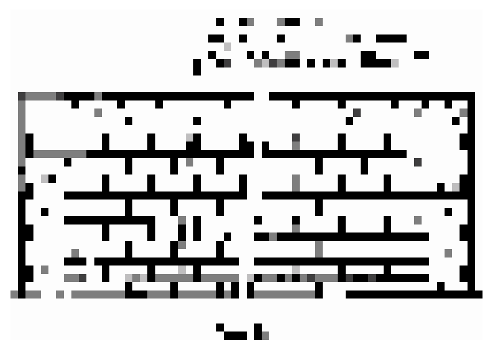


Figure 6.7.12 Horizontal reinforcement in confined masonry walls

Figure 6.7.13 Failure modes in the confined masonry walls with openings

7.11.6 Confined Masonry Members

7.11.6.1 Confined masonry members shall not exhibit flexural cracking nor deflect excessively under serviceability loading conditions.

7.11.6.2 The verification of confined masonry members at the serviceability limit states shall be based on the assumptions given for unreinforced masonry members.

7.11.7 Architectural Guideline

7.11.7.1 Building Layout

(a) The building should not be excessively long relative to its width; ideally, the length-to-width ratio should not exceed 4.

(b) The walls should be continuous up the building height.

(c) Openings (doors and windows) should be placed in the same position up the building height.

7.11.7.2 Walls

(a) At least two fully confined walls should be provided in each direction.

(b) For Seismic Zone 1 and 2, wall density of at least 2 percent in each of two orthogonal directions is required to ensure good earthquake performance of confined masonry construction. The wall density for Seismic Zones 3 and 4 should be at least 4 percent and 5 percent respectively. Wall density can be defined as the total cross sectional area of all walls in one direction divided by the sum of the floor plan areas for all floors in a building.

7.11.7.3 Building Height

Confined masonry is suitable for low- to medium-rise building construction. Confined masonry buildings will be subject to the following height restrictions:

(a) Up to 4-storey high for Seismic Zone 1 and 2

(b) Up to 3-storey high for Seismic Zone 3

(c) Up to 2-storey high for Seismic Zone 4

7.11.8 Confined Masonry Details

7.11.8.1 Confined masonry walls shall be provided with vertical and horizontal reinforced concrete or reinforced masonry confining elements so that they act together as a single structural member.

7.11.8.2 Top and sides confining elements shall be cast after the masonry has been built so that they will be duly anchored together.

7.11.8.3 Vertical confining elements should be placed:

(a) at the free edges of each structural wall element;

(b) at both sides of any wall opening with an area of more than 1.5 m^2;

(c) within the wall if necessary in order not to exceed a spacing of 5 m between the confining elements;

(d) at the intersections of structural walls, wherever the confining elements imposed by the above rules are at a distance larger than 1.5 m.

7.11.8.4 Horizontal confining elements shall be placed in the plane of the wall at every floor level and in any case with a vertical spacing of not more than 4 m.

7.11.8.5 Confining elements should have a cross-sectional area not less than 0.02 m^2, with a minimum dimension of 150 mm in the plan of the wall. In double-leaf walls the thickness of confining elements should assure the connection of the two leaves and their effective confinement.

7.11.8.6 The longitudinal reinforcement of confining elements may not have a cross-sectional area less than 300 mm^2, nor than 1 percent of the cross-sectional area of the confining element. The detailing of the reinforcements should be in accordance with Chapter 8.

7.11.8.7 Stirrups not less than 6 mm in diameter and spaced not more than 300 mm should be provided around the longitudinal reinforcement. Column ties should preferably have 135° hooks – the use of 90° hooks is not recommended. At a minimum, 6 mm ties at 200 mm spacing (6 mm@200 mm) should be provided. It is recommended to use 6 mm ties at 100 mm spacing (6 mm@100 mm) in the column end-zones (top and bottom).

7.11.8.8 To ensure the effectiveness of tie-beams in resisting earthquake loads, longitudinal bars should have a 90° hooked anchorage at intersections, as shown in Figure 6.7.14. The hook length should be at least 500 mm.

7.11.8.9 Proper detailing of the tie-beam-to-tie-column connections is a must for satisfactory earthquake performance of the entire building. Reinforcing bars must be properly anchored. A typical connection detail at the roof level is shown in Figure 6.7.15. Note that the tie-column reinforcement needs to be extended into the tie-beam as much as possible, preferably up to the underside of the top tie-beam reinforcement. A hooked anchorage needs to be provided (90° hooks) both for the tie-column and tie-beam reinforcement.

7.11.8.10 Special lintel beams may be required across larger openings having a width exceeding 1.5 m. Additional reinforcement bars need to be provided. Lintel beams can be integrated with the tie-beams at the floor level.

7.11.8.11 Lap splices may not be less than 60 bar diameters or 500 mm in length. Splicing should take place at column mid height, except for the ground floor level (where splicing is not permitted).

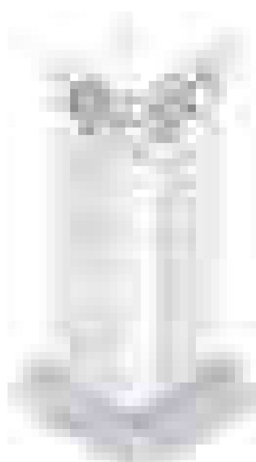
7.11.8.12 The minimum wall thickness should not be less than 100 mm. The wall height/thickness ratio should not exceed 30.

7.11.8.13 Toothed edges should be left on each side of the wall, as shown in Figure 6.7.16(a). Toothed edges are essential for adequate wall confinement, which contributes to satisfactory earthquake performance. Alternatively, when the interface between the masonry wall and the concrete tie-column needs to remain smooth for appearance's sake, steel dowels should be provided in mortar bed joints to ensure interaction between the masonry and the concrete during an earthquake, Figure 6.7.16(b).

7.11.8.14 Concrete in the tie-columns can be poured once the desired wall height has been reached. The masonry walls provide formwork for the tie-columns on two sides; however the formwork must be placed on the remaining two sides.

7.11.9 Foundation and Plinth Construction

The foundation should be constructed as in traditional brick masonry construction. Either an uncoursed random rubble stone masonry footing or a RC strip footing can be used. A RC plinth band should be constructed on top of the foundation. In confined masonry construction, plinth band is essential for preventing building settlements in soft soil areas. An alternative foundation solution with RC strip footing is also illustrated in Figure 6.7.17.

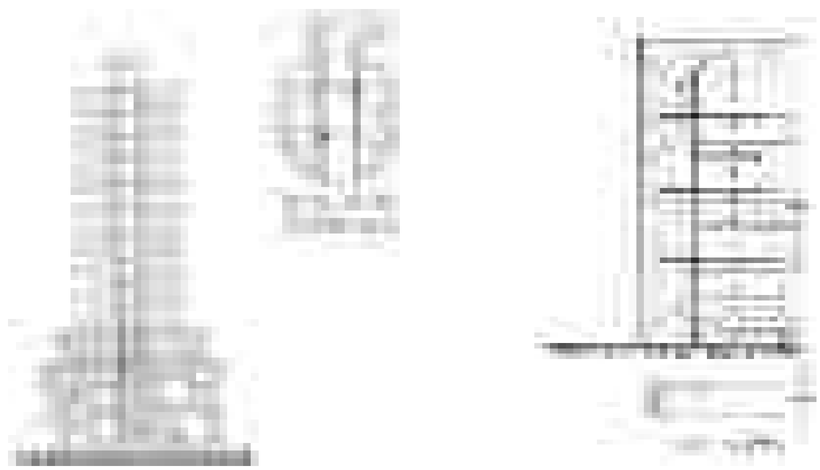


(a)

(b)

Figure 6.7.14 Tie-beam construction: (a) Wall intersections; (b) Hooked anchorage to longitudinal reinforcement

Figure 6.7.15: Detailing requirement for the tie-beam-to-tie-column connection



(a) (b)

Figure 6.7.16 (a) Toothed wall construction; (b) Horizontal dowels at the wall-to-column interface

(a) (b)

Figure 6.7.17 Foundation construction: (a) RC plinth band and stone masonry foundation; (b) RC strip footing

PART VI
Chapter 8
Detailing of Reinforcement in Concrete Structures

8.1 Introduction

Provisions of Sections 8.1 and 8.2 of Chapter 8 shall apply for detailing of reinforcement in reinforced concrete members, in general. For reinforced concrete structures, subject to earthquake loadings in seismic design categories B, C and D, special provisions contained in Sec 8.3 of this Chapter shall apply. The definitions and notation provided in the following Sections are related to Sec 8.3. The definitions and notation used in other Sections, unless otherwise mentioned, are similar to those provided in Sections 6.1.1 and 6.1.2 Chapter 6.

8.1.1 Definitions and Notation

8.1.1.1 Definitions

BASE OF STRUCTURE	The level at which earthquake motions are assumed to be imparted to a structure. This level does not necessarily coincide with the ground level.
BOUNDARY MEMBERS	Members along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement. These members do not necessarily require an increase in the thickness of the wall or diaphragm. If required, edges of openings within walls and diaphragms shall be provided with boundary members.
COLLECTOR ELEMENTS	Elements that are used to transmit the inertial forces within the diaphragms to members of the lateral force resisting systems.
CROSS TIE	A continuous bar having a hook not less than 135° with at least a six diameter extension at one end but not less than 75 mm, and a hook not less than 90° with at least a six diameter extension at the other end. The hooks shall engage peripheral longitudinal bars. The 90° hooks of two successive cross ties engaging the same longitudinal bars shall be alternated end for end.

DEVELOPMENT LENGTH OF A STANDARD HOOK	The shortest distance between the critical section and a tangent to the outer edge of the 90° hook.
HOOP	A hoop is a closed tie or continuously round tie. A closed tie can be made up of several reinforcing elements with 135° hooks having a six diameter extension at each end (but not less than 75 mm). A continuously round tie shall have at each end a 135° hook with a six diameter extension that engages the longitudinal reinforcement but not less than 75 mm.
LATERAL FORCE RESISTING SYSTEM	That portion of the structure composed of members designed to resist forces related to earthquake effects.
SHELL CONCRETE	Concrete outside the transverse reinforcement confining the concrete
STRUCTURAL DIAPHRAGMS	Structural members, such as floor and roof slabs, which transmit inertial forces to lateral force resisting members.
STRUCTURAL WALLS	Walls designed to resist combinations of shears, moments, and axial forces induced by earthquake motions. A shear wall is a structural wall.
STRUT	An element of a structural diaphragm used to provide continuity around an opening in the diaphragm.
TIE ELEMENTS	Elements used to transmit inertial forces and prevent separation of building components.

8.1.1.2 Notation

A_{ch} = Cross-sectional area of a structural member measured out to out of transverse reinforcement, mm^2

A_{cp} = Area of concrete section resisting shear of an individual pier or horizontal wall segment, mm^2

A_{cv} = Net area of concrete section bounded by web thickness and length of section in the direction of shear force considered, mm^2

A_g = Gross area of section, mm^2

A_j = Effective cross-sectional area within a joint, see Sec 8.3.7.3, in a plane parallel to plane of reinforcement generating shear in the joint. The joint depth shall be the overall depth of the column. Where a beam frames into a support of larger width, the effective width of the joint shall not exceed the smaller of :

(a) Beam width plus the joint depth

(b) twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side (See Sec 8.3.7.3)

A_{sh} = Total cross-sectional area of transverse reinforcement (including cross ties) within spacing s and perpendicular to dimension h_c

E = Load effects of earthquake or related internal moments and forces

M_{pr} = Probable flexural moment strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile strength in the longitudinal bars of at least $1.25f_y$and a strength reduction factor ϕ of 1.0, N-mm

M_s = Portion of slab moment balanced by support moment

V_c = Nominal shear strength provided by concrete, N

V_e = Design shear force corresponding to the development of the probable moment strength of the member, N

V_n = Nominal shear strength, N

V_u = Factored shear force at section, N

b = Effective compressive flange width of a structural member, mm

b_w = Web width or diameter of circular section, mm

d = Distance from extreme compression fibre to centroid of longitudinal tension reinforcement, mm

d_b = Bar diameter, mm

f_c' = Specified compressive strength of concrete, MPa

f_y = Specified yield strength of reinforcement, MPa

f_{yt} = Specified yield strength of transverse reinforcement, MPa

h = Overall thickness or height of member, mm

h_c = Cross-sectional dimension of column core measured to the outside edge of the transverse reinforcement composing area A_{sh} mm centre to centre of confining reinforcement

h_w = Height of entire wall (diaphragm) or of the segment of wall (diaphragm) considered, mm

h_x = Maximum centre to centre horizontal spacing of crossties or hoop legs on all faces of the column, mm

l_d = Development length in tension of deformed bar, deformed wire, plain and deformed welded wire reinforcement, mm

l_{dh} = Development length in tension of deformed bar or deformed wire with a standard hook, measured from critical section to outside end of hook [straight embedment length between critical section and start of hook (point of tangent) plus inside radius of bend and one bar diameter], mm

l_o = Minimum length, measured from joint face along axis of structural member, over which special transverse reinforcement must be provided, mm

l_w = Length of entire wall (diaphragm) or of segment of wall (diaphragm) considered in the direction of shear force, mm

s = Spacing of transverse reinforcement measured along longitudinal axis of the structural member, mm

s_o = Maximum spacing of transverse reinforcement, mm

α_c = Coefficient defining the relative contribution of concrete strength to wall strength

ρ = Ratio of tension reinforcement to member area = A_s/b_d

ρ_g = Ratio of total reinforcement area to cross-sectional area of column

ρ_n = Ratio of distributed shear reinforcement on a plane perpendicular to plane of A_{cv}

ρ_s = Ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out to out of spiral)

ρ_v = A_{sv}/A_{cv}; where A_{sv} is the projection on A_{cv} of area of distributed shear reinforcement crossing the plane of A_{cv}

ϕ = Strength reduction factor.

8.1.2 Standard Hooks and Minimum Bend Diameters

8.1.2.1 Standard hooks

The term "standard hook" as used in this Code shall mean one of the following:

(a) 180° bend plus an extension of at least 4 bar diameters, but not less than 65 mm at the free end of the bar.

(b) 90° bend plus an extension of at least 12 bar diameters at the free end of the bar.

(c) For stirrup and tie anchorage

(i) For 16 mm diameter bar and smaller, a 90° bend plus an extension of at least 6 bar diameters at the free end of the bar,

(ii) For 19 mm to 25 mm diameter bars, a 90° bend plus an extension of at least 12 bar diameters at the free end of the bar,

(iii) For 25 mm diameter bar and smaller, a 135° bend plus an extension of at least 6 bar diameters at the free end of the bar,

(iv) For closed ties and continuously wound ties, a 135° bend plus an extension of at least 6 bar diameters, but not less than 75 mm.

(d) Seismic hook is defined as a hook on a stirrup, hoop, or crosstie having a bend not less than 135°, except that circular hoops shall have a bend not less than 90°. Hooks shall have a six-diameter (but not less than 75 mm) extension that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop.

8.1.2.2 Minimum bend diameters

(a) The minimum diameter of bend measured on the inside of the bar, for standard hooks other than for stirrups and ties in sizes of 10 mm to 16 mm diameter shall not be less than the values shown in Table 6.8.1.

Table 6.8.1: Minimum Diameters of Bend

Bar Size	Minimum Diameter of Bend
10 mm $\leq d_b \leq$ 25 mm	$6d_b$
25 mm $< d_b \leq$ 40 mm	$8d_b$
40 mm $< d_b \leq$ 57 mm	$10d_b$

(b) For stirrups and tie hooks, inside diameter of bend shall not be less than 4 bar diameters for 16 mm diameter bar and smaller. For bars larger than 16 mm diameter, bend diameter shall be in accordance with Table 6.8.1.

(c) Inside diameter of bend in welded wire reinforcement for stirrups and ties shall not be less than 4 bar diameters for deformed wire larger than ASTM MD40 size (ASTM A1022) and 2 bar diameters for all other wires. Bends with inside diameter of less than 8 bar diameters shall not be less than 4 bar diameters from nearest welded intersection.

8.1.3 Bending

8.1.3.1 Unless otherwise permitted by the engineer, all reinforcement shall be bent cold.

8.1.3.2 Reinforcement partially embedded in concrete shall not be bent in place, except as permitted by the engineer or as shown in the design drawings.

8.1.4 Surface Conditions of Reinforcement

8.1.4.1 When concrete is placed, metal reinforcement shall be free from mud, oil, or other nonmetallic coatings that decrease bond. Epoxy-coating of steel reinforcement in accordance with standards referenced in this Code shall be permitted.

8.1.4.2 Metal reinforcement with rust, mill scale, or a combination of both, shall be considered satisfactory, provided the minimum dimensions (including height of deformations) and weight of a hand-wire-brushed test specimen are not less than applicable ASTM specification requirements.

8.1.5 Placing of Reinforcement

8.1.5.1 Reinforcement shall be accurately placed and adequately supported before concrete is placed, and shall be secured against displacement within tolerances permitted in Sec 8.1.5.2 below.

8.1.5.2 Reinforcement shall be placed within the following tolerances unless otherwise specified by the engineer:

(a) Tolerances for depth *d*, and minimum concrete cover in flexural members, walls and compression members shall be as set forth in Table 6.8.2.

Table 6.8.2: Tolerances for Placing Reinforcement

Depth of Member, d	Tolerance for d	Tolerance for Minimum Concrete Cover
$d \leq 200$ mm	$\pm$10 mm	–10 mm
$d > 200$ mm	$\pm$13 mm	–13 mm

(b) Notwithstanding the provision of (a) above, tolerance for the clear distance to formed soffits shall be minus 6 mm and tolerance for cover shall not exceed minus one third (1/3) of minimum concrete cover specified in the design drawings or specifications.

(c) Tolerance for longitudinal location of bends and ends of reinforcement shall be $\pm$ 50 mm, except at discontinuous ends of brackets and corbels, where tolerance shall be $\pm$ 13 mm and at discontinuous ends of other members, where tolerance shall be $\pm$25 mm. The tolerance for concrete cover of Sec 8.1.5.2a shall also apply at discontinuous ends of members.

8.1.5.3 Welded wire reinforcement (with ASTM wire size not greater than MW30 or MD30) used in slabs not exceeding 3 m in span shall be permitted to be curved from a point near the top of slab over the support to a point near the bottom of slab at midspan, provided such reinforcement is either continuous over, or securely anchored at support.

8.1.5.4 Welding of crossing bars shall not be permitted for assembly of reinforcement unless authorized by the engineer.

8.1.6 Spacing of Reinforcement

8.1.6.1 The minimum clear spacing between parallel bars in a layer shall be equal to one bar diameter, but not less than 25 mm, or 1.33 times of maximum nominal size of coarse aggregate, whichever is larger.

8.1.6.2 Where parallel reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above those in the bottom layer with clear distance between layers not less than 25 mm.

8.1.6.3 For compression members, the clear distance between longitudinal bars shall be not less than 1.5 bar diameters nor 40 mm nor 1.33 times of maximum nominal size of coarse aggregate.

8.1.6.4 Clear distance limitation between bars shall apply also to the clear distance between a contact lap splice and adjacent splices or bars.

8.1.6.5 In walls and one-way slabs the maximum bar spacing shall not be more than three times the wall or slab thickness h nor 450 mm.

8.1.6.6 For two-way slabs, maximum spacing of bars shall not exceed twice the slab thickness h nor 450 mm.

8.1.6.7 For temperature steel, maximum spacing shall not exceed 5 times the slab thickness h nor 450 mm.

8.1.6.8 Bundled bars

(a) Groups of parallel reinforcing bars bundled in contact to act as a single unit shall be limited to four.

(b) Bundled bars shall be enclosed within stirrups or ties.

(c) Bars larger than 32 mm diameter shall not be bundled in beams.

(d) Individual bars within a bundle terminated within the span of flexural members shall terminate at different points with at least $40d_b$ stagger.

(e) Where spacing limitations and minimum concrete cover are based on bar diameter d_b, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

8.1.7 Exposure Condition and Cover to Reinforcement

8.1.7.1 The nominal concrete cover to all reinforcement (including links), maximum free water-cement ratio and minimum cement content required for various minimum concrete strengths used in different exposure conditions shall be as specified in Table 6.8.3. However, for mild environment, the minimum concrete cover specified in Sections 8.1.7.2 and 8.1.7.3 for various structural elements may be used.

8.1.7.2 Cast-in-place concrete

(a) Minimum concrete cover for concrete cast against and permanently exposed to earth shall be 75 mm.

(b) Concrete exposed to earth or weather, the minimum clear cover shall be as under.

19 mm to 57 mm bar diameter: 50 mm

16 mm diameter bar and smaller: 40 mm

(c) The following minimum concrete cover may be provided for reinforcement for concrete surfaces not exposed to weather or in contact with ground:

	Minimum Cover
Slabs, Walls:	
40 mm to 57 mm bar diameter	40
36 mm bar diameter and smaller	20
Beams, Columns :	
Primary reinforcement, Ties, stirrups, spirals	40
Shells, folded plate members :	
19 mm bar diameter and larger	20
16 mm bar diameter and smaller	16

Table 6.8.3*: Concrete Cover and other Requirements for Various Exposure Conditions

Environment	Exposure Conditions	Minimum f'_c N/mm²						
		20	25	30	35	40	45	50
		Nominal cover (mm)						
Mild	Concrete surfaces protected against weather or aggressive conditions	30	25	20	20	20**	20**	20**

Environ ment	Exposure Conditions	Minimum f'_c N/mm^2						
		20	25	30	35	40	45	50
		Nominal cover (mm)						
Moderate	Concrete surface away from severe rain Concrete subject to condensation Concrete surfaces continuously under water Concrete in contact with non-aggressive soil	40	35	30	25	20	20	20
Severe	Concrete surfaces exposed to severe rain, alternate wetting and drying or severe condensation		45	40	30	25	25	20
Very severe	Concrete surfaces exposed to sea water spray, corrosive fumes			50	40	30	30	25
Extreme	Concrete surfaces exposed to abrasive action, e.g. sea water carrying solids or flowing water with pH $\leq$ 4.5 or machinery or vehicles				60	50	40	30
Maximum water/cement ratio		0.5	0.5	0.5	0.45	0.45	0.40	0.40
Minimum cement content, (kg/m^3)		315	325	350	375	400	410	420

* This Table relates to aggregate of 20 mm nominal maximum size.

** May be reduced to 15 mm provided the nominal maximum aggregate size does not exceed 15 mm

8.1.7.3 Precast concrete (manufactured under plant control conditions) :

(a) Concrete exposed to earth or weather:

Bar diameter	Minimum cover, mm
Wall Panels:	
40 mm to 57 mm diameter	40
36 mm diameter bar and smaller	20
Other Members:	
40 mm to 57 mm diameter	50
19 mm to 36 mm diameter	40
16 mm diameter bar and smaller	30

(b) Concrete not exposed to weather or in contact with ground:

Bar diameter	Minimum cover, mm
Slabs, Walls:	
40 mm to 57 mm diameter	30
36 mm diameter bar and smaller	16
Beams, columns :	
Primary reinforcement	$20 \leq d_b \leq 40$
Ties, stirrups, spiral	15
Shells, folded plate members :	
19 mm diameter bar and larger	16
16 mm diameter bar and smaller	10

8.1.7.4 For concrete cast against and permanently exposed to earth, minimum cover shall be 75 mm. If, concrete cover specified in Sec 8.1.7.1 (Table 6.8.3) conflicts with those specified in Sec 8.1.7.2 or Sec 8.1.7.3, the larger value shall be taken.

8.1.7.5 Bundled Bars: Minimum concrete cover shall be equal to the equivalent diameter of the bundle, but need not be greater than 50 mm.

8.1.7.6 Future Extension: Exposed reinforcement, inserts, and plates intended for bonding with future extensions shall be protected from corrosion.

8.1.7.7 Fire Protection: If a thickness of cover for fire protection greater than the concrete covers specified in Sections 8.1.7.1 to 8.1.7.6 is required, such greater thicknesses shall be specified.

8.1.7.8 Corrosive Environments: If a thickness of cover for corrosive environment or other severe exposure conditions greater than the concrete covers specified in Sections 8.1.7.1 to 8.1.7.6 is required, such greater thicknesses shall be specified. For corrosion protection, a specified concrete cover for reinforcement not less than 50 mm for walls and slabs and not less than 65 mm for other members may be used. For precast concrete members a specified concrete cover not less than 40 mm for walls and slabs and not less than 50 mm for other members may be used.

Minimum compressive strength of concrete f_c' for the corrosive environment or other severe exposure conditions shall be 25 MPa with minimum cement of 400 kg per cubic meter. Coarse aggregate shall be 20 mm down well-graded stone chips and fine aggregate shall be coarse sand of minimum FM 2.20.

For any non-structural member like drop wall, railing, fins etc., 12 mm down well graded stone chips may be used as coarse aggregate.

Use of brick chips (khoa) as coarse aggregate is strictly prohibited for the corrosive environment or other severe exposure conditions.

Water cement ratio shall be between 0.4-0.45. Potable water shall be used for all concreting.

8.1.8 Reinforcement Details for Columns

8.1.8.1 Offset Bars: Offset bent longitudinal bars shall conform to the following:

(a) The maximum slope of inclined portion of an offset bar with axis of column shall not exceed 1 in 6.

(b) Portions of bar above and below an offset shall be parallel to the axis of column.

(c) Horizontal support at offset bends shall be provided by lateral ties, spirals, or parts of the floor construction. Horizontal support provided shall be designed to resist 1.5 times the horizontal component of the computed force in the inclined portion of the offset bars. Lateral ties or spirals, if used, shall be placed not more than 150 mm away from points of bend.

(d) Offset bars shall be bent before placement in the forms (see Sec 8.1.3).

(e) Where the face of the column above is offset 75 mm or more from the face of the column below, longitudinal bars shall not be permitted to be offset bent. The longitudinal bars adjacent to the offset column faces shall be lap spliced using separate dowels. Lap splices shall conform to Sec 8.2.14.

8.1.8.2 Steel Cores: Load transfer in structural steel cores of composite compression members shall be provided by the following:

(a) Ends of structural steel cores shall be accurately finished to bear at end bearing splices, with positive provision for alignment of one core above the other in concentric contact.

(b) At end bearing splices, bearing shall be considered effective to transfer not more than 50 percent of the total compressive stress in the steel core.

(c) Transfer of stress between column base and footing shall be designed in accordance with Sec 6.8.8.

(d) Base of structural steel section shall be designed to transfer the total load from the entire composite member to the footing; or, the base shall be designed to transfer the load from the steel core only, provided ample concrete section is available for transfer of the portion of the total load carried by the reinforced concrete section to the footing by compression in the concrete and by reinforcement.

8.1.9 Lateral Reinforcement for Columns

8.1.9.1 Lateral reinforcement for compression members shall conform to the provisions of Sections 8.1.9.3 and 8.1.9.4 below and where shear or torsion reinforcement is required, shall also conform to provisions of Sec 6.4.

8.1.9.2 Lateral reinforcement requirements for composite columns shall conform to Sections 6.3.13.7 and 6.3.13.8 Chapter 6.

8.1.9.3 Spirals: Spiral reinforcement for columns shall conform to Sec 6.3.9.3 Chapter 6 and to the following:

(a) Spirals shall consist of evenly spaced continuous bar or wire of such size and so assembled as to permit handling and placing without distortion from designed dimensions.

(b) Size of spirals shall not be less than 10 mm diameter for cast-in-place construction.

(c) The minimum and maximum clear spacing between spirals shall be 25 mm and 75 mm respectively.

(d) Anchorage of spiral reinforcement shall be provided by 1.5 extra turns of spiral bar or wire at each end of a spiral unit.

(e) Splices in spiral reinforcement shall be lap splices of 48 spiral diameter for deformed uncoated bar or wire and 72 spiral diameter for other cases, but not less than 300 mm.

(f) Spirals shall extend from the top of footing or slab in any storey to the level of the lowest horizontal reinforcement in members supported above.

(g) Spirals shall extend above termination of spiral to bottom of slab or drop panel, where beams or brackets do not frame into all sides of a column.

(h) Spirals shall extend to a level at which the diameter or width of capital is 2 times that of the column, in case of columns with capitals.

(i) Spirals shall be held firmly in place and true to line.

8.1.9.4 Ties: Tie reinforcement for compression members shall conform to the following:

(a) All bars shall be enclosed by lateral ties, at least 10 mm diameter in size for longitudinal bars 32 mm diameter or smaller, and at least 12 mm diameter in size for 36 mm to 57 mm diameter and bundled longitudinal bars.

(b) Vertical spacing of ties shall not exceed 16 longitudinal bar diameters or 48 tie diameters, or the least dimension of the compression members.

(c) Ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle not more than 135^{o}. No vertical bar shall be farther than 150 mm clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie is allowed.

(d) The lowest tie in any storey shall be placed within one-half the required tie spacing from the top most horizontal reinforcement in the slab or footing below. The uppermost tie in any storey shall be within one-half the required tie spacing from the lowest horizontal reinforcement in the slab or drop panel above.

(e) Where beams or brackets provide concrete confinement at the top of the column on all (four) sides, top tie shall be within 75 mm of the lowest horizontal reinforcement in the shallowest of such beams or brackets.

(f) Where anchor bolts are placed in the top of columns or pedestals, the bolts shall be enclosed by lateral reinforcement that also surrounds at least four vertical bars of the column or pedestal. The lateral reinforcement shall be distributed within 125 mm of the top of the column or pedestal, and shall consist of at least two 12 mm diameter bars or three 10 mm diameter bars.

(g) Where longitudinal bars are arranged in a circular pattern, individual circular ties per specified spacing may be used.

8.1.10 Lateral Reinforcement for Beams

8.1.10.1 Compression reinforcement in beams shall be enclosed by ties or stirrups satisfying the size and spacing limitations in Sec 8.1.9.4 above. Such ties or stirrups shall be provided throughout the distance where compression reinforcement is required.

8.1.10.2 Lateral reinforcement for flexural framing members subject to stress reversals or to torsion at supports shall consist of closed ties, closed stirrups, or spirals extending around the flexural reinforcement.

8.1.10.3 Closed ties or stirrups shall be formed in one piece by overlapping standard stirrup or tie end hooks around a longitudinal bar, or formed in one or two pieces lap spliced with a Class B splice (lap of $1.3l_d$) or anchored in accordance with Sec 8.2.10.

8.1.11 Shrinkage and Temperature Reinforcement

8.1.11.1 Where the flexural reinforcement extends in one direction only, reinforcement for shrinkage and temperature stresses shall be provided perpendicular to flexural reinforcement in structural slabs. Shrinkage and temperature reinforcement shall be provided in accordance with Sec 8.1.11.2 below.

8.1.11.2 Deformed reinforcement conforming to Sec 5.3.2 Chapter 5 shall be provided in accordance with the following:

(a) Area of shrinkage and temperature reinforcement shall provide at least the following ratios of reinforcement area to gross concrete area:

Slabs where reinforcement with $f_y = 275$ N/mm² or 350 N/mm² are used:	0.0020
Slabs where reinforcement with $f_y = 420$ N/mm² are used:	0.0018
Slabs where reinforcement with f_y exceeding 420 N/mm² are used:	$0.0018\left(\frac{420}{f_y}\right)$

In any case, the reinforcement ratio shall not be less than 0.0014.

(b) Area of shrinkage and temperature reinforcement for brick aggregate concrete shall be at least 1.5 times that provided in (a) above.

(c) Shrinkage and temperature reinforcement shall be spaced not farther apart than 5 times the slab thickness, nor 450 mm.

(d) At all sections where required, reinforcement for shrinkage and temperature stresses shall develop the specified yield strength f_y in tension in accordance with Sec 8.2.

8.1.12 Requirements for Structural Integrity

8.1.12.1 In the detailing of reinforcement and connections, members of a structure shall be effectively tied together to improve integrity of the overall structure.

8.1.12.2 The minimum requirements for cast-in-place construction shall be:

(a) In one-way slab construction, at least one bottom bar shall be continuous or shall be spliced over the support with a Class A tension splice. At non-continuous supports, the bars may be terminated with a standard hook.

(b) Beams at the perimeter of the structure shall have at least one-sixth of the tension reinforcement required for negative moment at the support, but not less than two bars and one-quarter of the positive moment reinforcement required at midspan, but not less than two bars made continuous over the span length passing through the region bounded by the longitudinal reinforcement of the column around the perimeter and tied with closed stirrups. Closed stirrups need not be extended through any joints. The required continuity may be provided with top reinforcement spliced at mid-span and bottom reinforcement spliced at or near the support with Class B tension splices.

(c) When closed stirrups are not provided, in other than perimeter beams, at least one-quarter of the positive moment reinforcement required at mid-span, but not less than two bars shall pass through the region bounded by the longitudinal reinforcement of the column and shall be continuous or shall be spliced over the support with a Class B tension splice. At non-continuous supports the bars shall be anchored to develop f_y at the face of the support using a standard hook.

8.1.12.3 To effectively tie elements together, tension ties shall be provided in the transverse, longitudinal, and vertical directions and around the perimeter of the structure for precast concrete construction.

8.1.13 Connections

8.1.13.1 Enclosure shall be provided for splices of continuing reinforcement and for anchorage of terminating reinforcement at connections of principal framing elements (such as beams and columns),

8.1.13.2 External concrete or internal closed ties, spirals, or stirrups shall be used as enclosures at connections.

8.2 Development and Splices of Reinforcement

8.2.1 Development of Reinforcement - General

Calculated tension or compression stress in reinforcement at each section of reinforced concrete members shall be developed on each side of that section by embedment length, hook or mechanical device, or a combination thereof. Hooks may be used in developing bars in tension only.

8.2.2 Limitation

The values of $\sqrt{f_c'}$ used in Sec 8.2 shall not exceed 8.3 MPa. In addition to requirements stated here that affect detailing of reinforcement, structural integrity requirements of Sec 8.1.12 shall be satisfied.

8.2.3 Development of Deformed Bars and Deformed Wires in Tension

8.2.3.1 Development length for deformed bars and deformed wire in tension, l_d shall be determined from either Sec 8.2.3.2 or Sec 8.2.3.3 and applicable modification factors of Sections 8.2.3.4 and 8.2.3.5, but l_d shall not be less than 300 mm.

8.2.3.2 For deformed bars or deformed wire, l_d shall be as follows:

Spacing and cover	19 mm diameter and smaller bars and deformed wires	20 mm diameter and larger bars
Clear spacing of bars or wires being developed or spliced not less than d_b, clear cover not less than d_b, and stirrups or ties throughout l_d not less than the Code minimum Or, Clear spacing of bars or wires being developed or spliced not less than $2d_b$ and clear cover not less than d_b	$\left(\frac{f_y\psi_t\psi_e}{2.1\lambda\sqrt{f'_c}}\right)d_b$	$\left(\frac{f_y\psi_t\psi_e}{1.7\lambda\sqrt{f'_c}}\right)d_b$
Other cases		$\left(\frac{f_y\psi_t\psi_e}{1.1\lambda\sqrt{f'_c}}\right)d_b$

8.2.3.3 For deformed bars or deformed wire, l_d shall be

$$l_d = \left(\frac{f_y}{1.1\lambda\sqrt{f_c'}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b+K_{tr}}{d_b}\right)}\right)d_b \qquad (6.8.1)$$

In which the confinement term $\frac{c_b+K_{tr}}{d_b}$ shall not be taken greater than 2.5, and

$$K_{tr} = \frac{40A_{tr}}{sn} \qquad (6.8.2)$$

Where, n is the number of bars or wires being spliced or developed along the plane of splitting. It shall be permitted to use $K_{tr} = 0$ as a design simplification even if transverse reinforcement is present.

8.2.3.4 The factors used in the expressions for development of deformed bars and deformed wires in tension in Sec 8.2.3 are as follows:

(a) Where horizontal reinforcement is placed such that more than 300 mm of fresh concrete is cast below the development length or splice, $\psi_t = 1.3$. For other cases, $\psi_t = 1.0$.

(b) For epoxy-coated bars or wires with cover less than $3d_b$, or clear spacing less than $6d_b$, $\psi_e = 1.5$. For all other epoxy-coated bars or wires, $\psi_e = 1.2$. For uncoated and zinc-coated (galvanized) reinforcement, $\psi_e = 1.0$. However, the product $\psi_t\psi_e$ need not be greater than 1.7.

(c) For 19 mm diameter and smaller bars, and deformed wires, $\psi_s = 0.8$. For 20 mm diameter and larger bars, $\psi_s = 1.0$.

(d) Where lightweight concrete is used, λ shall not exceed 0.75 unless f_{ct} is specified (see Sec 6.1.9.1 Chapter 6). Where normal weight concrete is used, $\lambda = 1.0$.

8.2.3.5 Excess Reinforcement: Development length may be reduced by the factor $\left[\frac{A_{s\,required}}{A_{s\,provided}}\right]$ where reinforcement in a flexural member is in excess of that required by analysis except where anchorage or development for f_y is specifically required or the reinforcement is designed under the provisions of Sec 8.3.2(b).

8.2.4 Development of Deformed Bars and Deformed Wires in Compression

8.2.4.1 Development length for deformed bars and deformed wire in compression, l_{dc} shall be determined from Sec 8.2.4.2 and applicable modification factors of Sec 8.2.4.3, but l_{dc} shall not be less than 200 mm.

8.2.4.2 For deformed bars and deformed wire, l_{dc} shall be taken as the larger of $\frac{0.24 f_y d_b}{\lambda\sqrt{f_c'}}$ and $0.043 f_y d_b$ with λ as given in Sec 8.2.3.4(d) and the constant 0.043 carries the unit of mm²/N.

8.2.4.3 Length l_{dc} in Sec 8.2.4.2 shall be permitted to be multiplied by the applicable factors for:

(a) Reinforcement in excess of that required by analysis:	$\left[\frac{A_{s\,required}}{A_{s\,provided}}\right]$
(b) Reinforcement enclosed within spiral reinforcement not less than 6 mm diameter and not more than 100 mm pitch or within 12 mm diameter ties in conformance with Sec 8.1.9.4 and spaced at not more than 100 mm on center:	0.75

8.2.5 Development of Bundled Bars

8.2.5.1 Development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased 20 percent for 3 bar bundles and 33 percent for 4 bar bundles.

8.2.5.2 For determining the appropriate spacing and cover values in Sec 8.2.3.2, the confinement term in Sec 8.2.3.3, and the ψ_e factor in Sec 8.2.3.4(b), a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area and having a centroid that coincides with that of the bundled bars.

8.2.6 Development of Standard Hooks in Tension

8.2.6.1 Development length l_{dh} for deformed bars in tension terminating in a standard hook shall be computed as the product of the basic development length for deformed bars, l_{dh} of Sec 8.2.6.2 below and the applicable modification factor(s) of Sec 8.2.6.3, but l_{dh} shall be not less than $8d_b$ nor less than 150 mm.

8.2.6.2 For deformed bars, l_{dh} shall be $\frac{0.24\psi_e f_y d_b}{\lambda\sqrt{f_c'}}$ with ψ_e taken as 1.2 for epoxy-coated reinforcement, and λ taken as 0.75 for lightweight concrete. For other cases, ψ_e and λ shall be taken as 1.0.

8.2.6.3 Length l_{dh} in Sec 8.2.6.2 shall be permitted to be multiplied by the following applicable factors:

(a)	For 36 mm diameter bar and smaller hooks with side cover (normal to plane of hook) not less than 65 mm, and for 90° hook with cover on bar extension beyond hook not less than 50 mm	0.7
(b)	For 90° hooks of 36 mm diameter bar and smaller bars that are either enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along l_{dh}; or enclosed within ties or stirrups parallel to the bar being developed, spaced not greater than $3d_b$ along the length of the tail extension of the hook plus bend	0.8
(c)	For 180° hooks of 36 mm diameter bar and smaller bars that are enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along $l_{dh.}$	0.8
(d)	Where anchorage or development for f_y is not specifically required, reinforcement in excess of that required by analysis	$\left[\frac{A_{s\,required}}{A_{s\,provided}}\right]$

In Sections 8.2.6.3(b) and 8.2.6.3(c), d_b is the diameter of the hooked bar, and the first tie or stirrup shall enclose the bent portion of the hook, within $2d_b$ of the outside of the bend.

8.2.6.4 For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover over hook less than 65 mm, the hooked bar shall be enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along l_{dh}. The first tie or stirrup shall enclose the bent portion of the hook, within $2d_b$ of the outside of the bend, where d_b is the diameter of the hooked bar. For this case, the factors of Sec 8.2.6.3(b) and (c) shall not apply.

8.2.6.5 Hooks shall not be considered effective in developing bars in compression.

8.2.7 Development of Flexural Reinforcement - General

8.2.7.1 Tension reinforcement may be developed by bending across the web to be anchored or made continuous with reinforcement on the opposite face of member.

8.2.7.2 Critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates, or is bent. In addition, the provisions of Sec 8.2.8.3 shall also be satisfied.

8.2.7.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance not less than d nor less than $12d_{b,}$except at supports of simple spans and at free end of cantilevers.

8.2.7.4 Continuing reinforcement shall have an embedment length not less than the development length l_d beyond the point where the bent or terminated tension reinforcement is no longer needed to resist bending.

8.2.7.5 No flexural bar shall be terminated in a tension zone unless one of the following conditions is satisfied:

(a) V_u at the location of termination is not over two-thirds of ϕV_n.

(b) Stirrup area in excess of that normally required for shear and torsion is provided over a distance along each terminated bar or wire equal to $0.75d$ from the point of cut-off. Excess stirrup area A_v shall be not less than $\frac{0.41b_w s}{f_{yt}}$. Spacing, s shall not exceed $\frac{d}{8\beta_b}$, where β_b is the ratio of area of reinforcement cut off to total area of tension reinforcement at the section.

(c) For 36 mm diameter bar and smaller, the continuing bars provide twice the area required for flexure at the cut-off point and the shear V_u does not exceed three-quarter of ϕV_n.

8.2.7.6 Where the reinforcement stress is not directly proportional to moment, such as in sloped, stepped, or tapered footings, brackets, deep flexural members, or members in which tension reinforcement is not parallel to the compression face, adequate anchorage shall be provided for the tension reinforcement. See Sections 8.2.8.4 and 8.2.9.4 for deep flexural members.

8.2.8 Development of Positive Moment Reinforcement

8.2.8.1 At least one-third of the positive moment reinforcement in simple members and one-fourth of the positive moment reinforcement in continuous members shall extend along the same face of member into the support. In beams, such reinforcement shall extend into the support at least 150 mm.

8.2.8.2 When the flexural member is a part of the primary lateral load resisting system, positive moment reinforcement extended into the support by Sec 8.2.8.1 above shall be anchored to develop the specified yield strength f_y in tension at the face of support.

8.2.8.3 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_y by Sec 8.2.3 satisfies Eq. 6.8.3, except that Eq. 6.8.3 need not be satisfied for reinforcement terminating beyond the centreline of simple supports by a standard hook or a mechanical anchorage at least equivalent to a standard hook.

$$l_d \leq \frac{M_n}{V_u} + l_a \qquad (6.8.3)$$

Where,

M_n = nominal moment strength assuming all reinforcement at section to be stressed to f_y.

V_u = factored shear force at section

l_a = at a support, embedded length of bar beyond centre of support; at point of zero moment, shall be limited to d or $12d_b$, whichever is greater.

The value of $\frac{M_n}{V_u}$ may be increased 30 percent when the ends of reinforcement are confined by a compressive reaction.

8.2.8.4 At simple supports of deep beams, positive moment tension reinforcement shall be anchored to develop f_y in tension at the face of the support except that if design is carried out using Appendix I, the positive moment tension reinforcement shall be anchored in accordance with Sec I.4.3 Appendix I. At interior supports of deep beams, positive moment tension reinforcement shall be continuous or be spliced with that of the adjacent spans.

8.2.9 Development of Negative Moment Reinforcement

8.2.9.1 Negative moment reinforcement in a continuous, restrained, or cantilever member, or in any member of a rigid frame, shall be anchored in or through the supporting member by embedment length, hooks or mechanical anchorage.

8.2.9.2 Negative moment reinforcement shall have an embedment length into the span as required by Sections 8.2.1, 8.2.2 and 8.2.7.3.

8.2.9.3 At least one-third of the total tension reinforcement provided for negative moment at the support shall be extended beyond the point of inflection a distance not less than d, $\frac{l_n}{16}$, or $12d_b$, whichever is greater.

8.2.9.4 At interior supports of deep flexural members, negative moment tension reinforcement shall be continuous with that of the adjacent spans.

8.2.10 Development of Shear Reinforcement

8.2.10.1 Shear reinforcement shall be carried as close to compression and tension surfaces of member as cover requirements and proximity of other reinforcement permits.

8.2.10.2 The ends of single leg, simple U, or multiple U-stirrups shall be anchored by one of the following means:

(a) By a standard hook around longitudinal reinforcement for ASTM MD200 wires, and 16 mm diameter bars and smaller and for 19 mm to 25 mm diameter bars with $f_{yt} \leq 280$ N/mm^2.

(b) For 19 mm to 25 mm diameter stirrups with f_{yt} greater than 280 N/mm^2, a standard stirrup hook around a longitudinal bar plus an embedment between mid-height of the member and the outside end of the hook equal to or greater than $\frac{0.17 d_b f_{yt}}{\lambda \sqrt{f_c'}}$.

(c) For each leg of welded plain wire reinforcement forming simple U-stirrups, either: (i) Two longitudinal wires spaced at a 50 mm spacing along the member at the top of the U; or (ii) One longitudinal wire located not more than $\frac{d}{4}$ from the compression face and a second wire closer to the compression face and spaced not less than 50 mm from the first wire. The second wire shall be permitted to be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend not less than $8d_b$.

(d) For each end of a single leg stirrup of welded wire reinforcement, two longitudinal wires at a minimum spacing of 50 mm and with the inner wire at least the greater of $\frac{d}{4}$ or 50 mm from $\frac{d}{2}$. Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

(e) In joist construction, for 13 mm diameter bar and ASTM MD130 wire and smaller, a standard hook.

8.2.10.3 Each bend in the continuous portion of a simple U-stirrup or multiple U-stirrup shall enclose a longitudinal bar between anchored ends.

8.2.10.4 If extended into the region of tension, longitudinal bars bent to act as shear reinforcement shall be continuous with longitudinal reinforcement and, if extended into a region of compression, shall be anchored beyond mid-depth $\frac{d}{2}$ as specified for development length in Sec 8.2.3 for that part of f_{yt} required to satisfy Eq. 6.6.58.

8.2.10.5 Pairs of U-stirrups or ties so placed as to form a closed unit shall be considered properly spliced when length of laps are $1.3l_d$. In members at least 450 mm deep, such splices with $A_b f_{yt}$ not more than 40 kN per leg shall be considered adequate if stirrup legs extend the full available depth of member.

8.2.11 Development of Plain Bars

For plain bars, the minimum development length shall be twice that of deformed bars specified in Sections 8.2.1 to 8.2.10 above.

8.2.12 Splices of Reinforcement - General

8.2.12.1 Splices of reinforcement shall be made only as required or permitted on design drawings, or in specifications, or as authorized by the engineer.

8.2.12.2 Lap splices

(a) Lap splices shall not be used for 36 mm diameter bars and larger, except as provided in Sections 8.2.14.2 Chapter 8 and 6.8.8.2.3 Chapter 6.

(b) Lap splices of bundled bars shall be based on the lap splice length required for individual bars within the bundle, increased in accordance with Sec 8.2.5. Individual bar splices within a bundle shall not overlap. Entire bundles shall not be lap spliced.

(c) Bars spliced by noncontact lap splices in flexural members shall not be spaced transversely farther apart than one-fifth the required lap splice length, nor 150 mm.

8.2.12.3 Welded splices and mechanical connections

(a) Welded splices and other mechanical connections are allowed.

(b) Except as provided in this Code, all welding shall conform to "Structural Welding Code - Reinforcing Steel" (AWS D1.4).

(c) Welded splices shall be butted and welded to develop in tension at least 125 percent of specified yield strength f_y of the bar.

(d) A full mechanical connection shall develop in tension or compression, as required, at least 125 percent of specified yield strength f_y of the bar.

(e) Welded splices and mechanical connections not meeting the requirements of (c) or (d) above are allowed only for 16 mm diameter bar or smaller and in accordance with Sec 8.2.13.4.

8.2.13 Splices of Deformed Bars and Deformed Wire in Tension

8.2.13.1 The minimum length of lap for tension splices shall be as required for Class A or B splice, but not less than 300 mm, where the classification shall be as follows:

Class - A splice: $1.0l_d$

Class - B splice: $1.3l_d$

Where, l_d is calculated in accordance with Sec 8.2.3 to develop f_y but without the 300 mm minimum of Sec 8.2.3.1 and without the modification factor of Sec 8.2.3.5.

8.2.13.2 Lap splices of deformed bars and deformed wire in tension shall be class B splices except that Class A splices are allowed when the area of reinforcement provided is at least twice that required by analysis over the entire length of the splice, and one-half or less of total reinforcement is spliced within the required lap length.

8.2.13.3 Where area of reinforcement provided is less than twice that required by analysis, welded splices or mechanical connections used shall meet the requirements of Sec 8.2.12.3(c) or Sec 8.2.12.3(d) above.

8.2.13.4 Welded splices or mechanical connections not meeting the requirements of Sec 8.2.12.3(c) or Sec 8.2.12.3(d) shall be permitted for 16 mm diameter bars or smaller if the following requirements are met:

(a) Splices shall be staggered at least 600 mm and in such manner as to develop at every section at least twice the calculated tensile force at the section but not less than 140 N/mm^2 for total area of reinforcement provided.

(b) Spliced reinforcement stress shall be taken as the specified splice strength, in computing tensile force developed at each section, but not to exceed f_y. Unspliced reinforcement stress shall be taken as a fraction of f_y defined by the ratio of the shortest actual development length provided beyond the section to l_d but not to be taken greater than f_y .

8.2.13.5 When bars of different size are lap spliced in tension, splice length shall be the larger of l_d of larger bar and tension lap splice length of smaller bar.

8.2.13.6 Splices in tension tie members shall be made with a full welded splice or full mechanical connection in accordance with Sec 8.2.12.3(c) or (d) and splices in adjacent bars shall be staggered at least 750 mm.

8.2.14 Splices of Deformed Bars in Compression

8.2.14.1 The minimum length of lap for compression splice shall be $0.071 f_y d_b$ for f_y equal to 420 N/mm^2 or less or $(0.13 f_y - 24) d_b$ for f_y greater than 420 N/mm^2, but not less than 300 mm. For f_c' less than 21 N/mm^2, length of lap shall be increased by one-third.

8.2.14.2 When bars of different diameters are lap spliced in compression, the splice length shall be the larger of the development length, l_{dc} of the larger bar, and the compression splice length of the smaller bar. Lap splices of 40 mm 43 mm 50 mm and 57 mm diameter bars to 36 mm diameter and smaller bars shall be permitted.

8.2.14.3 Welded splices or mechanical connections used in compression shall satisfy the requirements of Sec 8.2.12.3(c) or Sec 8.2.12.3(d).

8.2.14.4 End bearing splices

(a) Compression splices for bars required to transmit compressive stress only may consist of end bearing of square cut ends held in concentric contact by a suitable device.

(b) Bar ends shall terminate in flat surfaces within 1.5^o of a right angle to the axis of the bars, and shall be fitted within 3 degrees of full bearing after assembly.

(c) End bearing splices shall be used only in members containing closed ties, closed stirrups or spirals.

8.2.15 Special Splice Requirements for Columns

8.2.15.1 Lap splices, butt welded splices, mechanical connections, or end-bearing splices shall be used with the limitations of Sections 8.2.15.2 to 8.2.15.4 below. A splice shall satisfy the requirements for all load combinations for the column.

8.2.15.2 Lap splices in columns

(a) Lap splices shall conform to Sec 8.2.14.1, Sec 8.2.14.2, and where applicable to Sec 8.2.15.2(d) or Sec 8.2.15.2(e) below, where the bar stresses due to factored loads is compressive.

(b) Where the bar stress due to factored loads is tensile and does not exceed $0.5f_y$ in tension, lap splices shall be Class B tension lap splices if more than one-half of the bars are spliced at any section, or Class A tension lap splices if half or fewer of the bars are spliced at any section and alternate lap splices are staggered by l_d.

(c) Where the bar stress due to factored loads is greater than $0.5f_y$ in tension, lap splices shall be Class B tension lap splices.

(d) In tied reinforced compression members, if throughout lap splice length ties have an effective area of at least $0.0015h_s$ in both directions, lap splice length is permitted to be multiplied by 0.83, but lap length shall not be less than 300 mm. Tie legs perpendicular to dimension h shall be used in determining effective area.

(e) For spirally reinforced compression members, lap splice length of bars within a spiral is permitted to be multiplied by 0.75, but lap length shall not be less than 300 mm.

8.2.15.3 Welded splices or mechanical connectors in columns: Welded splices or mechanical connectors in columns shall meet the requirements of Sec 8.2.12.3(c) or Sec 8.2.12.3(d).

8.2.15.4 End bearing splices in columns: End bearing splices complying with Sec 8.2.14.4 may be used for column bars stressed in compression provided the splices are staggered or additional bars are provided at splice locations. The continuing bars in each face of the column shall have a tensile strength at least $0.25f_y$ times the area of the vertical reinforcement in that face.

8.2.16 Splices of Plain Bars

For plain bars, the minimum length of lap shall be twice that of deformed bars specified in Sections 8.2.12 to 8.2.15 above.

8.2.17 Development of headed and mechanically anchored deformed bars in tension

8.2.17.1 Development length for headed deformed bars in tension, l_{dt} shall be determined from Sec 8.2.17.2. Use of heads to develop deformed bars in tension shall be limited to conditions satisfying (a) through (f):

(a) Bar f_y shall not exceed 420 MPa;

(b) Bar size shall not exceed 36 mm diameter;

(c) Concrete shall be normal weight;

(d) Net bearing area of head A_{brg} shall not be less than $4A_b$;

(e) Clear cover for bar shall not be less than $2d_b$; and

(f) Clear spacing between bars shall not be less than $4d_b$.

8.2.17.2 For headed deformed bars, development length in tension l_{dt} shall be $0.19\frac{\psi_e f_y}{\sqrt{f_c'}}d_b$, where the value of f_c' used to calculate l_{dt} shall not exceed 40 MPa, and factor ψ_e shall be taken as 1.2 for epoxy-coated reinforcement and 1.0 for other cases. Where reinforcement provided is in excess of that required by analysis, except where development of f_y is specifically required, a factor of $\frac{A_{s,required}}{A_{s,provided}}$ may be applied to the expression for l_{dt}. Length l_{dt} shall not be less than the larger of $8d_b$ and 150 mm.

8.2.17.3 Heads shall not be considered effective in developing bars in compression.

8.2.17.4 Any mechanical attachment or device capable of developing f_y of reinforcement is allowed, provided that test results showing the adequacy of such attachment or device are approved by the Engineer. Development of reinforcement shall be permitted to consist of a combination of mechanical anchorage plus additional embedment length of reinforcement between critical section and mechanical attachment or device.

8.2.18 Development of Welded Deformed Wire Reinforcement in Tension

8.2.18.1 Development length for welded deformed wire reinforcement in tension, l_d measured from the point of critical section to the end of wire shall be computed as the product of, l_d from Sec 8.2.3.2 or Sec 8.2.3.3, times welded deformed wire reinforcement factor, ψ_w from 8.2.18.2 or 8.2.18.3. It shall be permitted to reduce l_d in accordance with Sec 8.2.3.5 when applicable, but l_d shall not be less than 200 mm except in computation of lap splices by Sec 8.2.20. When using ψ_w from Sec 8.2.18.2, it shall be permitted to use an epoxy-coating factor ψ_e of 1.0 for epoxy-coated welded deformed wire reinforcement in Sections 8.2.3.2 and 8.2.3.3.

8.2.18.2 For welded deformed wire reinforcement with at least one cross wire within l_d and not less than 50 mm from the point of the critical section, ψ_w shall be the greater of $\left(\frac{f_y-240}{f_y}\right)$ and $\left(\frac{5d_b}{s}\right)$ but not greater than 1.0, where s is the spacing between the wires to be developed.

8.2.18.3 For welded deformed wire reinforcement with no cross wires within l_d or with a single cross wire less than 50 mm from the point of the critical section, ψ_w shall be taken as 1.0, and l_d shall be determined as for deformed wire.

8.2.18.4 Where any plain wires, or deformed wires larger than ASTM D 31, are present in the welded deformed wire reinforcement in the direction of the development length, the reinforcement shall be developed in accordance with Sec 8.2.19.

8.2.19 Development of Welded Plain Wire Reinforcement in Tension

Yield strength of welded plain wire reinforcement shall be considered developed by embedment of two cross wires with the closer cross wire not less than 50 mm from the point of the critical section. However, l_d shall not be less than

$$l_d = 3.3\frac{A_b}{s}\frac{f_y}{\lambda\sqrt{f_c'}} \tag{6.8.4}$$

Where l_d is measured from the point of the critical section to the outermost crosswire, s is the spacing between the wires to be developed, and λ as given in Sec 8.2.3.4(d). Where reinforcement provided is in excess of that required, l_d may be reduced in accordance with Sec 8.2.3.5. Length, l_d shall not be less than 150 mm except in computation of lap splices by Sec 8.2.21.

8.2.20 Splices of Welded Deformed Wire Reinforcement in Tension

8.2.20.1 Minimum lap splice length of welded deformed wire reinforcement measured between the ends of each reinforcement sheet shall be not less than the larger of $1.3l_d$ and 200 mm, and the overlap measured between outermost cross wires of each reinforcement sheet shall be not less than 50 mm, where l_d is calculated in accordance with Sec 8.2.18 to develop f_y.

8.2.20.2 Lap splices of welded deformed wire reinforcement, with no cross wires within the lap splice length, shall be determined as for deformed wire.

8.2.20.3 Where any plain wires, or deformed wires larger than ASTM MD200, are present in the welded deformed wire reinforcement in the direction of the lap splice or where welded deformed wire reinforcement is lap spliced to welded plain wire reinforcement, reinforcement shall be lap spliced in accordance with Sec 8.2.21.

8.2.21 Splices of Welded Plain Wire Reinforcement in Tension

Minimum length of lap for lap splices of welded plain wire reinforcement shall be in accordance with Sections 8.2.21.1 and 8.2.21.2.

8.2.21.1 Where A_s provided is less than twice that required by analysis at splice location, length of overlap measured between outermost cross wires of each reinforcement sheet shall be not less than the largest of one spacing of cross wires plus 50 mm, $1.5l_d$ and 150 mm, where l_d is calculated in accordance with Sec 8.2.19 to develop f_y.

8.2.21.2 Where A_s provided is at least twice that required by analysis at splice location, length of overlap measured between outermost cross wires of each reinforcement sheet shall not be less than the larger of $1.5l_d$ and 50 mm, where l_d is calculated in accordance with Sec 8.2.19 to develop f_y.

8.3 Earthquake-Resistant Design Provisions

8.3.1 Scope

This section contains special requirements for design and construction of reinforced concrete members of a structure for which the design forces, related to earthquake motions, have been determined on the basis of energy dissipation in the nonlinear range of response.

8.3.2 Provisions

(a) The provisions of Chapter 6, shall apply except as modified by the provisions of this Section.

(b) Structures assigned to seismic design category SDC D (see Chapter 2), all reinforced concrete structures shall satisfy the requirements of special seismic detailing as given in Sections 8.3.3 to 8.3.8 in addition to the requirements of Chapter 6. The provisions for special moment frames shall not permit the use of slab without beam as part of seismic force-resisting system.

(c) Structures assigned to SDC C (see Chapter 2), all reinforced concrete structures shall be built to satisfy the requirements of intermediate seismic detailing as given in Sec 8.3.10 in addition to the requirements of Chapter 6.

(d) Structures assigned to SDC B (see Chapter 2), all reinforced concrete structures shall be built to satisfy the requirements of ordinary detailing as given in Sec 8.3.9 in addition to the requirements of Chapter 6.

(e) Structures in lower SDCs are permitted to design with detailing provisions of higher SDCs to take advantage of lower design force levels.

8.3.3 General Requirements

8.3.3.1 Analysis and proportioning of structural members

(a) The interaction of all structural and nonstructural members shall be considered in the analysis.

(b) Rigid members which are not a part of the lateral force resisting system are allowed provided their effect on the response of the system is considered and accommodated in the structural design. Consequences of failure of structural and nonstructural members which are not a part of the lateral force resisting system shall also be considered.

(c) Structural members below base of structure required to transmit forces resulting from earthquake effects to the foundation shall also comply with the requirements of this section.

(d) All structural members which are not a part of the lateral force resisting system shall conform to Sec 8.3.9.

8.3.3.2 Strength reduction factors

Strength reduction factors shall be in accordance with Sections 6.2.3.2 to 6.2.3.4.

8.3.3.3 Concrete in special moment frames and special structural walls

Compressive strength f_c' of the concrete shall be not less than 21 N/mm^2. Specified compressive strength of light-weight concrete, f_c' shall not exceed 35MPa unless demonstrated by experimental evidence. Modification factor λ for lightweight concrete in Sec 8.3 shall be in accordance with Sec 6.1.8 unless noted otherwise.

8.3.3.4 Reinforcement in special moment frames and special structural walls

(a) Requirements of Sec 8.3.3.4 shall apply to special moment frames, special structural walls and all components of special structural walls including coupling beams and wall piers.

(b) Deformed reinforcement resisting earthquake-induced flexural and axial force, or both, shall comply with ASTM A706 Grade 420. Alternatively only BDS ISO 6935-2 Grades 300, 350, 400 and 420 or ASTM A615 Grades 275 and 420 reinforcement shall be permitted if:

(i) The actual yield strength based on mill tests does not exceed f_y by more than 125 N/mm^2 (retests shall not exceed this value by more than an additional 20 N/mm^2); and

(ii) The ratio of the actual tensile strength to the actual yield strength is not less than 1.25.

(iii) Minimum elongation in 200 mm shall be at least 14 percent for bar dia. 10 mm to 20 mm, at least 12 percent for bar dia. 22 mm through 36 mm, and at least 10 percent for bar dia. 40 mm to 60 mm.

(c) The value of f_{yt} used to compute the amount of confinement reinforcement shall not exceed 700 N/mm^2.

(d) The value of f_y or f_{yt} used in design of shear reinforcement shall conform to Sec 6.4.3.2.

8.3.3.5 Welding

Reinforcement required by factored load combinations which include earthquake effect shall not be welded except as specified in Sections 8.3.4.2(d) and 8.3.5.3(b). In addition, welding shall not be permitted on stirrups, ties, inserts, or other similar elements to longitudinal reinforcement required by design.

8.3.4 Flexural Members of Special Moment Frames

8.3.4.1 Scope

Requirements of this section shall apply to special moment frame members; (i) resisting earthquake induced forces, and (ii) proportioned primarily to resist flexure. These frame members shall also satisfy the following conditions. The requirements are also shown in Figure 6.8.1.

(a) Factored axial compressive force on frame member shall not exceed $0.1A_g f_c'$.

(b) Clear span for the member, l_n shall not be less than four times its effective depth.

(c) The width to depth ratio shall be at least 0.3.

(d) The width shall not be (i) less than 250 mm and (ii) more than the width of the supporting member (measured on a plane perpendicular to the longitudinal axis of the flexural member) plus distances on each side of the supporting member neither exceeding three-fourths of the depth of the flexural member c_1 nor width of supporting member c_2.

8.3.4.2 Longitudinal reinforcement

(a) At any section of a flexural member and for the top as well as for the bottom reinforcement, the amount of reinforcement shall be not less than $0.25\frac{f_c'}{f_y}b_w d$ or $1.4\frac{b_w d}{f_y}$ r*and* the reinforcement ratio, ρ shall not exceed 0.025 (Figure 6.8.2). At least two bars shall be provided continuously both top and bottom. The positive moment strength at the face of the joint shall be not less than one-half of the negative moment strength provided at that face as shown in Figure 6.8.2. Neither the negative nor the positive moment strength at any section along the member length shall be less than one-fourth the maximum moment strength provided at the face of either joint.

(b) Lap splices of flexural reinforcement shall be permitted only if hoop or spiral reinforcement is provided over the lap length. Maximum spacing of the transverse reinforcement enclosing the lapped bars shall not exceed $\frac{d}{4}$ nor 100 mm. Lap splices shall not be used; (i) within the joints, (ii) within a distance of twice the member depth from the face of the joint, and (iii) at locations where analysis indicates flexural yielding caused by inelastic lateral displacements of the frame. These requirements are shown in Figure 6.8.3.

Welded splices and mechanical connections conforming to Sections 8.2.12.3(a) to 8.2.12.3(d) are allowed for splicing provided not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section and the centre to centre distance between splices of adjacent bars is 600 mm or more measured along the longitudinal axis of the frame member. Welded splices and mechanical connections (Type 1) shall not be used within a distance equal to twice the member depth from the column or beam faces for special moment frames or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacement.

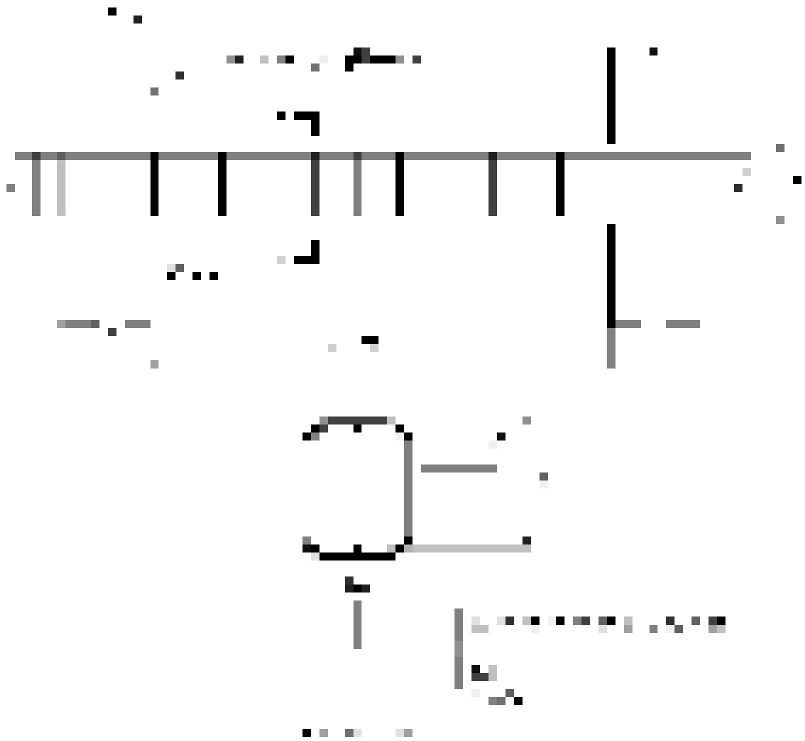
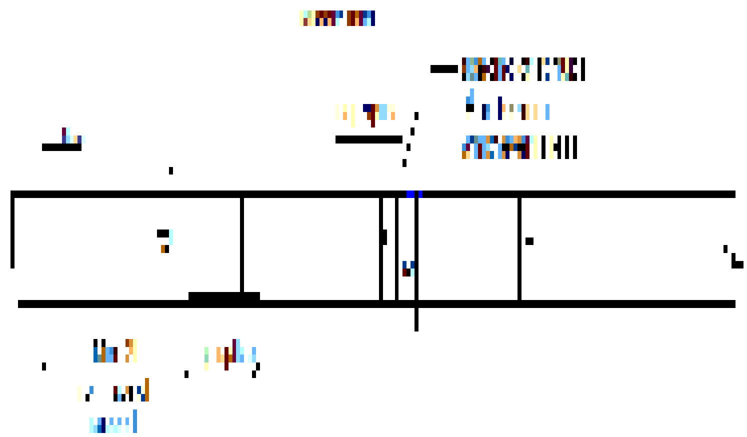


Figure 6.8.1. General requirement for flexural members of special moment frames (Sec 8.3.4.1)

Figure 6.8.2 Flexural Requirements for Flexural Members of Special Moment Frames (Sec 8.3.4.2)



Notes: (i) For beam bottom bars lap shall not be provided within a distance of twice the member depth from the face of the support; (ii) Preferred lap location of top bar is within middle third of the span but may be provided beyond 2h from the face of the support; (iii) Not more than 50% of the bars shall be spliced at one location; (iv) Lap splices are to be confined by stirrups with maximum spacing d/4 or 100 mm whichever is smaller.

Figure 6.8.3 Lap splice requirements for flexural members of special moment frames (Sec 8.3.4.2)

8.3.4.3 Transverse reinforcement

(a) Hoops shall be provided in the following regions of frame members:

(i) At both ends of the flexural member, over a length equal to twice the member depth measured from the face of the supporting member toward midspan (Figure 6.8.4).

(ii) Over lengths equal to twice the member depth (Figure 6.8.4), on both sides of a section where flexural yielding is likely to occur in connection with inelastic lateral displacements of the frame.

(b) The first hoop shall be located not more than 50 mm from the face of the supporting member (Figure 6.8.4). Maximum spacing of the hoops shall not exceed (i) $\frac{d}{4}$ (ii) eight times the diameter of the smallest longitudinal bars, (iii) 24 times the diameter of the hoop bars, and (iv) 300 mm.

(c) Where hoops are required, longitudinal bars on the perimeter shall have lateral support conforming to 8.1.9.4(c), and where hoops are not required, stirrups with seismic hooks shall be spaced not more than $\frac{d}{2}$ throughout the length of the member (Figure 6.8.4).

(d) Hoops in flexural members are allowed to be made up of two pieces of reinforcement consisting of a U-stirrup having hooks not less than 135° with 6 diameter but not less than 75 mm extension anchored in the confined core and a cross tie to make a closed hoop (Figure 6.8.5). Consecutive cross ties engaging the same longitudinal bar shall have their 90° hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the cross ties are confined by a slab only on one side of the flexural frame member, the 90° hooks of the cross ties shall all be placed on that side.

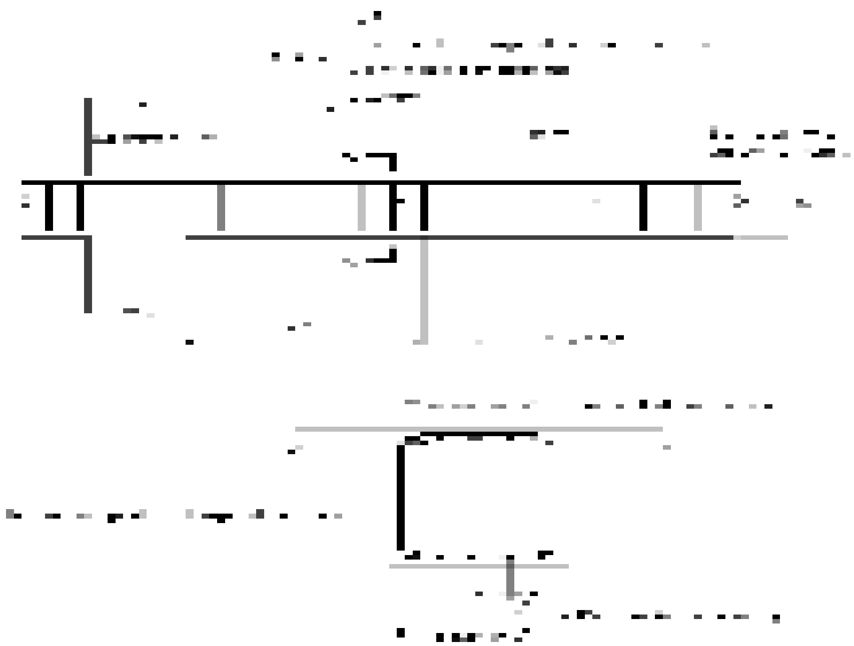


Figure 6.8.4 Transverse Reinforcement Requirements for Flexural Members of Special Moment Frames (Sec 8.3.4.3)

Figure 6.8.5 Hoop Reinforcement Requirements for Flexural Members of Special Moment Frames (Sec 8.3.4.3)

8.3.5 Special Moment Frame Members Subjected to Bending and Axial Load

8.3.5.1 Scope

The requirements of this section shall apply to columns and other frame members serving to resist earthquake forces and having a factored axial force exceeding $0.1A_g f_c'$. These frame members shall also satisfy the following conditions. The requirements are also shown in Figure 6.8.6.

(a) The shortest cross-sectional dimension shall not be less than 300 mm.

(b) The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

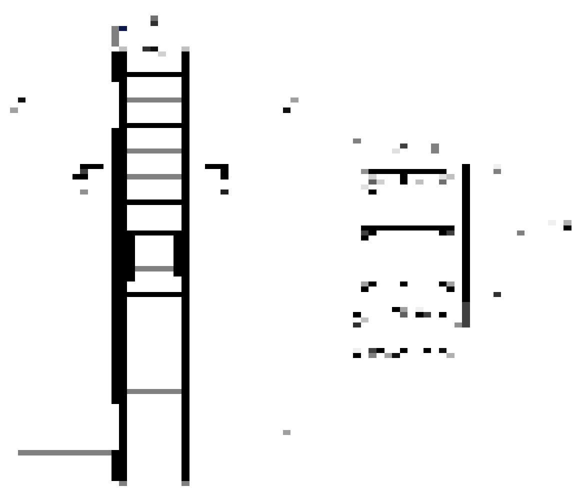


Figure 6.8.6 General requirements for special moment frames subjected to bending and axial load (Sec 8.3.5.1)

8.3.5.2 Minimum flexural strength of columns

(a) Flexural strength of any column designed to resist a factored axial compressive force exceeding $0.1A_g f_c'$ shall satisfy (b) or (c) below. Lateral strength and stiffness of columns not satisfying (b) below shall be ignored in calculating the strength and stiffness of the structure but shall conform to Sec 8.3.9.

(b) The flexural strength of the columns shall satisfy the following relation:

$$\sum M_c \geq 1.2 \sum M_g \tag{6.8.5}$$

Where,

$\sum M_c$ = sum of nominal flexural strengths of columns framing into the joint, evaluated at the face of the joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of lateral forces considered, resulting in the lowest flexural strength.

$\sum M_g$ = sum of nominal flexural strength of the beams framing into the joint evaluated at the face of the joint.

Flexural strengths shall be summed such that the column moments oppose the beam moments. Eq. 6.8.5 shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.

(c) If the requirements of (b) above is not satisfied at a joint, columns supporting reactions from that joint shall be provided with transverse reinforcement as specified in Sec 8.3.5.4 over their entire height.

8.3.5.3 Longitudinal reinforcement

The provisions of longitudinal reinforcement are as shown in Figure 6.8.7 and stated as under.

(a) The reinforcement ratio, ρ_g shall not be less than 0.01 and shall not exceed 0.06.

(b) Lap splices are permitted only within the centre half of the member length and shall be designed as tension splices. Welded splices and mechanical connections conforming to Sections 8.2.12.3(a) to 8.2.12.3(d) are allowed for splicing the reinforcement at any section provided not more than alternate longitudinal bars are spliced at a section and the distance between splices is 600 mm or more along the longitudinal axis of the reinforcement.

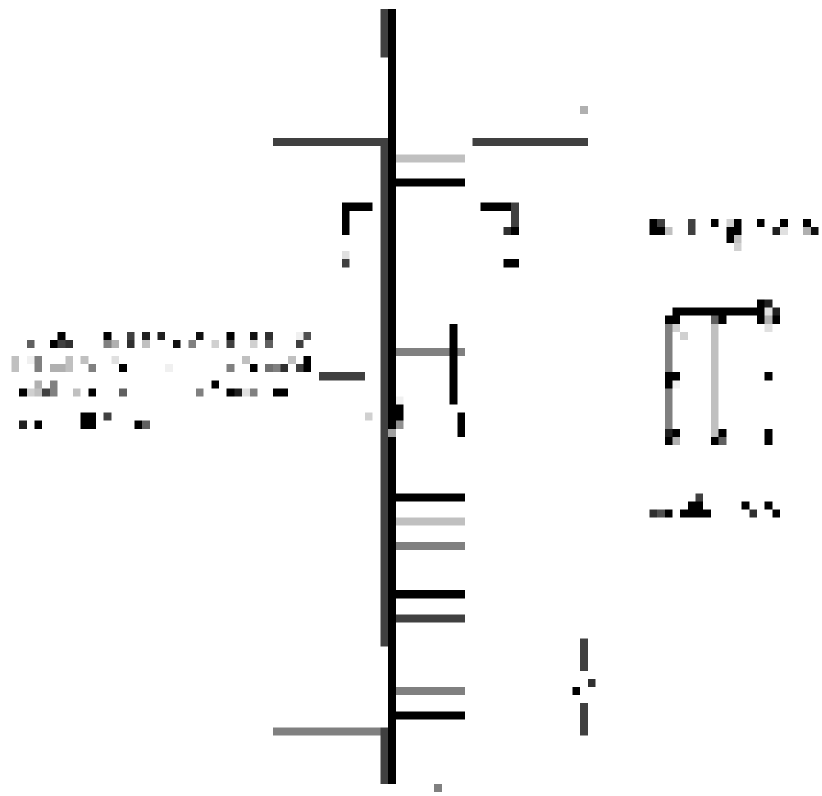


Figure 6.8.7 Longitudinal reinforcement requirements (SMF) (Sec 8.3.5.3)

8.3.5.4 Transverse reinforcement

(a) Transverse reinforcement shall be provided as specified below and shown in Figures 6.8.8 and 6.8.9 unless a larger amount is required by Sec 8.3.8.

(i) The volumetric ratio of spiral or circular hoop reinforcement, ρ_s shall not be less than that indicated by the following equation:

$$\rho_s = \frac{0.12 f_c'}{f_{yt}} \qquad (6.8.6)$$

and shall not be less than that required by Eq. (6.6.12).

(ii) The total cross-sectional area of rectangular hoop reinforcement shall not be less than that given by the following equations:

$$A_{sh} = 0.3\left(\frac{s h_c f_c'}{f_{yt}}\right)\left(\frac{A_g}{A_{ch}} - 1\right) \qquad (6.8.7)$$

$$A_{sh} = 0.09\left(\frac{s h_c f_c'}{f_{yt}}\right) \qquad (6.8.8)$$

(iii) Transverse reinforcement shall be provided by either single or overlapping hoops or cross ties of the same bar size and spacing. Each end of the cross ties shall engage a peripheral longitudinal reinforcing bar. Consecutive cross ties shall be alternated end for end along the longitudinal reinforcement.

(iv) If the design strength of member core satisfies the requirements of the specified loading combinations including earthquake effect, Eq. 6.8.7 and Eq. 6.6.12 need not be satisfied.

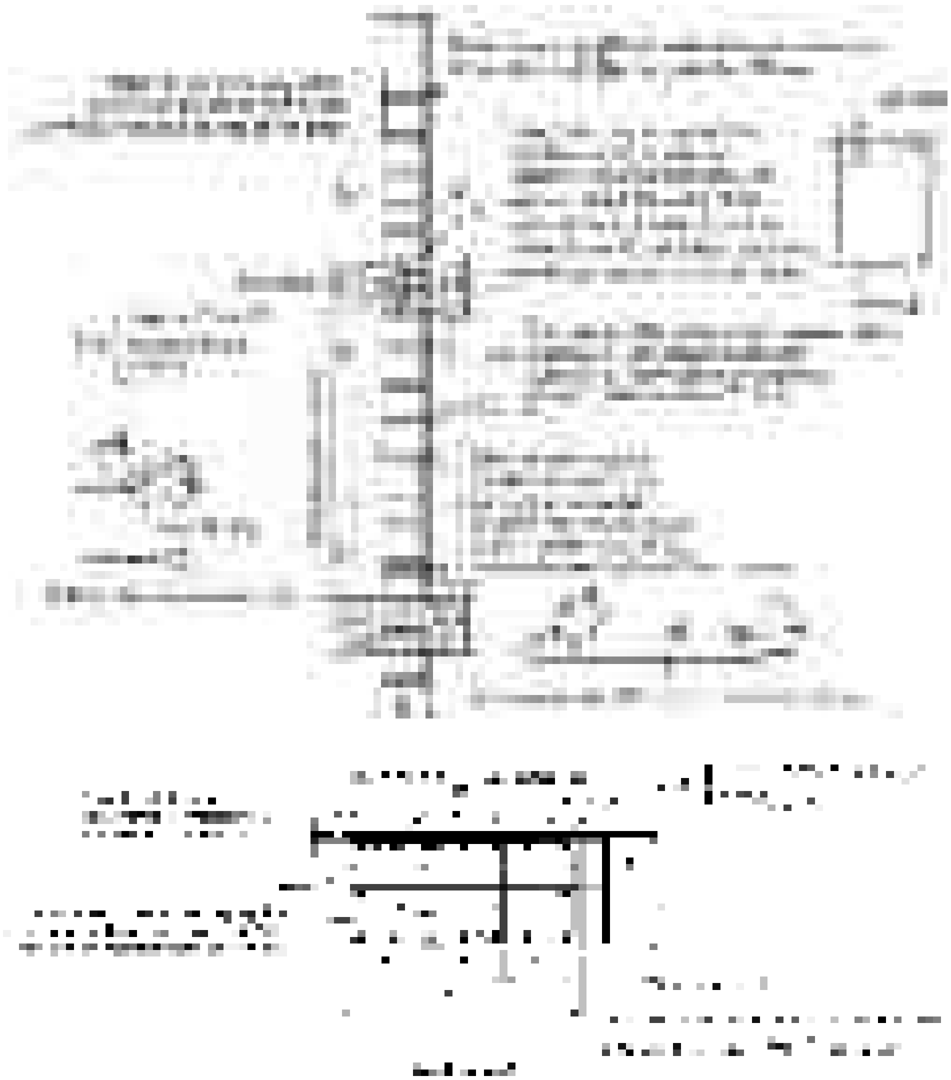
(b) Spacing of transverse reinforcement along the length l_o of the member shall not exceed the smallest of (i) one-quarter of the minimum dimension (ii) six time the diameter of the smallest longitudinal bar and (iii) $s_o = 100 + \frac{(350-h_x)}{3}$. The value of s_o shall not exceed 150 mm and need not be taken less than 100 mm.

(c) Spacing of cross ties or legs of overlapping hoops shall not be more than 350 mm on centre in the direction perpendicular to the longitudinal axis of the member.

(d) The volume of transverse reinforcement in amount specified in (a) through (c) above shall be provided over a length l_o from each joint face and on both sides of any section where flexural yielding is likely to occur in connection with inelastic lateral displacements of the frame. The length l_o shall not be less than (i) the depth of the member at the joint face or at the section where flexural yielding is likely to occur, (ii) one-sixth of the clear span of the member, and (iii) 450 mm.

(e) If the factored axial force in columns supporting reactions from discontinued stiff members, such as walls, exceeds $0.1A_g f_c'$ they shall be provided with transverse reinforcement as specified in (a) through (c) above over their full height beneath the level at which the discontinuity occurs. Transverse reinforcement shall extend into the discontinued member for at least the development length of the largest longitudinal reinforcement in the column in accordance with Sec 8.3.7.4. If the lower end of the column terminates on a wall, transverse reinforcement as specified above shall extend into the wall for at least the development length of the largest longitudinal reinforcement in the column at the point of termination. If the column terminates on a footing or mat, transverse reinforcement as specified in above shall extend at least 300 mm into the footing or mat.

(f) Where transverse reinforcement as specified in (a) through (c) above, is not provided throughout the full length of the column, the remainder of the column length shall contain spiral or hoop reinforcement with centre to centre spacing not exceeding the smaller of 6 times the diameter of the longitudinal column bars or 150 mm.



Note: In beam column joints where members frame into all four sides of the joint and each member width is at least three-fourths the column width, the spacing of transverse reinforcement shall be 150 mm within the overall depth of the shallowest frame member. For all other conditions spacing shall be S_0. Use hoops and cross ties in beam column joint.

Figure 6.8.8 Transverse reinforcement requirements- rectangular hoop for members subjected to bending and axial load rectangular hoop (SMF) (Sec 8.3.5.4)

8.3.6 Special Structural Walls and Coupling Beams

8.3.6.1 Scope: Requirements of Sec 8.3.6 apply to special structural walls and all components of special structural walls including coupling beams and wall piers forming part of the seismic-force-resisting system.

8.3.6.2 Reinforcement

(a) The distributed web reinforcement ratios, ρ_l and ρ_t, for structural walls shall not be less than 0.0025, except that if V_u does not exceed $0.083A_{cv}\lambda\sqrt{f_c'}$, ρ_l and ρ_t shall be permitted to be reduced to the values required as specified below. Reinforcement spacing each way in structural walls shall not exceed 450 mm. Reinforcement contributing to V_n shall be continuous and shall be distributed across the shear plane.

(i) Minimum ratio of vertical reinforcement area to gross concrete area, ρ_l, shall be:

Deformed bar not larger than 16 mm diameter with f_y not less than 420 MPa:	0.0012
Other deformed bars:	0.0015
Welded wire reinforcement not larger than ASTM MW 200 or MD 200:	0.0012

(ii) Minimum ratio of horizontal reinforcement area to gross concrete area, ρ_t, shall be:

Deformed bar not larger than 16 mm diameter with f_y not less than 420 MPa:	0.0020
Other deformed bars:	0.0025
Welded wire reinforcement not larger than ASTM MW 200 or MD 200:	0.0020

(b) At least two curtains of reinforcement shall be used in a wall if V_u exceeds $0.17A_{cv}\lambda\sqrt{f_c'}$.

(c) Reinforcement in structural walls shall be developed or spliced for f_y in tension in accordance with Sec 8.2, except:

(i) The effective depth of the member shall be permitted to be taken as $0.8l_w$ for walls where, reinforcement extended beyond the point at which it is no longer required to resist flexure for a distance equal to d or $12d_b$, whichever is greater, except at supports of simple spans and at free end of cantilevers.

(ii) The requirements of Sections 8.2.8, 8.2.9, and 8.2.10 need not be satisfied.

At locations where yielding of longitudinal reinforcement is likely to occur as a result of lateral displacements, development lengths of longitudinal reinforcement shall be 1.25 times the values calculated for f_y in tension.

Figure 6.8.9 Transverse reinforcement requirements- spiral hoop (SMF) (Sections 8.1.9.3, 8.3.7.2)

8.3.6.3 Design forces: V_u shall be obtained from the lateral load analysis in accordance with the factored load combinations.

8.3.6.4 Shear strength

(a) V_n of structural walls shall not exceed

$$V_n = A_{cv}\left(\alpha_c \lambda \sqrt{f_c'} + \rho_t f_y\right) \tag{6.8.9}$$

Where, the coefficient α_c is 0.25 for $h_w/l_w \leq 1.5$, is 0.17 for $h_w/l_w \geq 2.0$, and varies linearly between 0.25 and 0.17 for h_w/l_w between 1.5 and 2.0.

(b) In Sec 8.3.6.4(a), the value of ratio h_w/l_w used for determining V_n for segments of a wall shall be the larger of the ratios for the entire wall and the segment of wall considered.

(c) Walls shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall. If h_w/l_w does not exceed 2.0, reinforcement ratio ρ_l shall not be less than reinforcement ratio ρ_t.

(d) For all vertical wall segments resisting a common lateral force, combined V_n shall not be taken larger than $0.66A_{cv}\sqrt{f_c'}$, where, A_{cv} is the gross combined area of all vertical wall segments. For any one of the individual vertical wall segments, V_n shall not be taken larger than $0.83A_{cw}\sqrt{f_c'}$, where A_{cw} is the area of concrete section of the individual vertical wall segment considered.

(e) For horizontal wall segments as shown in Figure 6.8.10, including coupling beams, V_n shall not be taken larger than $0.83A_{cw}\sqrt{f_c'}$, where A_{cw} is the area of concrete section of a horizontal wall segment or coupling beam.

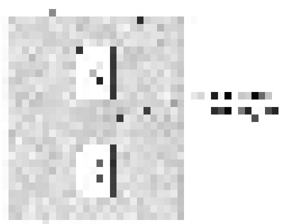


Figure 6.8.10 Wall with openings

8.3.6.5 Design for flexure and axial loads

(a) Structural walls and portions of such walls subject to combined flexural and axial loads shall be designed in accordance with Sections 6.3.2 and 6.3.3 except that Sec 6.3.3.7 and the nonlinear strain requirements of Sec 6.3.2.2 shall not apply. Concrete and developed longitudinal reinforcement within effective flange widths, boundary elements, and the wall web shall be considered effective. The effects of openings shall be considered.

(b) Unless a more detailed analysis is performed, effective flange widths of flanged sections shall extend from the face of the web a distance equal to the smaller of one-half the distance to an adjacent wall web and 25 percent of the total wall height.

8.3.6.6 Boundary elements of special structural walls

(a) The need for special boundary elements at the edges of structural walls shall be evaluated in accordance with Sec 8.3.6.6(b) or (c). The requirements of Sec 8.3.6.6(d) and (e) also shall be satisfied.

(b) This section applies to walls or wall piers that are effectively continuous from the base of structure to top of wall and designed to have a single critical section for flexure and axial loads. Walls not satisfying these requirements shall be designed by Sec 8.3.6.6(c).

(i) Compression zones shall be reinforced with special boundary elements where

$$c \geq \frac{l_w}{600(\delta_u/h_w)} \qquad (6.8.10)$$

In Eq. 6.8.10, c corresponds to the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with the design displacement δ_u. Ratio $\frac{\delta_u}{h_w}$ in Eq. 6.8.10 shall not be taken less than 0.007;

(ii) Where special boundary elements are required by b(i), the special boundary element reinforcement shall extend vertically from the critical section a distance not less than the larger of l_w or $\frac{M_u}{4V_u}$.

(c) Structural walls not designed to the provisions of (b) shall have special boundary elements at boundaries and edges around openings of structural walls where the maximum extreme fiber compressive stress, corresponding to load combinations including earthquake effects, E, exceeds $0.2f_c'$. The special boundary element shall be permitted to be discontinued where the calculated compressive stress is less than$0.15f_c'$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross section properties. For walls with flanges, an effective flange width as defined in Sec 8.3.6.5(b) shall be used.

(d) Where special boundary elements are required by Sec 8.3.6.6(b) or (c), following (i) to (v) shall be satisfied as shown in Figure 6.8.11:

(i) The boundary element shall extend horizontally from the extreme compression fiber a distance not less than the larger of $c - 0.1l_w$ and $\frac{c}{2}$, where c is the largest neutral axis depth calculated for the factored axial force and nominal moment strength consistent with δ_u;

(ii) In flanged sections, the boundary element shall include the effective flange width in compression and shall extend at least 300 mm into the web;

(iii) The boundary element transverse reinforcement shall satisfy the requirements of Sec 8.3.5.4 as shown in Figure 6.8.8, except Eq. 6.8.7 need not be satisfied and the transverse reinforcement spacing limit of 8.3.5.4.b(i) shall be one-third of the least dimension of the boundary element;

(iv) The boundary element transverse reinforcement at the wall base shall extend into the support at least l_d according to Sec 8.3.6.2(c), of the largest longitudinal reinforcement in the special boundary element unless the special boundary element terminates on a footing, mat, or pile cap, where special boundary element transverse reinforcement shall extend at least 300 mm into the footing, mat, or pile cap;

(v) Horizontal reinforcement in the wall web shall extend to within 150 mm of the end of the wall. Reinforcement shall be anchored to develop f_y in tension within the confined core of the boundary element using standard hooks or heads. Where the confined boundary element has sufficient length to develop the horizontal web reinforcement, and $\frac{A_v f_y}{s}$ of the web reinforcement is not greater than $\frac{A_{sh} f_{yt}}{s}$ of the boundary element transverse reinforcement parallel to the web reinforcement, it shall be permitted to terminate the web reinforcement without a standard hook or head.

(e) Where special boundary elements are not required by Sec 8.3.6.6(b) or (c), (i) and (ii) shall be satisfied as shown in Figure 6.8.12:

(i) If the longitudinal reinforcement ratio at the wall boundary is greater than $\frac{2.8}{f_y}$, boundary transverse reinforcement shall satisfy Sec 8.3.5.4.(a).(iii), Sec 8.3.5.4.(c) as shown in Figure 6.8.8 and Sec 8.3.6.6.(d).(i). The maximum longitudinal spacing of transverse reinforcement in the boundary shall not exceed 200 mm;

(ii) Except when V_u in the plane of the wall is less than $0.083 A_{cv} \lambda \sqrt{f_c'}$, horizontal reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

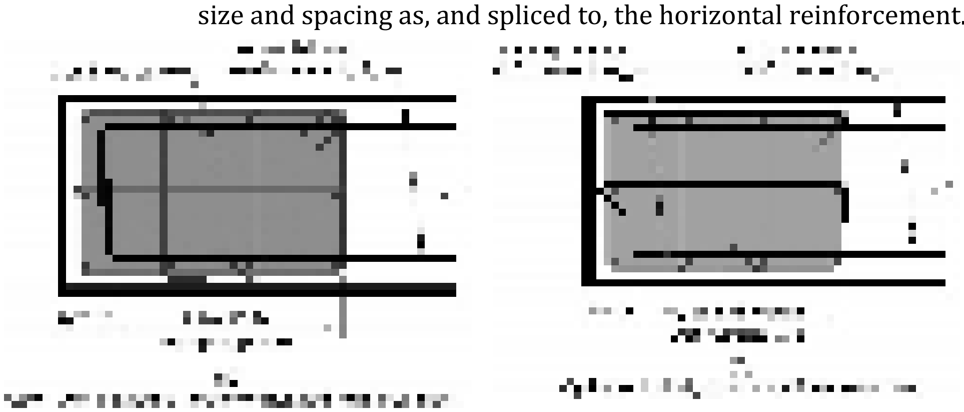


Figure 6.8.11 Development of wall horizontal reinforcement in confined boundary element

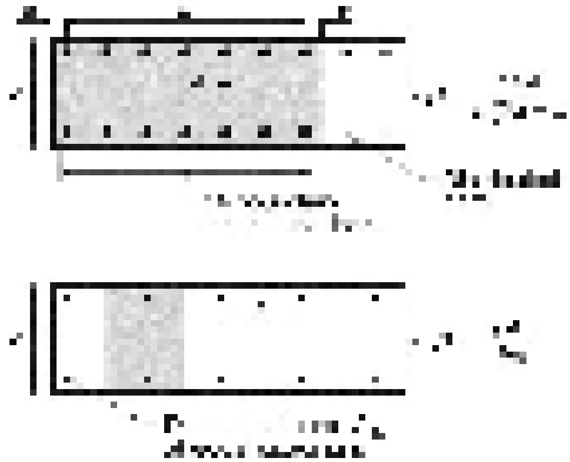


Figure 6.8.12 Longitudinal reinforcement ratios for typical wall boundary conditions.

8.3.6.7 Coupling beams

(a) Coupling beams with $\frac{l_n}{h} > 4$ shall satisfy the requirements of Sec 8.3.7. The provisions of Sec 8.3.7.1(c) and (d) need not be satisfied if it can be shown by analysis that the beam has adequate lateral stability.

(b) Coupling beams with $\frac{l_n}{h} < 2$ and with V_u exceeding $0.33A_{cw}\lambda\sqrt{f_c'}$, shall be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan, unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the vertical load-carrying ability of the structure, the egress from the structure, or the integrity of nonstructural components and their connections to the structure.

(c) Coupling beams not governed by Sec 8.3.6.7(a) or (b) shall be permitted to be reinforced either with two intersecting groups of diagonally placed bars symmetrical about the midspan or according to Sections 8.3.7.2 to 8.3.7.4.

(d) Coupling beams reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan shall satisfy (i), (ii), and either (iii) or (iv). Requirements of Sec 6.4.5 Chapter 6 shall not apply.

(i) V_n shall be determined by

$$V_n = 2A_{vd}f_y\sin\alpha \leq 0.83A_{cw}\sqrt{f_c'} \qquad (6.8.11)$$

Where, α is the angle between the diagonal bars and the longitudinal axis of the coupling beam.

(ii) Each group of diagonal bars shall consist of a minimum of four bars provided in two or more layers. The diagonal bars shall be embedded into the wall not less than 1.25 times the development length for f_y in tension.

(iii) Each group of diagonal bars shall be enclosed by transverse reinforcement having out-to-out dimensions not smaller than $\frac{b_w}{2}$ in the direction parallel to b_w and $\frac{b_w}{5}$ along the other sides, where b_w is the web width of the coupling beam. The transverse reinforcement shall satisfy Sec 8.3.5.4 as shown in Figure 6.8.8 and shall have spacing measured parallel to the diagonal bars satisfying Sec 8.3.5.4 and not exceeding six times the diameter of the diagonal bars, and shall have spacing of crossties or legs of hoops measured perpendicular to the diagonal bars not exceeding 350 mm. For the purpose of computing A_g for use in Figure 6.8.9 and Eq. 6.8.7, the concrete cover as required in Sec 8.1.7 shall be assumed on all four sides of each group of diagonal bars. The transverse reinforcement, or its alternatively configured transverse reinforcement satisfying the spacing and volume ratio requirements of the transverse reinforcement along the diagonals, shall continue through the intersection of the diagonal bars. Additional longitudinal and transverse reinforcement shall be distributed around the beam perimeter with total area in each direction not less than $0.002b_w s$ and spacing not exceeding 300 mm as shown in Figure 6.8.13(a).

(iv) Transverse reinforcement shall be provided for the entire beam cross section satisfying Sec 8.3.5.4 as shown in Figure 6.8.8, with longitudinal spacing not exceeding the smaller of 150 mm and six times the diameter of the diagonal bars, and with spacing of crossties or legs of hoops both vertically and horizontally in the plane of the beam cross section not exceeding 200 mm. Each crosstie and each hoop leg shall engage a longitudinal bar of equal or larger diameter. It shall be permitted to configure hoops as shown in Figure 6.8.13(b).

8.3.6.8 Wall piers

(a) Wall piers shall satisfy the special moment frame requirements for columns of Sec 8.3.5.3 with joint faces taken as the top and bottom of the clear height of the wall pier. Alternatively, wall piers with $\frac{l_w}{b_w} > 2.5$ shall satisfy (i) to (vi) below:

(i) Design shear force shall be determined in accordance with Sec 8.3.8.1 with joint faces taken as the top and bottom of the clear height of the wall pier. Where the Code includes provisions to account for overstrength of the seismic-force-resisting system, the design shear force need not exceed Ω_o times the factored shear determined by analysis of the structure for earthquake effects.

(ii) V_n and distributed shear reinforcement shall satisfy Sec 8.3.6.4.

(iii) Transverse reinforcement shall be in the form of hoops except it shall be permitted to use single-leg horizontal reinforcement parallel to l_w, where only one curtain of distributed shear reinforcement is provided. Single-leg horizontal reinforcement shall have 180° bends at each end that engage wall pier boundary longitudinal reinforcement.

(iv) Vertical spacing of transverse reinforcement shall not exceed 150 mm.

(v) Transverse reinforcement shall extend at least 300 mm above and below the clear height of wall pier.

(vi) Special boundary elements shall be provided if required by Sec 8.3.6.6(c).

(b) For wall piers at the edge of a wall, horizontal reinforcement shall be provided in adjacent wall segments above and below the wall pier and be proportioned to transfer the design shear force from the wall pier into the adjacent wall segments as shown in Figure 6.8.14.

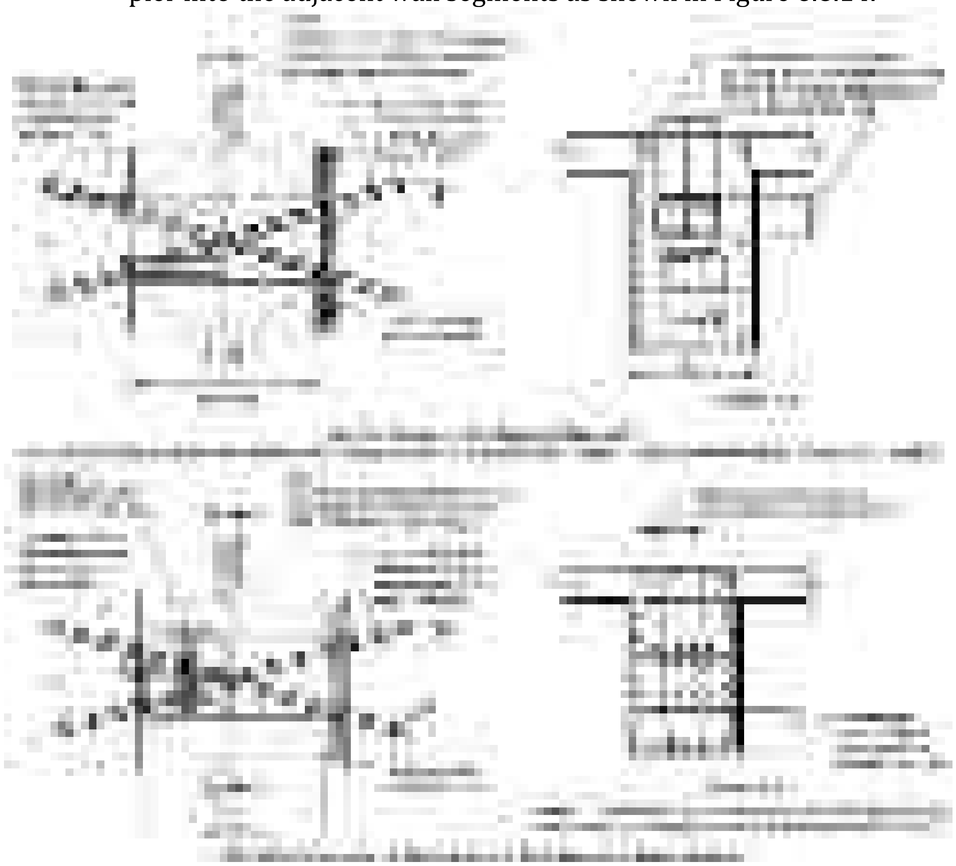


Figure 6.8.13 Coupling beams with diagonally oriented reinforcement. Wall Boundary reinforcement shown on one side only for clarity.

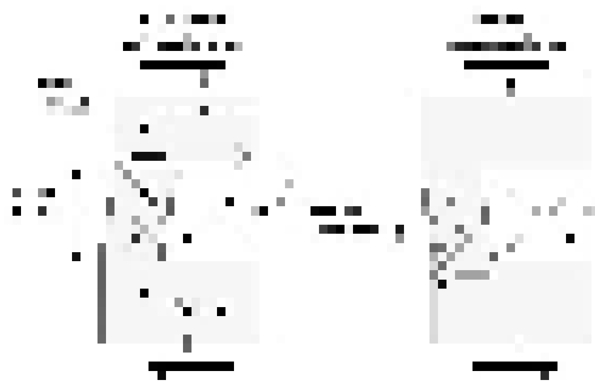


Figure 6.8.14 Required horizontal reinforcement in wall segments above and below wall piers at the edge of a wall.

8.3.6.9 Construction joints: All construction joints in structural walls shall conform to Sec 5.16.4 and contact surfaces shall be roughened as in Sec 6.4.5.9.

8.3.6.10 Discontinuous walls: Columns supporting discontinuous structural walls shall be reinforced in accordance with Sec 8.3.5.4(e).

8.3.7 Joints of Special Moment Frames

8.3.7.1 General requirements

(a) Forces in longitudinal beam reinforcement at the faces of joints of reinforced concrete frames shall be determined for a stress of $1.25f_y$ in the reinforcement.

(b) Joint strength shall be calculated by the appropriate strength reduction factors specified in Sec 6.2.3.1.

(c) Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored in tension as per Sec 8.3.7.4 below and in compression according to Sec 8.2.

(d) Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall not be less than 20 times the diameter of the largest longitudinal beam bar for normal-weight concrete. For light-weight concrete, the dimension shall not be less than 26 times the bar diameter.

8.3.7.2 Transverse reinforcement

The provisions of transverse reinforcement are shown in Figures 6.8.15 and 6.8.16, stated as under.

(a) As specified in Sec 8.3.5.4, transverse hoop reinforcement shall be provided within the joint, unless the joint is confined by structural members as specified in (b) below.

(b) Within the depth of the shallowest framing member, transverse reinforcement equal to at least one-half the amount required by Sec 8.3.5.4(a) shall be provided where members frame into all four sides of the joint and where each member width is at least three-fourths the column width. At these locations, the spacing specified in Sec 8.3.5.4(b) may be increased to 150 mm.

(c) As required by Sec 8.3.5.4, transverse reinforcement shall be provided through the joint to provide confinement for longitudinal beam reinforcement outside the column core if such confinement is not provided by a beam framing into the joint.

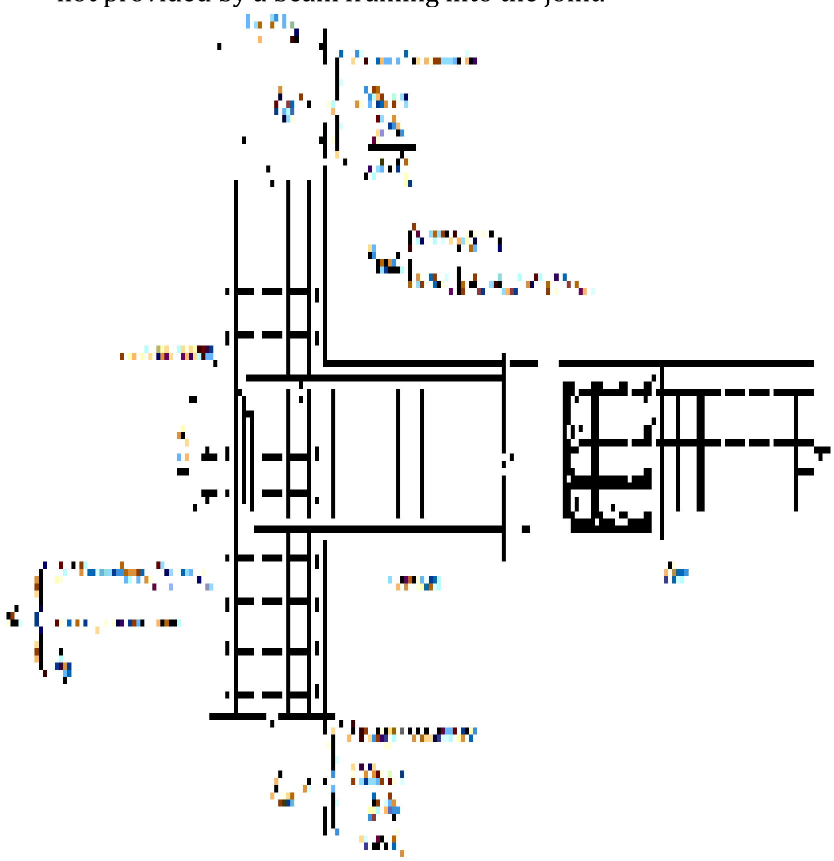


Figure 6.8.15 General requirements and transverse reinforcement requirements for joints not confined by structural member

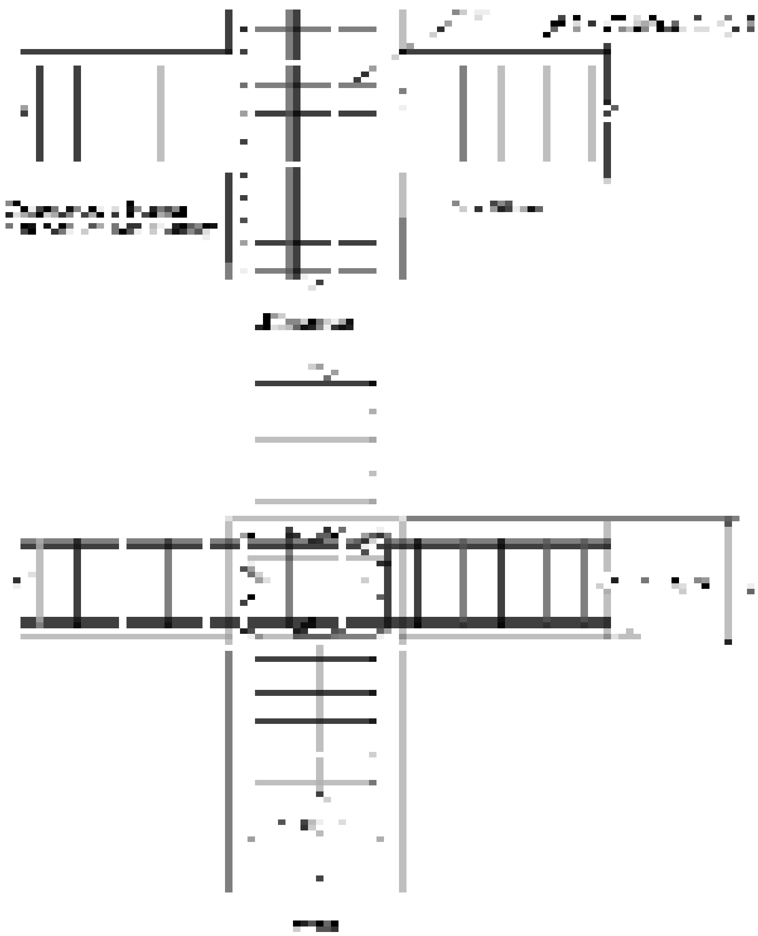


Figure 6.8.16 Transverse reinforcement requirements for joints confined by structural member

8.3.7.3 Shear Strength

The nominal shear strength for the joint shall be taken not greater than the forces specified below:

Joints confined on all four faces:	$1.7\sqrt{f_c'}\,A_j$
Joints confined on three faces or on two opposite faces:	$1.2\sqrt{f_c'}\,A_j$
Others:	$1.0\sqrt{f_c'}\,A_j$

A member that frames into a face is considered to provide confinement to the joint if at least three-quarters of the face of the joint is covered by the framing member. A joint is considered to be confined if such confining members frame into all faces of the joint.

8.3.7.4 Development length of bars in tension

(a) The development length, l_{dh}, for bar sizes 10 mm to 36 mm in diameter with a standard 90° hook shall be not less than (i) $8d_b$, (ii) 150 mm, and (iii) the length required by Eq. 6.8.9.

$$l_{dh} = \frac{f_y d_b}{5.4\sqrt{f_c'}} \qquad (6.8.12)$$

For light-weight concrete, l_{dh} for a bar with a standard 90° hook shall not be less than (i) $10d_b$, (ii) 190 mm, and (iii) 1.25 times the length required by Eq. 6.8.12. The 90° hook shall be located within the confined core of a column or a boundary element.

(b) For bar sizes 10 mm to 36 mm diameter, the development length, l_d for a straight bar shall be not less than (i) 2.5 times the length required by (a) above, if the depth of the concrete cast in one lift beneath the bar does not exceed 300 mm, and (ii) 3.5 times the length required by (a) above, if the depth of the concrete cast in one lift beneath the bar exceeds 300 mm.

(c) Straight bars terminated at a joint shall pass through the confined core of a column or of a boundary member. Any portion of the straight embedment length not within the confined core shall be increased by a factor of 1.6.

8.3.8 **Shear Strength Requirements**

8.3.8.1 Design forces

(a) Frame Members Subjected Primarily to Bending: The design shear force V_e shall be determined from consideration of the maximum forces that can be generated at the faces of the joints at each of the member. It shall be assumed that moments of opposite sign corresponding to probable strength M_{pr} act at the joint faces, and that the member is loaded with the factored tributary gravity load along its span.

(b) Frame Members Subjected to Combined Bending and Axial Load: The design shear force V_e shall be determined from consideration of the maximum forces that can be generated at the faces of the joints at each end of the member. These joint forces shall be determined using the maximum probable moment strengths M_{pr} of the member associated with the range of factored axial loads on the member. The member shears need not exceed those determined from joint strengths based on the probable moment strength M_{pr} of the transverse members framing into the joint. In no case, V_e shall be less than the factored shear determined by the analysis of the structure.

(c) Structural Walls and Diaphragms: The design shear force V_e shall be obtained from the lateral load analysis in accordance with the factored loads and combinations specified in Chapter 2, loads.

8.3.8.2 Transverse reinforcement in frame members

(a) For determining the required transverse reinforcement in frame members, the quantity V_c shall be assumed to be zero if the factored axial compressive force including earthquake effects is less than $0.05A_g f_c^{'}$ when the earthquake-induced shear forces, calculated in accordance with Sec 8.3.8.1(a), represents one-half or more of total design shear.

(b) Stirrups or ties required to resist shear shall be closed hoops over lengths of members as specified in Sections 8.3.4.3, 8.3.5.4 and 8.3.7.2.

8.3.8.3 Shear strength of special structural walls and diaphragms

(a) Nominal shear strength of structural walls and diaphragms shall be determined using either (b) or (c) below.

(b) Nominal shear strength, V_n of structural walls and diaphragms shall be assumed not to exceed the shear force calculated from

$$V_n = A_{cv}\left(0.17\lambda\sqrt{f_c'} + \rho_n f_y\right) \qquad (6.8.13)$$

(c) For walls and wall segments having a ratio of $\frac{h_w}{l_w}$ less than 2.0, nominal shear strength of wall and diaphragm shall be determined from

$$V_n = A_{cv}\left(\alpha_c\lambda\sqrt{f_c'} + \rho_n f_y\right) \qquad (6.8.14)$$

Where the coefficient α_c is 0.25 for $\frac{h_w}{l_w} \le 1.5$, is 0.17 for $\frac{h_w}{l_w} \ge 2.0$, and varies linearly between 0.25 and 0.17 for $\frac{h_w}{l_w}$ between 1.5 and 2.0.

(d) Value of ratio $\frac{h_w}{l_w}$ used in (c) above for determining V_n for segments of a wall or diaphragm shall be the larger of the ratios for the entire wall (diaphragm) and the segment of wall (diaphragm) considered.

(e) Walls and diaphragms shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall. If the ratio $\frac{h_w}{l_w}$ does not exceed 2.0, reinforcement ratio, ρ_v shall not be less than reinforcement ratio ρ_n .

(f) Nominal shear strength of all wall piers sharing a common lateral force shall not be assumed to exceed $0.67A_{cv}\sqrt{f_c'}$, where A_{cv} is the total cross-sectional area, and the nominal shear strength of any one of the individual wall piers shall not be assumed to exceed $0.83A_{cp}\sqrt{f_c'}$

Where A_{cp} represents the cross-sectional area of the pier considered.

(g) Nominal shear strength of horizontal wall segments shall be assumed not to exceed $0.83A_{cp}\sqrt{f_c'}$ where A_{cp} represents the cross-sectional area of a horizontal wall segment.

8.3.9 Ordinary Moment Frame Members not Proportioned to Resist Forces Induced by Earthquake Motion

8.3.9.1 Induced moments

Frame members assumed not to contribute to lateral resistance shall be detailed according to (a) or (b) below depending on the magnitude of moments induced in those members when subjected to twice the lateral displacement under the factored lateral forces.

(a) Members with factored gravity axial forces not exceeding $0.1A_gf_c'$ shall satisfy Sections 8.3.4.2(a) and 8.3.8.1(a) and members with factored gravity axial forces exceeding $0.1A_gf_c'$ shall satisfy Sections 8.3.5.4, 8.3.7.2(a) and 8.3.8.1(b) when the induced moment exceeds the design moment strength of the frame member.

(b) The member shall satisfy Sec 8.3.4.2(a) when the induced moment does not exceed the design moment strength of the frame members.

8.3.9.2 Tie requirements

All frame members with factored axial compressive forces exceeding $0.1A_gf_c'$ shall satisfy the following special requirements unless they comply with Sec 8.3.5.4.

(a) Ties shall have hooks not less than 135° with extensions not less than 6 tie bar diameter or 60 mm. Cross ties as defined in Sec 8.3.2 are allowed.

(b) The maximum tie spacing shall be s_0 over a length l_0 measured from the joint face. The spacing s_0 shall be not more than (i) eight diameters of the smallest longitudinal bar enclosed, (ii) 24 tie bar diameters, and (iii) one-half the least cross-sectional dimension of the column. The length l_0 shall not be less than (i) one-sixth of the clear height of the column, (ii) the maximum cross-sectional dimension of the column, and (iii) 450 mm.

(c) The first tie shall be within a distance equal to $0.5s_0$ from the face of the joint.

(d) The tie spacing shall not exceed $2s_0$ in any part of the column.

8.3.10 Requirements for Intermediate Moment Frames

8.3.10.1 Scope

For structures assigned to SDC C, structural frames proportioned to resist forces induced by earthquake motions shall satisfy the requirements of Sec 8.3.10 in addition to those of Chapter 6.

8.3.10.2 Reinforcement requirements

Reinforcement details in a frame member shall satisfy 8.3.10.4 below if the factored compressive axial load for the member does not exceed $0.1A_g f_c'$. If the factored compressive axial load is larger, frame reinforcement details shall satisfy Sec 8.3.10.5 below unless the member has spiral reinforcement according to Eq. 6.6.12. If a two-way slab system without beams is treated as part of a frame resisting earthquake effect, reinforcement details in any span resisting moments caused by lateral force shall satisfy Sec 8.3.10.6 below.

8.3.10.3 Shear requirements

Design shear strength of beams and columns resisting earthquake effect, *E,* shall not be less than the smaller of (i) sum of the shear associated with development of nominal moment strengths of the member at each restrained end of clear span and the shear calculated for factored gravity loads, or (ii) maximum shear obtained from design load combinations that include *E*, with the E assumed to be twice that prescribed by this Code.

8.3.10.4 Beams

(a) The positive moment strength at the face of the joint shall not be less than one-third the negative moment strength provided at that face (Figure 6.8.17). Neither the negative nor positive moment strength at any section along the length of the member shall be less than one-fifth of the maximum moment strength provided at the face of either joint.

(b) At both ends of the member, stirrups shall be provided over lengths equal to twice the member depth measured from the face of the supporting member toward midspan (Figure 6.8.18). The first stirrup shall be located not more than 50 mm from the face of the supporting member. Maximum stirrup spacing shall not exceed (a) $\frac{d}{4}$ (b) 8 times the diameter of the smallest longitudinal bar enclosed, (c) 24 times the diameter of the stirrup bar, and (d) 300 mm.

(c) Stirrups shall be placed at not more than $\frac{d}{2}$ throughout the length of the member.

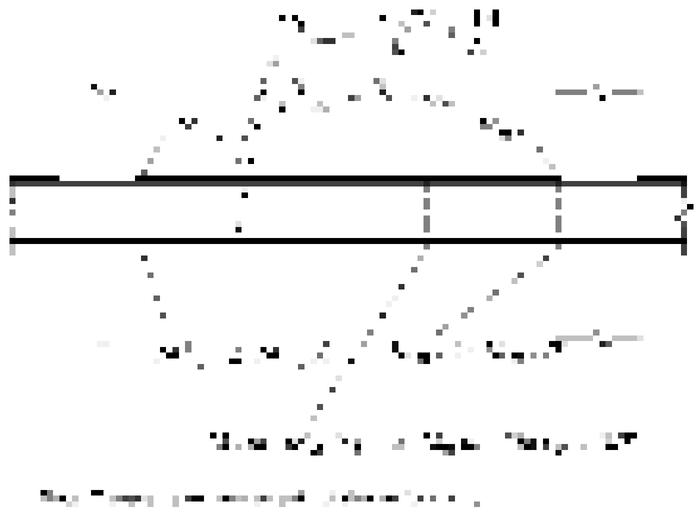


Figure 6.8.17 Flexural requirements for beams (IMF)

8.3.10.5 Columns

(a) Maximum tie spacing shall not exceed s_0 over a length l_0 measured from the joint face. The spacing s_0 shall not exceed (i) 8 times the diameter of the smallest longitudinal bar enclosed, (ii) 24 times the diameter of the tie bar, (iii) one-half of the smallest cross-sectional dimension of the frame member, and (iv) 300 mm. The length l_0 shall not be less than (i) one-sixth of the clear span of the member, (ii) maximum cross-sectional dimension of the member, and (iii) 450 mm.

(b) The first tie shall be located not more than $\frac{s_0}{2}$ from the joint face.

(c) Joint reinforcement shall conform to Sec 6.4.9.

(d) Tie spacing shall not exceed $2s_0$ throughout the length of the member.

These requirements are shown in Figure 6.8.19.

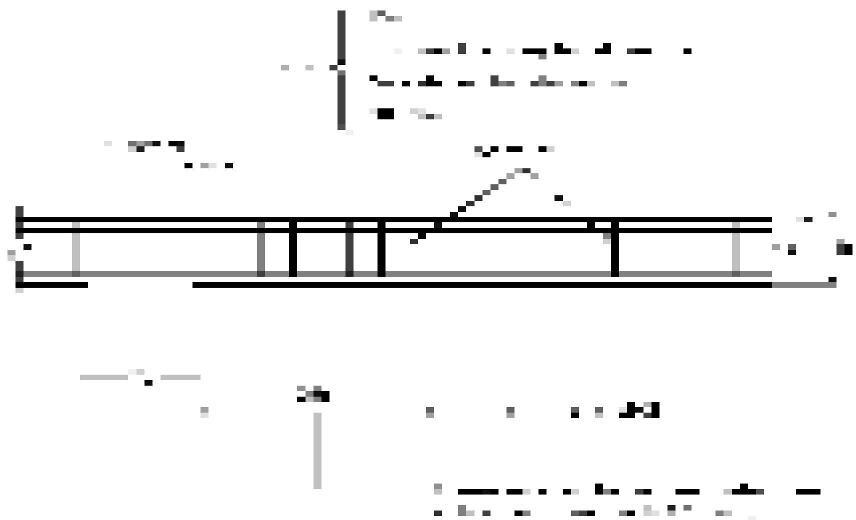


Figure 6.8.18 Transverse reinforcement requirements for beams (IMF)

8.3.10.6 Two-way slabs without beams

(a) The factored slab moment at the supports relating to earthquake effect shall be determined for load combinations specified in Chapter 2, Loads. All reinforcement provided to resist the portion of slab moment balanced by support moment shall be placed within the column strip defined in Sec 6.5.2.1 (Figure 6.8.20).

(b) The fractional part of the column strip moment shall be resisted by reinforcement placed within the effective width (Figure 6.8.20) specified in Sec 6.5.5.3.2.

(c) Not less than one-half of the total reinforcement in the column strip at the support shall be placed within the effective slab width (Figure 6.8.15) specified in Sec 6.5.5.3.2.

(d) Not less than one-quarter of the top steel at the support in the column strip shall be continuous throughout the span (Figure 6.8.21).

(e) Continuous bottom reinforcement in the column strip shall be not less than one-third of the top reinforcement at the support in the column strip.

(f) Not less than one-half of all bottom reinforcement at midspan shall be continuous and shall develop its yield strength at the face of support (Figure 6.8.22).

(g) At discontinuous edges of the slab all top and bottom reinforcement at the support shall be developed at the face of the support (Figures 6.8.21 and 6.8.22).

(h) For edge and corner connections flexural reinforcement perpendicular to the edge is not considered fully effective unless it is placed within the effective slab width as shown in Figure 6.8.23.

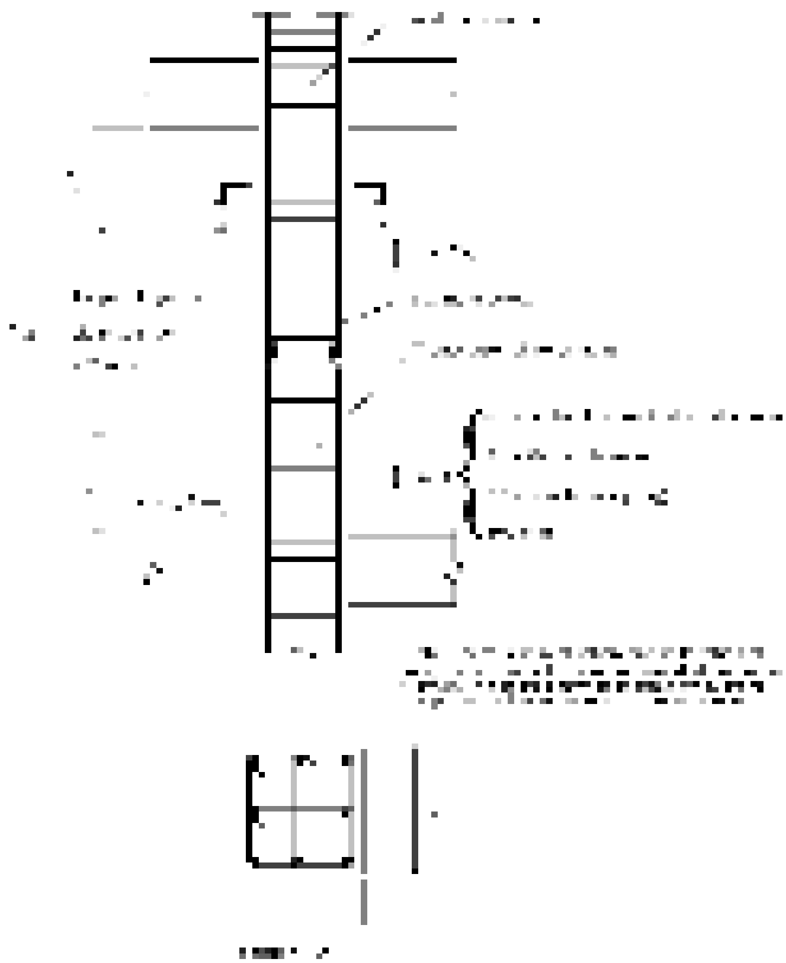


Figure 6.8.19 Transverse reinforcement requirements for columns (IMF)

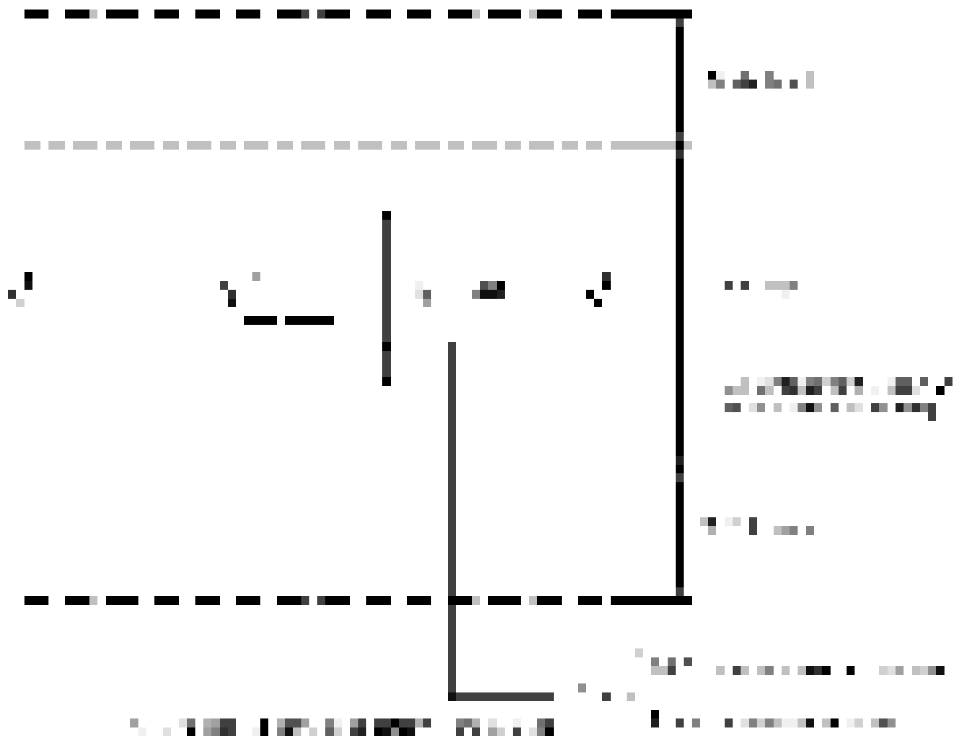


Figure 6.8.20 Reinforcement details at support of two-way slabs without beams

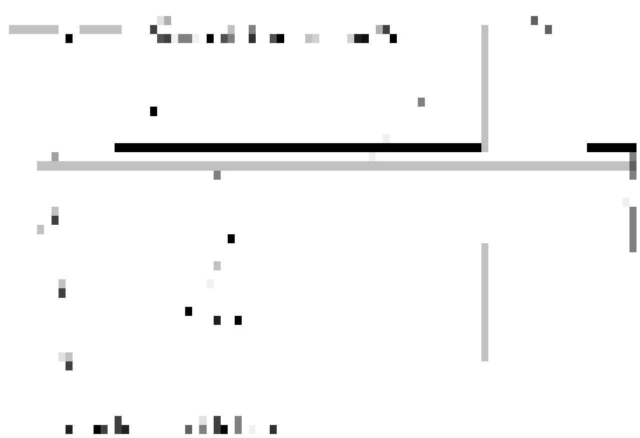


Figure 6.8.21 Reinforcement Details in Two-way Slabs without beams: Column Strip

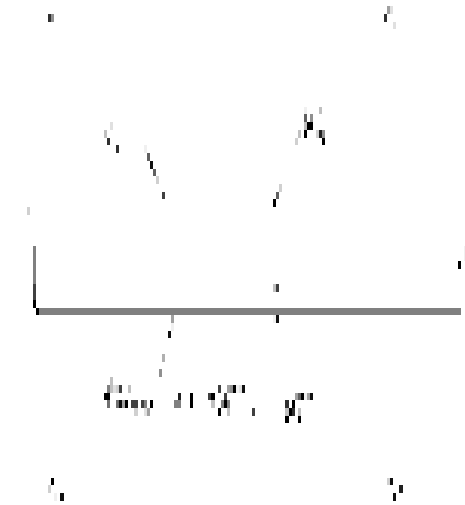
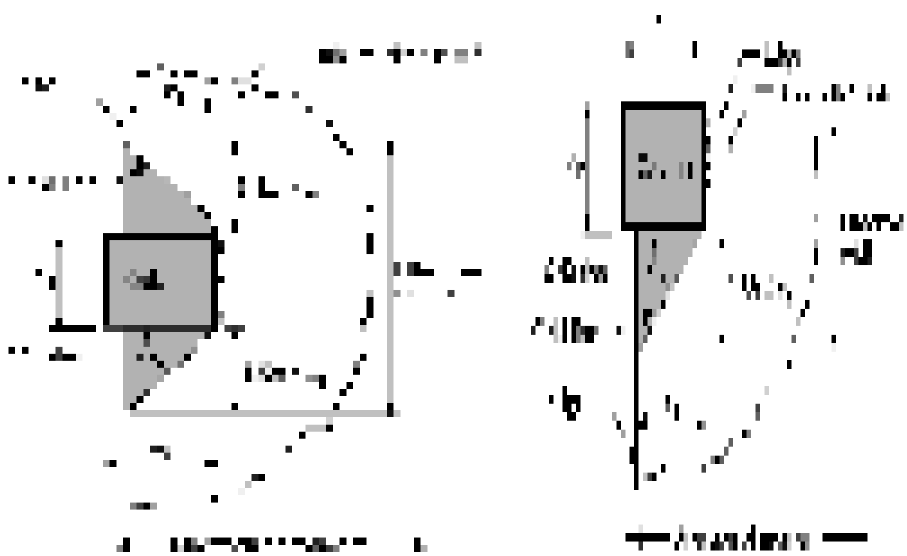


Figure 6.8.22 Reinforcement details in two-way slabs without beams: middle strip



(a) Edge connection (b) Corner connection

Figure 6.8.23 Effective width for reinforcement placement in edge and corner connections.

8.3.11 Requirements for Foundation

8.3.11.1 Scope

Foundations resisting earthquake induced forces or transferring earthquake-induced forces between structure and ground in structures assigned to SDC D shall comply with Sec 8.3.11 and other applicable Code provisions.

The provisions in this section for piles, drilled piers, caissons, and slabs-on-ground shall supplement other applicable Code design and construction criteria.

8.3.11.2 Footings, foundation mats, and pile caps

(a) Longitudinal reinforcement of columns and structural walls resisting forces induced by earthquake effects shall extend into the footing, mat, or pile cap, and shall be fully developed for tension at the interface.

(b) Columns designed assuming fixed-end conditions at the foundation shall comply with Sec 8.3.11.2(a) and, if hooks are required, longitudinal reinforcement resisting flexure shall have 90^{o} hooks near the bottom of the foundation with the free end of the bars oriented toward the centre of the column.

(c) Columns or boundary elements of special structural walls that have an edge within one-half the footing depth from an edge of the footing shall have transverse reinforcement in accordance with Sec 8.3.5.4 provided below the top of the footing. This reinforcement shall extend into the footing, mat, or pile cap and be developed for f_y in tension.

(d) Where earthquake effects create uplift forces in boundary elements of special structural walls or columns, flexural reinforcement shall be provided in the top of the footing, mat, or pile cap to resist actions resulting from the design load combinations, and shall not be less than required by Sec 6.3.5.

8.3.11.3 Grade beams and slabs-on-ground

(a) Grade beams designed to act as horizontal ties between pile caps or footings shall have continuous longitudinal reinforcement that shall be developed within or beyond the supported column or anchored within the pile cap or footing at all discontinuities.

(b) Grade beams designed to act as horizontal ties between pile caps or footings shall be proportioned such that the smallest cross-sectional dimension shall be equal to or greater than the clear spacing between connected columns divided by 20, but need not be greater than 450 mm. Closed ties shall be provided at a spacing not to exceed the lesser of one-half the smallest orthogonal cross-sectional dimension and 300 mm.

(c) Grade beams and beams that are part of a mat foundation subjected to flexure from columns that are part of the seismic-force-resisting system shall conform to Sec 8.3.4.

(d) Slabs-on-ground that resist seismic forces from walls or columns that are part of the seismic-force-resisting system shall be designed as structural diaphragms in accordance with Sec 8.3.6. The design drawings shall clearly state that the slab on ground is a structural diaphragm and part of the seismic-force-resisting system.

8.3.11.4 Piles, piers, and caissons

(a) Provisions of Sec 8.3.11.4 shall apply to concrete piles, piers, and caissons supporting structures designed for earthquake resistance.

(b) Piles, piers, or caissons resisting tension loads shall have continuous longitudinal reinforcement over the length resisting design tension forces. The longitudinal reinforcement shall be detailed to transfer tension forces within the pile cap to supported structural members as shown in Figure 6.8.24.

(c) Where tension forces induced by earthquake effects are transferred between pile cap or mat foundation and precast pile by reinforcing bars grouted or post-installed in the top of the pile, the grouting system shall have been demonstrated by test to develop at least $1.25 f_y$ of the bar.

(d) Piles, piers, or caissons shall have transverse reinforcement, Figure 6.8.24, in accordance with Sec 8.3.5.4 at locations—

(i) Top of the member for at least 5 times the member cross-sectional dimension, but not less than 1.8 m below the bottom of the pile cap;

(ii) Portion of piles in soil that is not capable of providing lateral support, or in air and water, along the entire unsupported length plus the length required in (i).

(e) For precast concrete driven piles, the length of transverse reinforcement provided shall be sufficient to account for potential variations in the elevation in pile tips.

(f) Concrete piles, piers, or caissons in foundations supporting one- and two-story stud bearing wall construction are exempt from the transverse reinforcement requirements of Sec 8.3.11.4(d) and (e).

(g) Pile caps incorporating batter piles shall be designed to resist the full compressive strength of the batter piles acting as short columns. The slenderness effects of batter piles shall be considered for the portion of the piles in soil that is not capable of providing lateral support, or in air or water.

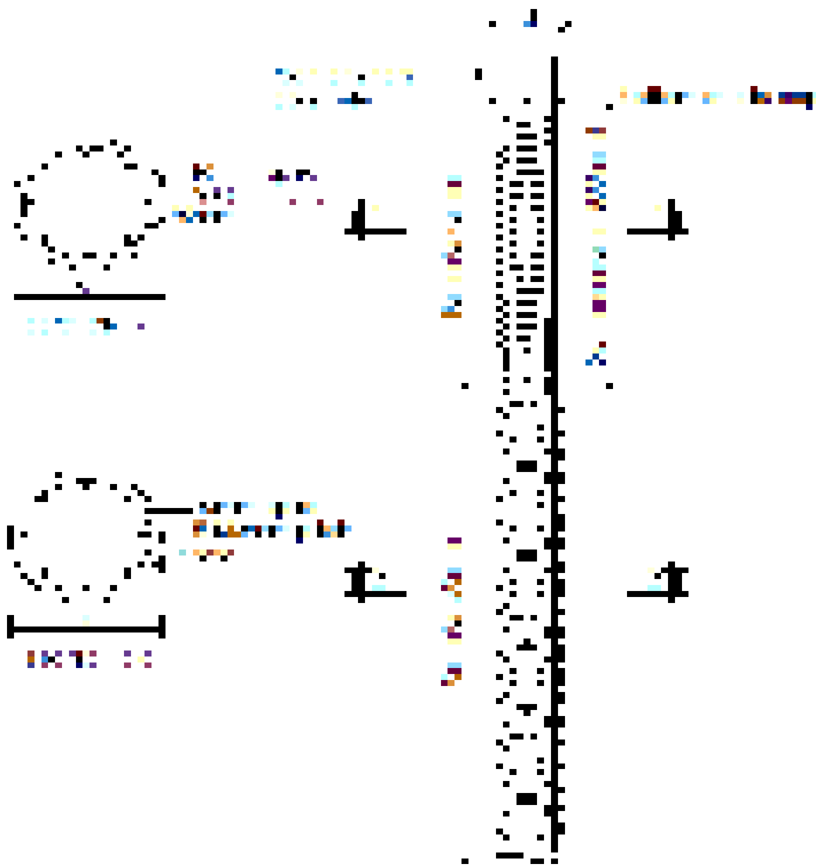


Figure 6.8.24 Spiral details of cast-in-situ pile in seismic zone 4 and SDC D

8.3.12 Requirement Members not Designated as Part of the Seismic-Force-Resisting System

8.3.12.1 Scope

(a) Requirements of Sec 8.3.12 apply to frame members not designated as part of the seismic-force-resisting system in structures assigned to SDC D.

Frame members assumed not to contribute to lateral resistance, except two-way slabs without beams, shall be detailed according to Sec 8.3.12.2 or Sec 8.3.12.3 depending on the magnitude of moments induced in those members when subjected to the design displacement δ_u. If effects of δ_u are not explicitly checked, it shall be permitted to apply the requirements of Sec 8.3.12.3. For two-way slabs without beams, slab-column connections shall meet the requirements of Sec 8.3.12.5.

8.3.12.2 Induced moment and shear do not exceed design capacities

Where the induced moments and shears under design displacements, δ_u, combined with the factored gravity moments and shears do not exceed the design moment and shear strength of the frame member, the conditions of Sections 8.3.12.2(a), 8.3.12.2(b), and 8.3.12.2(c) shall be satisfied. The gravity load combinations of $(1.2D + 1.0L + 0.2S)$ or $0.9D$, whichever is critical, shall be used. The load factor on the live load, L, shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where L is greater than 4.8 kN/m^2.

(a) Members with factored gravity axial forces not exceeding $\frac{A_g f_c'}{10}$ shall satisfy Sec 8.3.4.2(a).Stirrups shall be spaced not more than $\frac{d}{2}$ throughout the length of the member.

(b) Members with factored gravity axial forces exceeding $\frac{A_g f_c'}{10}$ shall satisfy Sections 8.3.5.3(a) and 8.3.5.4. The maximum longitudinal spacing of ties shall be s_o for the full member length. Spacing s_o shall not exceed the smaller of six diameters of the smallest longitudinal bar enclosed and 150 mm.

(c) Members with factored gravity axial forces exceeding $0.35P_o$ shall satisfy Sec 8.3.12.2(b). The amount of transverse reinforcement provided shall be one-half of that required by Sec 8.3.5.4(a) but shall not be spaced greater than s_o for the full member length.

8.3.12.3 Induced moment or shear exceeds design capacities

If the induced moment or shear under design displacements, δ_u exceeds ϕM_n or ϕV_n of the frame member, or if induced moments are not calculated, the conditions of Sec 8.3.12.3(a), (b) and (c) shall be satisfied.

(a) Materials shall satisfy 8.3.3.3 and 8.3.3.4. Welded splices shall satisfy 8.3.3.5.

(b) Members with factored gravity axial forces not exceeding $\frac{A_g f_c'}{10}$ shall satisfy Sections 8.3.4.2and 8.3.8. Stirrups shall be spaced at not more than $\frac{d}{2}$ throughout the length of the member.

(c) Members with factored gravity axial forces exceeding $\frac{A_g f_c'}{10}$ shall satisfy Sections 8.3.5.3, 8.3.5.4, 8.3.7.1 and 8.3.8.

8.3.12.4 Two-way slabs without beams

For slab-column connections of two-way slabs without beams, slab shear reinforcement satisfying the requirements of Sections 6.4.10.3 and 6.4.10.5 and providing V_s not less than $0.29\sqrt{f_c'}\, b_o d$ shall extend at least four times the slab thickness from the face of the support, unless either (i) or (ii) is satisfied :

(i) The requirements of Sec 6.4.10.7 using the design shear V_{ug} and the induced moment transferred between the slab and column under the design displacement;

(ii) The design story drift ratio does not exceed the larger of 0.005 and $\left[0.035 - 0.05\left(\frac{V_{ug}}{\phi V_c}\right)\right]$.

Design story drift ratio shall be taken as the larger of the design story drift ratios of the adjacent stories above and below the slab-column connection. V_c is defined in Sec 6.4.10.2. V_{ug} is the factored shear force on the slab critical section for two-way action, calculated for the load combination $1.2D + 1.0L + 0.2S$.

The load factor on the live load, L, shall be permitted to be reduced to 0.5 except for garages, areas occupied as places of public assembly, and all areas where L is greater than 4.8 kN/m^2.

PART VI
Chapter 9
Prestressed Concrete Structures

9.1 General

The Prestressed Concrete Structures Chapter of the Code is divided into the following three Divisions:

Division A : Design

Division B : Material and Construction

Division C : Maintenance

Division A : Scope, Definitions, Notation, Design And Analysis (Sections 9.2 To 9.18)

9.2 Scope

9.2.1 Provisions of this Chapter shall apply to members prestressed with wires, strands, or bars conforming to the specifications of prestressing tendons given in Sec 9.5.1.3.

9.2.2 All provisions of this Code not specifically excluded, and not in conflict with provisions of this Chapter 9, shall apply to prestressed concrete.

9.3 Definitions, Symbols and Notation

9.3.1 Definitions

ACTION	Mechanical force or environmental effect to which the structure (or structural component) is subjected.
ANALYSIS	Acceptable methods of evaluating the performance indices or verifying the compliance of specific criteria.
ANCHORAGE	In post-tensioning, a mechanical device used to anchor the tendon to the concrete; in pretensioning, a device used to anchor the tendon until the concrete has reached a pre-determined strength, and the prestressing force has been transferred to the concrete; for reinforcing bars, a length of reinforcement, or a mechanical anchor or hook, or combination thereof at the end of a bar needed to transfer the force carried by the bar into the concrete.

ANCHORAGE BLISTER	A build-up area on the web, flange, or flange-web junction for the incorporation of tendon anchorage fittings.
ANCHORAGE ZONE	The portion of the structure in which the prestressing force is transferred from the anchorage device on to the local zone of the concrete, and then distributed more widely in the general zone of the structure.
AT JACKING	At the time of tensioning the prestressing tendons.
AT LOADING	The maturity of the concrete when loads are applied. Such loads include prestressing forces and permanent loads but generally not live loads.
AT TRANSFER	Immediately after the transfer of prestressing force to the concrete.
AUTOGENEOUS SHRINKAGE	Volume decrease due to loss of water in the hydration process causing negative pore pressure in concrete.
BIOLOGICAL DEGRADATION	The physical or chemical degradation of concrete due to the effect of organic matters such as bacteria, lichens, fungi, moss, etc.
BONDED MEMBER	A prestressed concrete member in which tendons are bonded to the concrete either directly or through grouting.
BONDED POST-TENSIONING	Post-tensioned construction in which the annular space around the tendons is grouted after stressing, thereby bonding the tendon to the concrete section.
BONDED TENDON	Prestressing tendon that is bonded to concrete either directly or through grouting.
BURSTING FORCE	Tensile forces in the concrete in the vicinity of the transfer or anchorage of prestressing forces.
CAST-IN-PLACE CONCRETE	Concrete placed in its final position in the structure while still in a plastic state.
CHARACTERISTIC STRENGTH	Unless otherwise stated in this Code, the characteristic strength of material refers to the value of the strength below which none of the test results should fall below by more than 15% or 3.5 MPa for ≤35 MPa concrete, and 10% or 3.5 MPa for ≥35 MPa concrete, whichever is larger.

CHEMICAL ADMIXTUREs	Admixtures which are usually used in small quantities typically in the form of liquid and can be added to the concrete both at the time of mixing and before placing to improve various concrete properties such as workability, air content and durability, etc.
CLOSELY SPACED ANCHORAGES	Anchorage devices are defined as closely spaced if their centre to centre spacing does not exceed 1.5 times the width of the anchorage devices in the direction considered.
CLOSURE	A placement of cast-in-place concrete used to connect two or more previously cast portions of a structure.
COMPOSITE CONSTRUCTION	Concrete components or concrete and steel components interconnected to respond to force effects as a unit.
COMPRESSION-CONTROLLED SECTION	A cross-section in which the net tensile strain in the extreme tension steel at nominal resistance is less than or equal to the compression-controlled strain limit.
COMPRESSION-CONTROLLED STRAIN LIMIT	The net tensile strain in the extreme tension steel at balanced strain conditions.
CONCRETE COVER	The specified minimum distance between the surface of the reinforcing bars, strands, post-tensioning ducts, anchorages, or other embedded items, and the surface of the concrete.
CONFINEMENT	A condition where the disintegration of the concrete under compression is prevented by the development of lateral and/or circumferential forces such as may be provided by appropriate reinforcing steel or composite tubes, or similar devices.
CONFINEMENT ANCHORAGE	Anchorage for a post-tensioning tendon that functions on the basis of containment of the concrete in the anchorage zone by special reinforcement.
CREEP	Time dependent deformation of concrete under permanent load.
CREEP COEFFICIENT	The ratio of creep strain to elastic strain in concrete.
CREEP IN CONCRETE	Increase in strain with time in concrete subjected to sustained stress.

CURVATURE FRICTION	Friction resulting from bends or curves in the specified prestressing tend stage at which the compressive stresses on profile.
DAMAGE CONTROL	A means to ensure that the limit state requirement is met for restorability or reparability of a structure.
DECOMPRESSION	The stage at which the compressive stresses, induced prestress, are overcome by the tensile stresses.
DEFORMABILITY	A term expressing the ability of concrete to deform.
DEGREE OF DETERIORATION	The extent to which the performance of a structure is degraded or the extent to which the deterioration has progressed from the time of construction, as a result of its exposure to the environment.
DESIGN LIFE	Assumed period for which the structure is to be used satisfactorily for its intended purpose or function with anticipated maintenance but without substantial repair being necessary.
DETERIORATION INDEX	An index selected for estimating and evaluating the extent of the deterioration process.
DETERIORATION PREDICTION	Prediction of the future rate of deterioration of a structure based on results of inspection and relevant records made during the design and construction stages.
DEVIATION SADDLE	A concrete block build-out in web, flange, or web-flange junction used to control the geometry of or to provide a means for changing direction of, external tendons.
DRYING SHRINKAGE	Volume decrease due to loss of moisture from concrete in the hardened state which is usually serious in hot and dry environment.
DURABILITY DESIGN	Design to ensure that the structure can maintain its required functions during service life under environmental actions.
DURABILITY GRADE	The extent of durability to which the structure shall be maintained in order to satisfy the required performance during its design life. This affects the degree and frequency of the remedial actions to be carried out during that life.

DYNAMIC APPROACH	An approach based on dynamic analysis to assess the overall forces on a structure liable to have a resonant response to wind action.
DYNAMIC RESPONSE FACTOR	Factor to account for the effects of correlation and resonant response.
EARLY AGE STATE	The state of concrete from final setting until the achievement of the required characteristic strength.
EFFECTIVE PRESTRESS	Stress remaining in prestressing tendons after all losses have occurred, excluding effects of dead load and superimposed load.
ENVIRONMENTAL ACTIONS	An assembly of physical, chemical or biological influences which may cause deterioration to the materials making up the structure, which in turn may adversely affect its serviceability, restorability and safety.
FATIGUE LOADS	Repetitive loads causing fatigue in the material which reduces its strength, stiffness and deformability.
FINAL PRESTRESS	Stress which exists after substantially all losses have occurred.
FINAL TENSION	The tension in the steel corresponding to the state of the final prestress.
FORMWORK	Total system of support for freshly placed concrete including the mould or sheathing, all supporting members, hardware and necessary bracings.
FUNCTION	The task which a structure is required to perform.
GENERAL ZONE	Region adjacent to a post-tensioned anchorage within which the prestressing force spreads out to an essentially linear stress distribution over the cross section of the component.
GROUT	A mixture of cementitious material and water with or without admixtures.
INITIAL PRESTRESS	The prestress in the concrete at transfer.
INITIAL TENSION	The maximum stress induced in the prestressing tendon at the time of stressing operation.

JACKING FORCE	Temporary force exerted by device that introduces tension into prestressing tendons.
LIMIT STATE	A critical state specified using a performance index, beyond which the structure no longer satisfies the design performance requirements.
LIMITS OF DISPLACEMENT	Allowable deformation of structure in terms of such parameters as inter-storey drift and relative horizontal displacement, to control excessive deflection, cracking and vibration.
LONG-TERM PERFORMANCE INDEX	Index defining the remaining capacity of a structure in performing its design functions during the design life.
LOCAL ZONE	The volume of concrete that surrounds and is immediately ahead of the anchorage device and that is subjected to high compressive stresses.
MAINTENANCE	A set of activities taken to ensure that the structure continues to perform its functions satisfactorily during the design life.
MECHANICAL FORCES	An assembly of concentrated or distributed forces acting on a structure, or deformations imposed on it.
MODEL	Mathematical description or experimental setup simulating the actions, material properties and behavior of a structure.
MONITORING	Continuous recording of data pertaining to deterioration and/or performance of structure using appropriate equipment.
NOMINAL STRENGTH OF MATERIAL	The characteristic values of the strength of materials used for calculation, in absence of the available statistical data.
NORMAL CONCRETE	Concrete which is commonly used in construction; it does not include special constituent materials other than Portland cement, water, fine aggregate, coarse aggregate and common mineral and chemical admixtures; it does not require any special practice for its manufacturing and handling.

OVERALL PERFORMANCE INDEX	Index indicating the overall performance of the structure.
PARTIAL PERFORMANCE INDEX	Index indicating a partial performance of the structure.
PARTIAL SAFETY FACTOR FOR MATERIAL	For analysis purposes, the design strength of a material is determined as the characteristic strength divided by a partial safety factor.
PERFORMANCE	Ability (or efficiency) of a structure to perform its design functions.
PERFORMANCE INDEX	Index indicating structural performance quantitatively.
PERMANENT ACTIONS	Self-weights of structures inclusive of permanent attachments, fixtures and fittings.
PLASTIC SHRINKAGE	Shrinkage arising from loss of water from the exposed surface of concrete during the plastic state, leading to cracking at the exposed surface.
PLASTIC STATE	The state of concrete from just after placing until the final setting of concrete.
POST-TENSIONING	Method of prestressing in which tendons are tensioned after concrete has hardened.
PRESTRESSED CONCRETE	Reinforced concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.
PRETENSIONING	Method of prestressing in which tendons are tensioned before concrete is placed.
SHRINKAGE LOSS	The loss of stress in the prestressing steel resulting from the shrinkage of concrete.
RELIABILITY	Ability of a structure to fulfill specified requirements during its design life.
REMAINING SERVICE LIFE	Period from the point of inspection to the time when the structure is no longer useable, or does not satisfactorily perform the functions determined at the time of design.

REMEDIAL ACTION	Maintenance action carried out with the objective of arresting or slowing down the deterioration process, restoring or improving the performance of a structure, or reducing the danger of damage or injury to the users or any third party.
REPAIR	Remedial action taken with the objective of arresting or slowing down the deterioration of a structure, or reducing the possibility of damage to the users or third party.
RESTORABILITY	Ability of a structure to be repaired physically and economically when damaged under the effects of considered actions. Also known as REPAIRABILITY.
ROBUSTNESS	Ability of a structure to withstand damage by events like fire, explosion, impact, instability or consequences of human errors. Also known as STRUCTURAL INSENSITIVITY.
SAFETY	Ability of a structure to ensure that no harm would come to the users and to people in the vicinity of the structure under any action.
SERVICE LIFE	The length of time from the completion of a structure until the time when it is no longer usable because of its failure to adequately perform its design functions.
SERVICEABILITY	Ability of a structure to provide adequate services or functionality in use under the effects of considered actions.
SETTLEMENT OF CONCRETE	Sinking of the concrete surface after placing due to bleeding and/or escaping of the entrapped and entrained air in the concrete.
SPECIAL CONCRETE	Concrete other than normal concrete including light weight concrete, roller compacted concrete, self-compacting concrete, fiber-reinforced concrete, anti-washout under water concrete, etc.
STIFF AND FLEXIBLE STRUCURES	Stiff structures refer to those that are not sensitive to dynamic effects of wind, while flexible ones are those that are sensitive to such effects.
STRENGTHENING	Remedial action applied to a structure with the objective of restoring or improving its load bearing capacity to a level which is equal to, or higher than, the original design level.

STRESS AT TRANSFER	The stress in both the prestressing tendon and the concrete at the stage when the prestressing tendon is released from the prestressing mechanism.
TEMPERATURE CRACKING	Cracking caused by thermal stress which arises from differential temperatures in the concrete mass.
TENDON	Steel element such as wire, cable, bar, rod, or strand, or a bundle of such elements, used to impart prestress to concrete.
THRESHOLD LEVEL OF PERFORMANCE	Minimum acceptable level of performance of a structure.
TRANSFER	Act of transferring stress in prestressing tendons from jacks or pretensioning bed to concrete member.
TRANSFER LENGTH	The distance required at the end of a pretensioned tendon for developing the maximum tendon stress by bond.
ULTIMATE LIMIT STATE	Limit state for safety.
VARIABLE ACTION	Action due to a moving object on the structure as well as any load whose intensity is variable, including traffic load, wave load, water pressure, and load induced by temperature variation.
WOBBLE FRICTION	Friction caused by unintended deviation of prestressing sheath or duct from its specified profile.
WORKABILITY	The term expressing the ease with which concrete can be placed, compacted and filled.

9.3.2 Notation and Symbols

A = Area of the part of cross-section between flexural tension face and centre of gravity of gross section, mm^2

A_{ch} = Cross-sectional area of a structural member measured to the outside edges of transverse reinforcement, mm^2

A_g = Gross area of concrete section, mm^2. For a hollow section, A_g is the area of the concrete only and does not include the area of the void(s)

A_{ps} = Area of prestressed reinforcement in tension zone, mm^2

A_s = Area of nonprestressed tension reinforcement, mm^2

A'_s = Area of compression reinforcement, mm^2

C_c = Clear cover of reinforcement, mm

D = Dead loads, or related internal moments and forces

I = Moment of inertia of cross-section resisting externally applied factored loads, mm^4

I_{cr} = Moment of inertia of cracked section transformed to concrete, mm^4, Sec 6.

I_g = Moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, mm^4

I_e = Effective moment of inertia for computation of deflection, mm^4

K = Wobble friction coefficient per meter of prestressing tendon

L = Live loads, or related internal moments and forces

M_a = Maximum moment in member due to service loads at stage deflection is computed, N-mm

M_{cr} = Moment causing flexural cracking at section due to externally applied loads, N-mm

M_{max} = Maximum factored moment at section due to externally applied loads, N-mm

M_u = Factored moment at section, N-mm

N_c = Tensile force in concrete due to unfactored dead load plus live load (D + L), N

P_j = Prestressing tendon force at jacking end, N

PI_P = Inherent or possessed performance index

PI_R = Inherent or possessed performance index

P_x = Prestressing tendon force at any point x

V_c = Nominal shear strength provided by concrete, N

V_{ci} = Nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment, N

V_{cw} = Nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web, N

V_d = Shear force at section due to unfactored dead load, N

V_i = Factored shear force at section due to externally applied loads occurring simultaneously with M_{max}, N

V_n = Nominal shear strength, N

V_p = Vertical component of effective prestress force at section, N

V_s = Nominal shear strength provided by shear reinforcement, N

V_u = Factored shear force at section, N

X = Shorter overall dimension of rectangular part of cross-section

a = Depth of equivalent rectangular stress block, mm

b = Width of compression face of member, mm

d = Distance from extreme compression fiber to centroid of nonprestressed tension reinforcement, mm

d' = Distance from extreme compression fiber to centroid of compression reinforcement, mm

d_b = Nominal diameter of bar, wire, or prestressing strand, mm

d_p = Distance from extreme compression fiber to centroid of prestressed reinforcement, mm

e = Base of Napierian logarithm

f_c' = Specified compressive strength of concrete, N/mm^2

f_{ci}' = Compressive strength of concrete at transfer of prestress, N/mm^2

f_d = Stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, N/mm^2

f_{pe} = Compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads, N/mm^2

f_{pc} = Average compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses), N/mm^2

f_{ps} = Stress in prestressed reinforcement at nominal strength, N/mm^2

f_{pu} = Specified tensile strength of prestressing tendons, N/mm^2

f_{py} = Specified yield strength of prestressing tendons, N/mm^2

f_r = Modulus of rupture of concrete, N/mm^2

f_{se} = Effective stress in prestressed reinforcement (after allowance for all prestress losses), N/mm^2

f_t = Extreme fiber stress in tension in the pre-compressed tensile zone calculated at service loads using gross section properties, N/mm^2 (MPa)

f_y = Specified yield strength of nonprestressed reinforcement, N/mm^2

f_{yt} = Specified yield strength f_y of transverse reinforcement, N/mm^2

h = Overall thickness of member, mm

h_f = Overall thickness of flange of flanged section, mm

l = Length of span of two-way flat plates in direction parallel to that of the reinforcement being determined, mm

l_x = Length of prestressing tendon element from jacking end to any point x, metre

s = Spacing of shear or torsion reinforcement in direction parallel to longitudinal reinforcement, mm

y = Longer overall dimension of rectangular part of cross-section

y_t = Distance from centroidal axis of gross section, neglecting reinforcement, to extreme fibre in tension

α = Total angular change of prestressing tendon profile in radians from tendon jacking end to a point x

β_1 = Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth

γ_p = A factor for type of prestressing steel

μ = Curvature friction coefficient

λ = Modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normal weight concrete of the same compressive strength (i.e., $\lambda = 1.0$ for normal weight concrete and 0.75 for all lightweight concrete. Else, λ shall be determined based on volumetric proportions of lightweight and normal weight aggregates, but shall not exceed 0.85.)

ρ	=	Ratio of nonprestressed tension reinforcement = $A_s/(bd)$
ρ'	=	Ratio of compression reinforcement = $A'_s/(bd)$
ρ_p	=	Ratio of prestressed reinforcement = $A_{ps}/(bd_p)$
ϕ	=	Strength reduction factor
ω	=	$\rho f_y/f'_c$
ω'	=	$\rho' f_y/f'_c$
ω_p	=	$\rho_p f_{ps}/f'_c$
$\omega_p, a_{p\omega}$ ω_w	=	Reinforcement indices for flanged sections computed for ω, ω_p and ω' except that b shall be the web width, and reinforcement area shall be that required to develop compressive strength of web only.

For other symbols and units of quantities, reference may be made to Chapter 6.

9.4 Analysis and Design

9.4.1 General

9.4.1.1 Prestressed members shall be designed for adequate strength in accordance with the provisions of this Chapter.

9.4.1.2 Unless specifically excluded or superseded by the provisions of this Chapter, all other relevant provisions of this Code shall apply to prestressed concrete.

9.4.1.3 Design of prestressed members shall be based on strength and on the behavior at service conditions at all stages that will be critical during the life of the structure from the time prestress is first applied.

9.4.1.4 Stress concentrations due to prestressing shall be considered in design.

9.4.1.5 Provisions shall be made for effects on adjoining construction of elastic and plastic deformations, deflections, changes in length and rotations due to prestressing. Effects of creep, temperature and shrinkage shall also be considered.

9.4.1.6 The possibility of buckling in a member between points where there is intermittent contact between prestressing steel and an oversized duct and buckling in thin webs and flanges shall be considered.

9.4.1.7 In computing section properties before bonding of prestressing steel, effect of loss of area due to open ducts shall be considered.

9.4.1.8 Thermal gradient and differential shrinkage shall be considered in composite construction using prestressed concrete members.

9.4.1.9 In evaluating the slenderness effects during lifting of slender beams, consideration shall be given to beam geometry, location of lifting points, method of lifting and tolerances in construction. All beams which are lifted on vertical or inclined slings shall be checked for lateral stability and lateral moment on account of tilting of beam. Reference may be made to specialist literature in this regard.

9.4.2 Design Assumptions

9.4.2.1 Strength design of prestressed members for flexure and axial loads shall be based on assumptions given in Sections 9.4.2.2 to 9.4.2.7 and shall satisfy the applicable conditions of equilibrium and compatibility of strains.

9.4.2.2 Strains in steel and concrete shall be assumed to be directly proportional to the distance from the neutral axis except for Deep Beams.

9.4.2.3 If nonprestressed reinforcement conforming to Sec 5.3.2 is used then, stress in such reinforcements below f_y, shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y, stress in reinforcement shall be considered independent of strain and equal to f_y.

9.4.2.4 Maximum usable strain at extreme concrete compression fiber shall be assumed equal to 0.003.

9.4.2.5 The relationship between concrete compressive stress distribution and concrete strain shall be assumed to be rectangular, trapezoidal, parabolic, or any other shape that results in prediction of strength in substantial agreement with results of comprehensive tests

9.4.2.6 Requirements of Sec 9.4.2.5 are satisfied by an equivalent rectangular concrete stress distribution defined by the following:

(a) Concrete stress of $0.85f_c'$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a=\beta_1 c$ from the fiber of maximum compressive strain.

(b) Distance from the fiber of maximum strain to the neutral axis, c is measured in a direction perpendicular to the neutral axis.

(c) For f_c' between 17.5 and 28 MPa, β_1 shall be taken as 0.85. For f_c' above 28 MPa, β_1 shall be reduced linearly at a rate of 0.05 for each 7 MPa of strength in excess of 28 MPa, but β_1 shall not be taken less than 0.65.

9.4.2.7 For investigation of stresses at transfer of prestress, at service loads, and at cracking loads, elastic theory shall be used with the following assumptions:

(i) Strains vary linearly with depth through the entire load range.

(ii) At cracked sections, concrete resists no tension.

9.4.3 Classification of Prestressed Concrete Members

Prestressed concrete flexural members shall be classified as Class U (uncracked), Class T (transition) and Class C (cracked) based on f_t, the computed extreme fiber stress in tension in the pre-compressed tensile zone calculated at service load as follows:

(a) Class U: $f_t \leq 0.62\sqrt{f'_c}$

(b) Class T: $0.62\sqrt{f'_c} \leq f_t \leq 1.0\sqrt{f'_c}$

(c) Class C: $f_t > 1.0\sqrt{f'_c}$

Prestressed two-way slab systems shall be designed as class U with $f_t \leq 0.50\sqrt{f'_c}$

9.4.4 Shapes of Beams and Girders

For prestressed concrete non-composite beams/girders, the frequently used shapes are:

(a) Symmetrical I-section,

(b) Unsymmetrical I-section,

(c) T-section,

(d) Inverted T-section,

(e) Box section and

(f) Solid/hollow rectangular section.

Commentary:

The suitability of selecting a particular shape will depend on the specific design requirement and economy of construction. In general, T or equal or unequal I-section are common choices to achieve economy in steel and concrete. Due consideration to the simplicity of formwork is also required.

9.4.5 Material Properties for Design

9.4.5.1 Concrete Preparation and Design: Concrete shall be prepared, conveyed, placed/cast, cured, tested and maintained following appropriate sections of Chapter 5 of the Code. Relevant applicable standards are mentioned in Chapter 5 and also listed in Table 6.9.9. Unless specifically applicable to prestressed concrete, general design requirements of normal concrete are those of Chapter 6 of the Code.

9.4.5.2 Class: The Class of concrete is defined by the specified strength of concrete cylinder f_c' at 28 days. For example, Class 20 indicates concrete cylinder crushing strength of $f_c' = 20$ N/mm^2. Commonly, the classes of concrete shall be in steps of 5 N/mm^2 as given by: Class 20, 25, 30 35… … … … 65 and 70 etc. although concrete in between these classes may also be permitted (like class 21, 24, 28, 31, 38, 42, and 49 etc.).

9.4.5.3 Modulus of Elasticity, E_c: Modulus of elasticity, E_c for concrete shall be permitted to be taken as $w_c^{1.5} 0.043\sqrt{f'_c}$ (in N/mm^2) for values of w_c between 1440 and 2560 kg/m^3. For normal weight concrete, E_c may be permitted to be taken as $4700\sqrt{f_c'}$.

9.4.5.4 Modulus of Rupture, f_r: Modulus of rupture, f_r for concrete shall be permitted to be taken as $0.62\,\lambda\sqrt{f_c'}$ where $\lambda = 1$ for normal weight concrete and 0.75 for all lightweight concrete.

9.4.5.5 Reinforcing steel: Appropriate applicable standards for reinforcing steel are given in Chapter 5 and also listed in Table 6.9.10. Unless specifically applicable to prestressed concrete, general design requirements of reinforcing steel are those that has been laid down in Chapter 6 of the Code.

9.4.5.6 Modulus of elasticity, E_s: Where it is not possible to ascertain the modulus of elasticity of reinforcing steel by test and from the manufacturer of steel, the modulus of elasticity of reinforcing steel may be permitted to be taken as $E_s = 200{,}000$ N/mm^2.

9.4.5.7 Prestressing Steel: Appropriate applicable standards for prestressing steel are listed in Table 6.9.11.

9.4.5.8 Modulus of elasticity, E_s : Where it is not possible to ascertain the modulus of elasticity of pain/ indented steel wire and prestressing steel (bar or strand) by test and from the manufacturer of steel, the values of E_s given in Table 6.9.1 may be used:

Table 6.9.1: Modulus of Elasticity of Prestressing Steel and Cold Drawn Wire

Type of steel	Modulus of elasticity, E_s (kN/mm^2)
Plain/indented cold-drawn wire	200
High tensile steel bars rolled or heat-treated	205
Strands	195

9.5 Serviceability Requirements – Flexural Members

9.5.1 Stress in Concrete At Transfer

Stresses in concrete immediately after prestress transfer (before time-dependent prestress losses occur) are as follows:

(a) Extreme fiber stress in compression except as permitted in (b) shall not exceed $0.60f'_{ci}$

(b) Extreme fiber stress in compression at ends of simply support members shall not exceed $0.70f'_{ci}$

(c) Where computed concrete tensile strength, f_t exceeds$0.5\sqrt{f'_c}$ at ends of simply supported members, or $0.25\sqrt{f'_c}$ at other locations, additional bonded reinforcement shall be provided in the tensile zone to resist the total tensile force in concrete computed with the assumption of an uncracked section.

<u>Allowable stresses in concrete</u>

For Class U and Class T prestressed flexural members, stresses in concrete at service loads (based on uncracked section properties and after allowance for all prestress losses) shall not exceed the following:

(a)	Extreme fiber stress in compression due to prestress plus sustained load	$0.45f'_c$
(b)	Extreme fiber stress in compression due to prestress plus total load	$0.60f'_c$

9.5.2 Permissible stresses in Sections 9.5.1 and 9.5.2 shall be permitted to be exceeded if shown by test or analysis that performance will not be impaired.

9.5.3 Reinforcement Spacing

9.5.3.1 For Class C prestressed flexural members not subject to fatigue or to aggressive exposure, the spacing **s** of bonded reinforcement nearest the extreme tension face shall not exceed that for normal Reinforced Concrete, as given below:

$$s = 380(280/f_s) - 2.5C_c \qquad (6.9.1)$$

But, not greater than $300(280/f_s)$, where c_c is the least distance from the surface of reinforcement or prestressing steel to the tension face. If there is only one bar or wire nearest to the extreme tension face, s used in the above equation is the width of the extreme tension face.

Calculated stress f_s in reinforcement closest to the tension face at service loads shall be computed based on the unfactored moment. It shall be permitted to take f_s as $\frac{2}{3} f_y$.

For structures subject to fatigue or exposed to corrosive environments, investigations, judgment and precautions are required.

9.5.3.2 The spacing requirements Sec 9.5.3.1 shall be met by nonprestressed reinforcement and bonded tendons.

(a) The spacing of bonded tendons shall not exceed 2/3rd of the maximum spacing permitted for nonprestressed reinforcement.

Where both reinforcement and bonded tendons are used to meet the spacing requirement, the spacing between a bar and a tendon shall not exceed 5/6th of that permitted by 9.5.3.1. See also (c) below.

(b) In applying Eq. 6.9.1 to prestressing tendons, Δf_{ps} shall be substituted for f_s, where Δf_{ps} shall be taken as the calculated stress in the prestressing steel at service loads based on a cracked section analysis minus the decompression stress f_{dc}. It shall be permitted to take f_{dc} equal to the effective stress in the prestressing steel f_{se}. See also (c) below.

(c) In applying Eq. 6.9.1 to prestressing tendons, the magnitude of Δf_{ps} shall not exceed 250 N/mm^2. When Δf_{ps} is less than or equal to 140 N/mm^2, the spacing requirements of Sec 9.5.3.2(a) and (b) shall not apply.

(d) Where depth h of a beam exceeds 900 mm, the area of longitudinal skin reinforcement consisting of untensioned reinforcing steel or bonded tendons shall be uniformly distributed along both side faces of the member as required by Sec 6.3.6.7. The spacing s shall be determined using Sections 9.5.3.1 and 9.5.3.2 (a), (b) and (c). It shall be permitted to include such reinforcement in strength computations if a strain compatibility analysis is made to determine stress in the individual bars or wires.

9.5.4 Permissible Stresses in Prestressing Steel

Tensile stress in prestressing tendons shall not exceed the following:

(a) Due to prestressing steel jacking force $0.94 f_{py}$ but not greater than the lesser of $0.80 f_{pu}$ and the maximum value recommended by the manufacturer of prestressing steel or anchorage devices.

(b) Immediately after prestress transfer $0.82 f_{py}$ but not greater than $0.74 f_{pu}$.

(c) Post-tensioning tendons, at anchorage devices and couplers, immediately after force transfer $0.70 f_{pu}$

9.6 Losses of Prestress

Effective stress in prestressing steel is usually subject to different losses at different stages. Superimposed loads can result in gain of prestress due to bending of the member which shall be taken into consideration if significant. To determine effective stress in the prestressing steel, f_{se}, allowance for the following sources of loss of prestress shall be considered:

9.6.1 Immediate Losses

(a) Loss due to elastic shortening of concrete;

(b) Loss due to prestressing steel seating at transfer (Anchorage slip);

(c) Loss due to friction (for post-tensioned concrete only).

9.6.2 Long-term Losses

(a) Loss due to relaxation of prestressing steel stress;

(b) Loss due to creep of concrete;

(c) Loss due to shrinkage of concrete.

Unless otherwise determined by actual tests, allowance for these losses shall be made in accordance with the provisions of Sections 9.6.3 to 9.6.8.

9.6.3 Loss due to Elastic Shortening of Concrete

(a) The loss of prestress due to immediate elastic shortening of adjacent concrete upon transfer of initial prestress shall be calculated as specified in this section. For pretensioning, the loss of prestress in the tendons at transfer shall be calculated on a modular ratio basis using the stress in the adjacent concrete.

(b) For members with post-tensioned tendons which are not stressed simultaneously, there is a progressive loss of prestress during transfer due to the gradual application of the prestressing forces. This loss of prestress shall be calculated on the basis of half the product of the stress in the concrete adjacent to the tendons averaged along their lengths and the modular ratio. Alternatively, the loss of prestress may be exactly computed based on the sequence of tensioning.

9.6.4 Loss due to Prestressing Steel Seating at Transfer (Anchorage Slip)

(a) Any loss of prestress which may occur due to slip of wire or strand during anchoring or due to straining of the anchorage shall be allowed for in the design.

(b) Necessary additional elongation may be provided for at the time of tensioning to compensate for this loss.

9.6.5 Loss due to Relaxation of Prestressing Steel Stress

(a) The relaxation losses in prestressing steel shall be determined from experiments. When experimental values are not available, the relaxation losses, considering normal relaxation steel, may be assumed as given in Table 6.9.2.

Table 6.9.2: Relaxation Losses for Prestressing Steel at 1000 Hours at 27°C

Initial Stress	Relaxation Loss N/mm^2
$0.5f_{pu}$	0
$0.6f_{pu}$	35
$0.7f_{pu}$	70
$0.8f_{pu}$	90

For tendons at higher temperature or subject to large lateral loads, greater relaxation losses may be allowed, subject to the advice of the metallurgy specialist.

(b) No reduction in the value of the relaxation losses should be made for a tendon with a load equal to or greater than the relevant jacking force that has been applied for a short duration prior to the anchoring of the tendon.

9.6.6 Loss due to Creep of Concrete

(a) Creep occurs due to superimposed permanent dead load added to the member after it has been prestressed. Creep of concrete may be assumed to be proportional to the stress provided the stress in concrete does not exceed 40 percent of its compressive strength.

(b) In the absence of test data, the ultimate creep strain may be estimated from the following values of creep coefficient, which is the ratio of the ultimate creep strain to the elastic strain at the age of loading. Table 6.9.3 shows the values at different days.

Table 6.9.3: Creep Coefficient of Concrete

Age at Loading	Creep coefficient
7 days	2.2
28 days	1.6
1 year	1.1

(c) The ultimate creep strain estimated as above does not include the elastic strain. For the calculation of deformation at some stage before the total creep is reached, it may be assumed that 50 percent of the total creep takes place in the first month after loading and about 75 percent of the total creep takes place in the first six months after loading. For post-tensioning the creep coefficients shall be taken as 80% of those given here.

(d) The loss of prestress due to creep of concrete shall be determined for all the permanently applied loads including the prestress. Loss due to stresses of short duration including live load and erection stresses may be ignored.

(e) The loss of prestress due to creep of concrete shall be obtained as the product of the modulus of elasticity of the prestressing steel and the ultimate creep strain of the concrete fiber integrated along the centre-line of the prestressing steel over its entire length.

(f) The total creep strain during any specific period shall be assumed to be the creep strain due to sustained stress equal to the average of the stresses at the beginning and end of the period.

9.6.7 Loss due to Shrinkage of Concrete

(a) In the absence of test data, the approximate value of shrinkage strain in concrete for design purposes shall be assumed as follows:

For pretensioning : 0.0003

For post-tensioning : $0.0002/[log_{10}(t+2)]$

Where, t = age of concrete at transfer in days.

Other standard procedures like AASHTO LRFD Specifications may be used.

(b) For the calculation of deformation of concrete at some stage before the maximum shrinkage occurs it may be assumed that 50 percent of the shrinkage takes place during the first month and about 75 percent of the shrinkage takes place in the first six months after drying of concrete starts.

(c) The loss of prestress due to shrinkage of concrete shall be obtained as the product of the modulus of elasticity of steel and the shrinkage strain of concrete.

9.6.8 Loss due to Friction (For Post-tensioned Tendons Only)

(a) The design shall take into consideration all losses in prestress that may occur during tensioning due to friction between the post-tensioning tendons and the surrounding concrete or any fixture attached to the steel or concrete.

(b) The value of prestressing force P_x at a distance l_x metre from the jacking end and acting in the direction of the tangent to the curve of the cable shall be calculated from the relation:

$$P_x = P_j e^{-(Kl_x + \mu\alpha)} \qquad (6.9.2)$$

When $(Kl_x + \mu\alpha)$ is greater than 0.3, P_x may be computed from

$$P_x = \frac{P_j}{1 + Kl_x + \mu\alpha} \qquad (6.9.3)$$

For use in Equations 6.9.2 and 6.9.3, the values of wobble friction coefficient K and curvature friction coefficient μ shall be experimentally determined or obtained from the tendon manufacturer, and verified during tendon stressing operations.

(c) Values of K and μ used in the design shall be shown on design drawings

(d) In absence of test results or manufacturer's recommendation, the following values of μ and K shown in Table 6.9.4 may be taken as a guide:

Table 6.9.4: Friction Coefficients (K and μ) for Post-Tensioned Tendons

<table>
<tr><th colspan="3">Types of Tendons</th><th>Coefficient, K per meter</th><th>Curvature coefficient, μ per radian</th></tr>
<tr><td colspan="2" rowspan="3">Grouted Tendons in metal sheathing</td><td>Wire tendons</td><td>0.0033-0.0049</td><td>0.15-0.25</td></tr>
<tr><td>High-strength bars</td><td>0.0003-0.0020</td><td>0.08-0.30</td></tr>
<tr><td>7-wire strand</td><td>0.0016-0.0066</td><td>0.15-0.25</td></tr>
<tr><td rowspan="4">Unbonded tendons</td><td rowspan="2">Mastic coated</td><td>Wire tendons</td><td>0.0033-0.0066</td><td>0.05-0.15</td></tr>
<tr><td>7-wire strand</td><td>0.0033-0.0066</td><td>0.05-0.15</td></tr>
<tr><td rowspan="2">Pre-greased</td><td>Wire tendons</td><td>0.001-0.0066</td><td>0.05-0.15</td></tr>
<tr><td>7-wire strand</td><td>0.001-0.0066</td><td>0.05-0.15</td></tr>
</table>

9.6.9 Values of wobble and curvature friction coefficients used in design shall be shown on design drawings.

9.6.10 The effect of reverse friction shall be taken into consideration in such cases where the initial tension applied to a prestressing tendon is partially released (e.g., anchorage slip) and action of friction in the reverse direction causes significant alteration in the distribution of stress along the length of the tendon.

9.6.11 Where loss of prestress in a member occurs due to connection of member to adjoining construction, such loss of prestress shall be allowed for in design.

9.7 Control of Deflection

9.7.1 For prestressed concrete flexural members, designed in accordance with the provisions of this Chapter, immediate deflection shall be computed by usual methods or formulas for elastic deflections, and the moment of inertia of gross concrete section, I_g, shall be permitted to be used for Class U flexural members.

9.7.2 For Class C and Class T flexural members, deflection calculations shall be based on cracked transformed section analysis. It shall be permitted to base calculations on an effective moment of inertia, I_e as given in Eq. 6.9.4a.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad (6.9.4a)$$

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad (6.9.4b)$$

$$f_r = 0.62\,\lambda\sqrt{f_c'} \qquad (6.9.4c)$$

Deflection computed in accordance with Sec 9.7.1 shall not exceed the limits stipulated in Table 6.6.2, Chapter 6.

9.7.3 Additional long-term deflection of prestressed concrete members shall be computed taking into account stresses in concrete and steel under sustained load and including effects of creep and shrinkage of concrete and relaxation of steel.

9.8 Flexural Strength

9.8.1 Design moment strength of flexural members shall be computed by the strength methods of the Code. For prestressing steel, f_{ps} shall be substituted for f_y in strength computations.

9.8.2 As an alternative to a more accurate determination of f_{ps} based on strain compatibility, the following approximate values of f_{ps} shall be permitted to be used if f_{se} is not less than $0.5 f_{pu}$.

(a) For members with bonded tendons

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left\{\rho_p \frac{f_{pu}}{f_c'} + \frac{d}{d_p}(\omega - \omega')\right\}\right] \qquad (6.9.5)$$

Where, $\omega = \frac{\rho f_y}{f_c'}, \omega' = \frac{\rho' f_y}{f_c'}$ and γ_p is 0.55 for f_{py}/f_{pu} not less than 0.80; 0.40 for f_{py}/f_{pu} not less than 0.85; and 0.28 for f_{py}/f_{pu} not less than 0.90.

If any compression reinforcement is taken into account when calculating f_{ps} by Eq. 6.9.5:

The term $\left[\rho_p \frac{f_{pu}}{f_c'} + \frac{d}{d_p}(\omega - \omega')\right]$ shall be taken not less than 0.17 and d' shall be no greater than $0.15d_p$.

(b) For members with unbonded tendons and with a span-to-depth ratio of 35 or less:

$$f_{ps} = f_{se} + 70 + \frac{f_c'}{100\rho_p} \qquad (6.9.6)$$

But f_{ps} in Eq. 6.9.6 shall not be taken greater than the lesser of f_{py} and $(f_{se} + 420)$.

(c) For members with unbonded tendons and with a span-to-depth ratio greater than 35:

$$f_{ps} = f_{se} + 70 + \frac{f_c'}{300\rho_p} \qquad (6.9.7)$$

But, f_{ps} in Eq. 6.9.7 shall not be taken greater than the lesser of f_{py} and $(f_{se} + 210)$

9.8.3 Non prestressed reinforcement conforming to Sec 5.3 Chapter 5 of this Part, if used with prestressing steel, shall be permitted to be considered to contribute to the tensile force and to be included in moment strength computations at a stress equal to f_y. Other non prestressed reinforcement shall be permitted to be included in strength computations only if a strain compatibility analysis is performed to determine stresses in such reinforcement.

9.9 Limits For Flexural Reinforcement

9.9.1 Prestressed concrete sections shall be classified as either tension-controlled, transition, or compression-controlled sections, in accordance with a. and b. below.

(a) Sections are compression-controlled if the net tensile strain in the extreme tension fiber ε_t, is equal to or less than the compression-controlled strain limit when the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 420 reinforcement, and for all prestressed reinforcement, it shall be permitted to set the compression-controlled strain limit to 0.002.

(b) Sections are tension-controlled if the net tensile strain in the extreme tension steel, ε_t, is equal to or greater than 0.005 when the concrete in compression reaches its assumed strain limit of 0.003. Sections with ε_t between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections. Appropriate strength reduction factor, ϕ, from
Sec 9.9.2 shall apply.

9.9.2 The appropriate strength reduction factor, ϕ , shall apply as given in (a) to (f) below.

(a) Tension-controlled sections 0.90

(b) For compression-controlled sections

(i) Members with spiral reinforcement as defined in Sec 6.2.3.2.20.75

(ii) Other reinforced members 0.65

(c) Shear and torsion 0.75

(d) Post-tensioned anchorage zones 0.85

(e) Strut and tie models 0.75

(f) Flexural sections in pre-tensioned members where strand embedment length is less than the development length

(i) From the end of the member to the end of the transfer length 0.75

(ii) From the end of transfer length to the end of the development length, ϕ shall be taken as 0.75 to 0.90

Where bonding of the strand does not extend to the end of the member, strand embedment shall be assumed to begin at the end of the debonded length.

9.9.3 Total amount of prestressed and non-prestressed reinforcement in members with bonded prestressed reinforcement shall be adequate to develop a factored load at least 1.2 times the cracking load computed on the basis of the modulus of rupture f_c, as given in Sec 9.4.5.4. This provision shall be permitted to be waived for flexural members with shear and flexural strength at least twice the required strength (U) calculated for the factored loads and forces in such combinations as are stipulated in Chapter 2, Loads.

9.9.4 Minimum Bonded Reinforcement

9.9.4.1 A minimum area of bonded reinforcement shall be provided in all flexural members with unbonded tendons as required by Sections 9.9.4.2 and 9.9.4.3.

9.9.4.2 Except as provided in Sec 9.9.4.3, minimum area of bonded reinforcement shall be computed by

$$A_s = 0.004A_{ct} \qquad (6.9.8)$$

Where, A_{ct} is area of that part of cross-section between the flexural tension face and center of gravity of gross-section.

(a) Bonded reinforcement required by Eq. 6.9.8 shall be uniformly distributed over pre-compressed tensile zone as close as practicable to extreme tension fibre.

(b) Bonded reinforcement shall be required regardless of service load stress conditions.

9.9.4.3 For two-way flat slab systems, minimum area and distribution of bonded reinforcement shall be as required in (a), (b) and (c) below.

(a) Bonded reinforcement shall not be required in positive moment areas where f_t, the extreme fibre stress in tension in the precompressed tensile zone at service loads (after allowance for all prestress losses), does not exceed $0.17\sqrt{f_c'}$.

(b) In positive moment areas where computed tensile stress in concrete at service load exceeds $0.17\sqrt{f_c'}$ minimum area of bonded reinforcement shall be computed by

$$A_s = \frac{N_c}{0.5\, f_y} \qquad (6.9.9)$$

Where, the value of f_y used in Eq. 6.9.9 shall not exceed 420 MPa. Bonded reinforcement shall be uniformly distributed over precompressed tensile zone as close as practicable to the extreme tension fibre.

(c) In negative moment areas at column supports, the minimum area of bonded reinforcement A_s in the top of the slab in each direction shall be computed by

$$A_s = 0.00075 A_{cf} \tag{6.9.10}$$

Where, A_{cf} is the larger gross cross-sectional area of the slab-beam strips in two orthogonal equivalent frames intersecting at a column in a two-way slab.

9.9.4.4 Bonded reinforcement required by Eq. 6.9.10 shall be distributed between lines that are $1.5h$ outside opposite faces of the column support. At least four bars or wires shall be provided in each direction. Spacing of bonded reinforcement shall not exceed 300 mm.

9.9.4.5 Minimum length of bonded reinforcement required by Sections 9.9.4.2 and 9.9.4.3 shall be as required in Sec 9.9.4.5 (a), (b) and (c).

(a) In positive moment areas, minimum length of bonded reinforcement shall be one-third the clear span length, l_n and centered in positive moment area.

(b) In negative moment areas, bonded reinforcement shall extend one-sixth the clear span, l_n on each side of support.

(c) Where bonded reinforcement is provided for ϕM_n in accordance with Sec 9.8.3 or for tensile stress conditions as per Sec 9.9.4.3 (b), minimum length also shall conform to provisions of Chapter 6.

9.10 Statically Indeterminate Structures

9.10.1 Frames and continuous construction of prestressed concrete shall be designed for satisfactory performance at service load conditions and for adequate strength.

9.10.2 Performance at service load conditions shall be determined by elastic analysis, considering reactions, moments, shears, and axial forces induced by prestressing, creep, shrinkage, temperature change, axial deformation, restraint of attached structural elements, and foundation settlement.

9.10.3 Moments used to compute required strength shall be the sum of the moments due to reactions induced by prestressing (with a load factor of 1.0) and the moments due to factored loads. Adjustment of the sum of these moments shall be permitted as allowed in Sec 9.10.4.

9.10.4 Redistribution of moments in continuous prestressed flexural members shall be:

(a) Where bonded reinforcement is provided at supports in accordance with Sec 9.9.4, it shall be permitted to decrease negative or positive moments calculated by elastic theory for any assumed loading, in accordance with Sec 9.10.4 (b) and (c) below.

(b) Except where approximate values for moments are used, it shall be permitted to decrease factored moments calculated by elastic theory at sections of maximum negative or maximum positive moment in any span of continuous flexural members for any assumed loading arrangement by not more than $1000\varepsilon_t$ percent, with a maximum of 20 percent.

(c) Redistribution of moment shall be made only when ε_t is equal to or greater than 0.0075 at the section at which moment is reduced.

9.10.5 The reduced moment shall be used for calculating redistributed moments at all other sections within the spans. Static equilibrium shall be maintained after redistribution of moments for each loading arrangement.

9.11 Compression Members - Combined Flexure And Axial Load

9.11.1 Prestressed Concrete Members Subject to Combined Flexure and Axial Load

With or without non-prestressed reinforcement, Prestressed concrete members subject to combined flexure and axial load shall be proportioned by the strength design methods of this Code. Effects of prestress, creep, shrinkage, and temperature change shall be included.

9.11.2 Limits for Reinforcement of Prestressed Compression Members

9.11.2.1 Members with average compressive stress in concrete less than 1.6 N/mm^2, due to effective prestress force only, shall have minimum reinforcement in accordance with Sections 6.3.9.1, 6.3.9.2 for columns and Sec 6.6.3 for walls and minimum transverse reinforcement for compression members of Chapter 6.

9.11.2.2 Except for walls, members with average compressive stress in concrete due to effective prestress force only, equal to or greater than 1.6 N/mm^2 shall have all tendons enclosed by spirals or lateral ties in accordance with (a) through (d).

(a) Spirals shall conform to the spiral reinforcement requirement for compression members of this Code and Sec 9.11.3.

(b) Lateral ties shall be at least No. 10 in size or welded wire reinforcement of equivalent area, and shall be spaced vertically not to exceed 48 tie bar or wire diameters, or the least dimension of the compression member.

(c) Ties shall be located vertically not more than half a tie spacing above top of footing or slab in any story, and not more than half a tie spacing below the lowest horizontal reinforcement in members supported above.

(d) Where beams or brackets frame into all sides of a column, ties shall be terminated not more than 75 mm below lowest reinforcement in such beams or brackets.

9.11.2.3 For walls with average compressive stress in concrete due to effective prestress force only equal to or greater than 1.6 N/mm², minimum reinforcement required by Sec 6.6.3 shall not apply where structural analysis shows adequate strength and stability.

9.11.3 Volumetric Spiral Reinforcement Ratio

Volumetric spiral reinforcement ratio, ρ_s shall be not less than the value given by

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}} \qquad (6.9.11)$$

Where, the value of f_{yt} in Eq. 6.9.11 shall not exceed 700 N/mm². For f_{yt} greater than 420 N/mm², lap splices according to Sec 9.9.3.1(a) shall not be used.

(a) Spiral reinforcement shall be spliced, if needed, by any one of the following methods:

Lap splices not less than the larger of 300 mm and the length indicated in Sec 8.1.9.3 (a) to (e) of Chapter 8 and summarized below:

(i) deformed uncoated bar or wire $48d_b$

(ii) plain uncoated bar or wire $72d_b$

(iii) epoxy-coated deformed bar or wire $72d_b$

(iv) plain uncoated bar or wire with a standard stirrup or tie hook in accordance with Sec 8.1.9.3 (d) of Chapter 8 at ends of lapped spiral reinforcement.

(b) The term "standard hook" as used in this Code shall mean one of the following:

(i) 180-degree bend plus $4d_b$ extension, but not less than 65 mm at free end of bar.

(ii) 90-degree bend plus $12d_b$ extension at free end of bar.

(c) For stirrup and tie hooks

(i) No. 16 bar and smaller, 90° bend plus $6d_b$ extension at free end of bar; or

(ii) No. 19, No. 22 bar and No. 25 bar, 90° bend plus $12d_b$ extension at free end of bar; or

(iii) No. 25 bar and smaller, 135° bend plus $6d_b$ extension at free end of bar.

9.12 Slab Systems

9.12.1 Factored moments and shears in prestressed slab systems reinforced for flexure in more than one direction shall be determined in accordance with provisions of Sec 6.5.7 Chapter 6 or by more detailed design procedures.

9.12.2 ϕM_n of prestressed slabs with loads and load combinations required by Chapter 2 and 6 at every section shall be greater than or equal to M_u considering Sections 9.10.3 and 9.10.4. ϕV_n (design strength) of prestressed slabs at columns following Chapter 6 shall be greater than or equal to V_u (the required strength, Chapter 2).

9.12.3 At service load conditions, all serviceability limitations, including limits on deflections, shall be met, with appropriate consideration of the factors listed in Sec 9.10.2.

9.12.4 For uniformly distributed loads, spacing of tendons or groups of tendons in at least one direction shall not exceed the smaller of eight times the slab thickness and 1.5 m. Spacing of tendons also shall provide a minimum average effective prestress of 0.9 N/mm^2 on the slab section tributary to the tendon or tendon group. For slabs with varying cross section along the slab span, either parallel or perpendicular to the tendon or tendon group, the minimum average effective prestress of 0.9 N/mm^2 is required at every cross section tributary to the tendon or tendon group along the span. Concentrated loads and opening in slabs shall be considered when determining tendon spacing.

9.12.5 In slabs with unbonded tendons, bonded reinforcement shall be provided in accordance with Sections 9.9.4.3 to 9.9.4.5.

9.12.6 Except as permitted in Sec 9.12.7, in slabs with unbonded tendons, a minimum of two 12.7 mm diameter or larger, seven-wire post-tensioned strands shall be provided in each direction at columns, either passing through or anchored within the region bounded by the longitudinal reinforcement of the column. Outside column and shear cap faces, these two structural integrity tendons shall pass under any orthogonal tendons in adjacent spans. Where the two structural integrity tendons are anchored within the region bounded by the longitudinal reinforcement of the column, the anchorage shall be located beyond the column centroid and away from the anchored span.

9.12.7 Prestressed slabs not satisfying Sec 9.12.6 shall be permitted provided they contain bottom reinforcement in each direction passing within the region bounded by the longitudinal reinforcement of the column and anchored at exterior supports as required by Sec 6.5.3.8 Chapter 6. The area of bottom reinforcement in each direction shall be not less than 1.5 times that required by Eq. 6.9.12 as given below.

$$A_{s,min} = \frac{0.25\sqrt{f_c'}}{f_y}\, b_w d \qquad (6.9.12)$$

and not less than $2.1 b_w d/f_y$, where b_w is the width of the column face through which the reinforcement passes. Minimum extension of these bars beyond the column or shear cap face shall be equal to or greater than the bar development length required by Sec 8.2.

9.12.8 In lift slabs, bonded bottom reinforcement shall be detailed in accordance with Sec 9.12.9.

9.12.9 In slabs with shear heads and in lift slab construction where it is not practical to pass to the bottom bars, required by bar detailing requirement of Sec 6.5.3.8 Chapter 6, at least two bonded bars or wires in each direction shall pass through the shear head or lifting collar as close to the column as practicable and be continuous or spliced with a Class A splice. At the exterior columns, the reinforcement shall be anchored the spear head or lifting collar.

9.13 Post-Tensioned Tendon Anchorage Zones

9.13.1 Division into Zones

The anchorage zone shall be considered as composed of two zones as described below and shown in Figure 6.9.1.

(a) The local zone is the rectangular prism (or equivalent rectangular prism for circular or oval anchorages) of concrete immediately surrounding the anchorage device and any confining reinforcement;

(b) The general zone is the anchorage zone beyond the local zone.

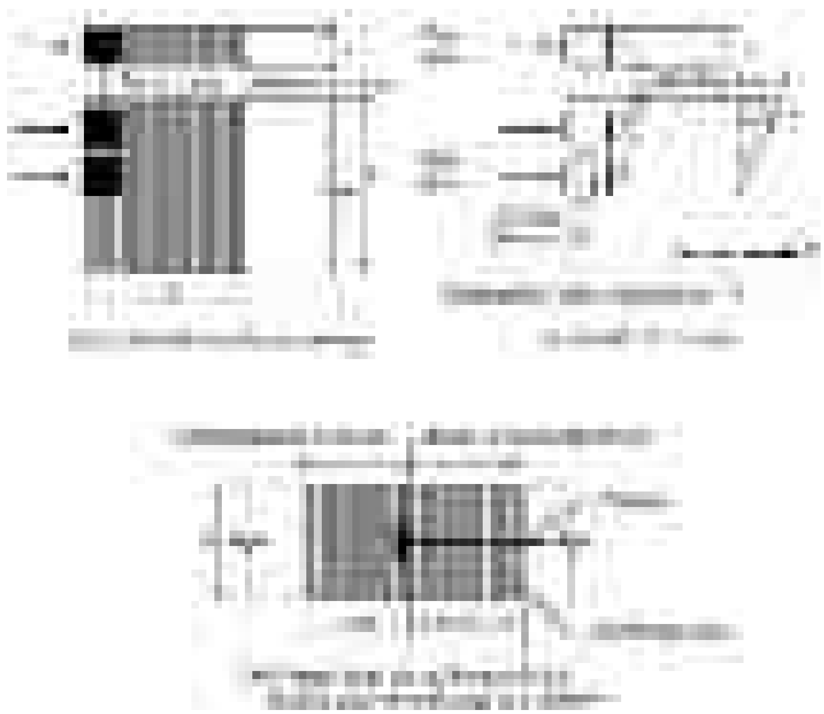


Figure 6.9.1 Anchorage zones

9.13.2 Local Zone

9.13.2.1 Design of local zones shall be based upon the factored prestressing force, P_{pu} and the requirements of Sections 9.9.2 (d)-(f) and 9.13.2.2.

9.13.2.2 For post-tensioned anchorage zone design, a load factor of 1.2 shall be applied to the maximum steel jacking force.

9.13.2.3 Local-zone reinforcement shall be provided where required for proper functioning of the anchorage device.

9.13.3 General Zone

9.13.3.1 Design of general zones shall be based upon the factored prestressing force, P_{pu} and the requirements of Sec 9.4.14.3 b and c.

9.13.3.2 General-zone reinforcement shall be provided where required to resist bursting, spalling, and longitudinal edge tension forces induced by anchorage devices. Effects of abrupt change in section shall be considered.

The general zone requirements of Sec 9.13.3.2 are satisfied by Sections 9.13.4, 9.13.5, and 9.13.6 and whichever one of Sec 9.4.15.2 or Sec 9.4.15.3 or Sec 9.4.16.3 is applicable.

9.13.4 Design Methods

9.13.4.1 The following methods shall be permitted for the design of the general zones of the prestressed components provided that the specific procedures used result in prediction of strength in substantial agreement with results of comprehensive tests:

(a) Equilibrium-based plasticity models (strut-and-tie models);

(b) Linear stress analysis (including finite element analysis or equivalent); or

(c) Simplified equations where applicable.

9.13.4.2 Simplified equations shall not be used where member cross-sections are nonrectangular, where discontinuities in or near the general zone cause deviations in the force flow path, where minimum edge distance is less than 1-1/2 times the anchorage device lateral dimension in that direction, or where multiple anchorage devices are used in other than one closely spaced group.

9.13.4.3 The stressing sequence shall be considered in the design and specified on the design drawings.

9.13.4.4 Three-dimensional effects shall be considered in design and analyzed using three-dimensional procedures or approximated by considering the summation of effects for two orthogonal planes.

9.13.4.5 For anchorage devices located away from the end of the member, bonded reinforcement shall be provided to transfer at least $0.35A_{ps}f_{pu}$ into the concrete section behind the anchor. Such reinforcement shall be placed symmetrically around the anchorage devices and shall be fully developed both behind and ahead of the anchorage devices.

9.13.4.6 Where tendons are curved in the general zone, except for mono-strand tendons in slabs or where analysis shows reinforcement is not required, bonded reinforcement shall be provided to resist radial and splitting forces.

9.13.4.7 Except for mono-strand tendons in slabs or where analysis shows reinforcement is not required, minimum reinforcement with a nominal tensile strength equal to 2 percent of each factored prestressing force shall be provided in orthogonal directions parallel to the back face of all anchorage zones to limit spalling.

9.13.4.8 Tensile strength of concrete shall be neglected in calculations of reinforcement requirements.

9.13.5 Nominal Material Strengths

9.13.5.1 Tensile stress at nominal strength of bonded reinforcement is limited to f_y for nonprestressed reinforcement and to f_{py} for prestressed reinforcement. Tensile stress at nominal strength of unbounded prestressed reinforcement for resisting tensile forces in the anchorage zone shall be limited to $f_{ps} = f_{se} + 70$.

9.13.5.2 Except for concrete confined within spirals or hoops providing confinement equivalent to that corresponding to Eq. 6.9.11, compressive strength in concrete at nominal strength in the general zone shall be limited to $0.7\lambda f'_{ci}$.

9.13.5.3 Concrete strength at transfer (Anchorage): Unless oversize anchorage devices are sized to compensate for the lower compressive strength or the prestressing steel is stressed to no more than 50 percent of the final prestressing force, prestressing steel shall not be stressed until compressive strength of concrete as indicated by tests consistent with the curing of the member, is at least 28 N/mm^2 for multi-strand tendons or at least 17 N/mm^2 for single-strand or bar tendons. Compressive strength of concrete at the time of post-tensioning shall be specified in the contract documents and in design drawings.

9.13.6 Detailing Requirements

Selection of reinforcement sizes, spacing, cover, and other details for anchorage zones shall make allowances for tolerances on the bending, fabrication, and placement of reinforcement, for the size of aggregate, and for adequate placement and consolidation of the concrete.

9.14 Design of Anchorage Zones For Monostrand or Single 16 Mm Diameter Bar Tendons

9.14.1 Local Zone Design

Monostrand or single 16 mm diameter or smaller diameter bar anchorage devices and local zone reinforcement shall meet the requirements of ACI 423.7 or the special anchorage device requirements of Sec 9.15.2.

9.14.2 General Zone Design for Slab Tendons

9.14.2.1 For anchorage devices of 12.7 mm diameter or smaller diameter strands in normal weight concrete slabs, minimum reinforcement meeting the requirements of Sections 9.14.2.2 and 9.14.2.3 shall be provided unless a detailed analysis satisfying Sec 9.13.4 shows such reinforcement is not required.

9.14.2.2 Two horizontal bars at least 12 mm diameter in size shall be provided parallel to the slab edge. They shall be permitted to be in contact with the front face of the anchorage device and shall be within a distance of h/2 ahead of each device. Those bars shall extend at least 150 mm either side of the outer edges of each device.

9.14.2.3 If the center-to-center spacing of anchorage devices is 300 mm or less, the anchorage devices shall be considered as a group. For each group of six or more anchorage devices, (n+1) hairpin bars or closed stirrups at least No. 10 in size shall be provided, where n is the number of anchorage devices. One hairpin bar or stirrup shall be placed between each anchorage device and one on each side of the group. The hairpin bars or stirrups shall be placed with the legs extending into the slab perpendicular to the edge. The center portion of the hairpin bars or stirrups shall be placed perpendicular to the plane of the slab from 3h/8 to h/2 ahead of the anchorage devices.

9.14.2.4 For anchorage devices not conforming to Sec 9.14.2.1, minimum reinforcement shall be based upon a detailed analysis satisfying Sec 9.13.4.

9.14.3 General Zone Design for Groups of Monostrand Tendons in Beams and Girders

Design of general zones for groups of monostrand tendons in beams and girders shall meet the requirements of Sections 9.13.3 and 9.13.4.

9.15 Design of Anchorage Zones For Multi-Strand Tendons

9.15.1 Local Zone Design

Basic multistrand anchorage devices and the related local and general zone reinforcement shall meet the requirements of AASHTO "LRFD Bridge Design Specifications (SI), 2007", Articles 5.10.9.6, Approximate Stress Analysis and Design, and 5.10.9.7, Design of Local Zones.

Special Anchorage Devices (AASHTO "LRFD Bridge Design Specifications (SI), 2007", Articles 5.10.9.7.3) requires that special anchorage devices that do not satisfy the requirements specified in Sec 9.15.1, they have been tested by an independent testing agency acceptable to the Engineer and have met the acceptance criteria specified in Articles 10.3.2 and 10.3.2.3.10 of AASHTO LRFD Bridge Construction Specifications.

9.15.2 Special Anchorage Devices

Where special anchorage devices are to be used, supplemental skin reinforcement shall be furnished in the corresponding regions of the anchorage zone, in addition to the confining reinforcement specified for the anchorage device. This supplemental reinforcement shall be similar in configuration and at least equivalent in volumetric ratio to any supplementary skin reinforcement used in the qualifying acceptance tests of the anchorage device.

9.15.3 General Zone Design

Design for general zones for multistrand tendons shall meet the requirements of Sections 9.13.3 to 9.13.5.

9.16 Cold Drawn Low Carbon Wire Prestressed Concrete (Cwpc)

9.16.1 CWPC (Cold drawn wire prestressed concrete) is termed as prestressed concrete technology of Chinese pattern. This technology is a modification of conventional prestressed concrete. In the conventional prestressed concrete high strength wire is used as reinforcement while in Chinese pattern cold drawn low carbon mild steel wire is used as such this technology is named as cold drawn wire prestressed concrete. In short it is termed as CWPC. CWPC technology is a process whereby cold drawn low carbon steel wire has been adopted as reinforcement for pre-fabricated prestressed concrete members of medium and small size as produced by pre tensioning method. On the other hand, large size structural members are produced by conventional prestressed concrete. The main features and advantages of CWPC technology can be summarized as follows:

(a) Availability (Availability of materials): The raw material of cold drawn wire is made from low carbon mild steel which can be supplied by the local mills. The tensioning process of cold-drawn wire and production of pre-cast members are also simple and very easy to handle.

(b) Simplicity (Simplicity of equipment and devices for production): The cold process of low carbon mild steel and prefabrication process of members are done using simple equipment and devices. The precise and large sized equipment are not necessary. The production techniques of manufacturing members are rather simple.

(c) Quality (Good in quality): The members so manufactured have high crack resistance and stiffness. After pre-tensioning no crack would occur under the service load, thus the wires within the concrete members are well protected. In contrast to conventional reinforced concrete members under the same service conditions, they have comparatively high durability to ensure long term quality.

(d) Economy (Low cost): The cold drawn low carbon steel wire used for prestressing is made of ordinary hot-rolled carbon steel coil rod. This is processed at room temperature through a special wire drawing die. The low carbon coil rods are manufactured by the steel mills; the wires are processed at the construction site or in a prefabrication plant; or are supplied by the cold drown wire plants as readymade products. By cold drawing the low carbon rod into wires the usable strength is enhanced about twice as much as that of the coil rod. This reduces the amount of steel required in prefabricating prestressed concrete members.

(e) Therefore, in comparison with conventional reinforced concrete reinforced with common carbon steel, a prestressed concrete member reinforced with cold drawn wire would have saving of steel consumption between 30-40%. Furthermore, since prestressed concrete members have high stiffness a reduction of cross section of members is possible. A considerable amount of concrete can also be saved and hence transportation, handling and erection work can be reduced.

(f) Light weight (Lightness in weight): As already mentioned that the stiffness of prestressed concrete members may be enhanced, the dimension of its cross-section can be reduced correspondingly. This not only results in reduction of concrete volume but also its dead weight which is estimated as 10-30%.

9.16.2 Materials

Basically the materials used in CWPC technology are steel and concrete.

(a) Steel: steel used for CWPC is obtained by cold drawing. Cold drawing as already mentioned is a process of reducing the diameter of the coil rod by forcing it to pass through a conical die. By this process, the usable strength of steel can be increased by nearly 100%.

(b) Concrete: The requirement of concrete in CWPC is same as that of ordinary reinforced concrete.

9.16.3 Design

Similar to other reinforced concrete structures, CWPC structures have a complete set of design specification and computational approaches by which various members of the CWPC can be designed. In the design of prestressed members the function of pre-stressing force and pre-stressing losses should be calculated. CWPC members should be checked for its strength, stability and cracking resistance respectively at different stages including service, manufacturing, handling, erection and construction. In designing members conformity to local specifications should be considered.

Cold drawn low carbon wire conforming to ASTM A615 or equivalent may be permitted for prestressing provided the mechanical requirements shown in Table 6.9.5 are satisfied.

Table 6.9.5: Tensile Strength and Elongation of Cold Drawn Wire

Diameter of wire (mm)	Minimum tensile strength (N/mm^2)	Minimum elongation (percent)
3	650	2.0
4	600	2.5
5	550	3.0

9.17 External Post-Tensioning

9.17.1 Post-tensioning tendons shall be permitted to be external to any concrete section of a member. The strength and serviceability design methods of this Code shall be used in evaluating the effects of external tendon forces on the concrete structure.

9.17.2 External tendons shall be considered as unbonded tendons when computing flexural strength unless provisions are made to effectively bond the external tendons to the concrete section along its entire length.

9.17.3 External tendons shall be attached to the concrete member in a manner that maintains the desired eccentricity between the tendons and the concrete centroid throughout the full range of anticipated member deflection.

9.17.4 External tendons and tendon anchorage regions shall be protected against corrosion, and the details of the protection method shall be indicated on the drawings or in the project specifications.

9.18 Performance Requirement Of Prestressed Concrete Design

9.18.1 Classification of Performance Requirement

After the outline of the member dimensions are determined and the most suitable kind and type of prestressing options are selected at the structural planning stage, the prestressed concrete non-composite and composite structures and members shall satisfy all of the required performances such as safety, serviceability, restorability, durability, reparability, societal and environmental compatibility, etc. at every stage of design, construction and maintenance throughout the design life of the structure. Table 6.9.6 gives the performance requirement of prestressed concrete structures and components and related performance items.

Table 6.9.6: Classification of Performance Requirement for Prestressed Concrete Structures

Performance requirements	Performance item	Examples of check items	Example of verification index
Safety	Structural safety	Resistance of whole structure, components, stability, deformation performance	Stress resultant, stress
	Public safety	Injury to users and third parties	-
Serviceability	Live load operating performance	Soundness and rigidity of structures /members under usual conditions	Floor flatness, deformation of main girder
	User comfort	User-comfort under walking-induced vibrations	Natural frequency of main girders
Restorability	Restorability after earthquake, cyclone, tidal bore, fire, etc.	Level of damage (ease of restoration)	Response value (damage level)/ limit value of performance (damage level)
Durability	Fatigue resistance	Fatigue durability against variable actions	Equivalent stress range/ allowable stress range
	Corrosion resistance	Rust prevention and corrosion protection performance of steel material	Corrosion environment and surface finish, paint specification
	Resistance to material deterioration	Concrete deterioration	Water- cement ratio, cover of concrete
	Maintainability	Ease of maintenance (inspection, ease of repair, etc.) and ease of restoration	-

Performance requirements	Performance item	Examples of check items	Example of verification index
Social and environmental compatibility	Social compatibility	Appropriateness of partial factor (consideration of social importance of structure)	Partial factor, structural factor, etc.
	Economic rationality	Social utility during life cycle of structure	Life cycle cost (LCC), life cycle utility (LTU)
	Environment al compatibility	Noise, vibration, environmental impact, aesthetics, etc.	Noise and vibration levels for surrounding residents, aesthetic reaction to structural shape and color, monumental aspect, etc.
Constructabili ty / workability	Safety during construction	Safety during construction	Stress resultant, stress, deformation
	Initial soundness	Material quality, welding quality, etc.	Material properties, workmanship
	Ease of construction	Ease of fabrication and construction work	User-friendly construction methodology conceived at design stage

9.18.2 Performance Verification Method

(a) Performance verification shall be based on the partial factor method on the basis of reliability theory and as a standard design procedure, it shall be based on the limit state method.

(b) In general verification shall be based on design responses to design actions, design limits as determined by design material strengths, and individual partial factors. The performance of the structure shall, in general, be verified using Equations 6.9.13 and 6.9.14:

$$\gamma_i \frac{S_d}{R_d} \leq 1.0 \qquad (6.9.13)$$

$$\gamma_i \frac{\sum \gamma_a S(\gamma_f F_k)}{R(f_k/\gamma_m)} \leq 1.0 \qquad (6.9.14)$$

Where, R_d : design resistance

f_k : characteristic value of material strength

γ_m : material factor

γ_b : structural member factor

R(...)	:	function for calculating limit value of structure from material strength
S_d	:	design response
F_k	:	individual characteristic value of action
γ_a	:	structural analysis factor
γ_f	:	action factor corresponding to each action (load factor)
S(...)	:	function for calculating response value of structure from action
γ_i	:	structural factor

(c) During design, a verification shall be carried out for every limit state that can be considered.

(d) The flow chart explaining the concept of verification of safety is given in Figure 6.9.2.

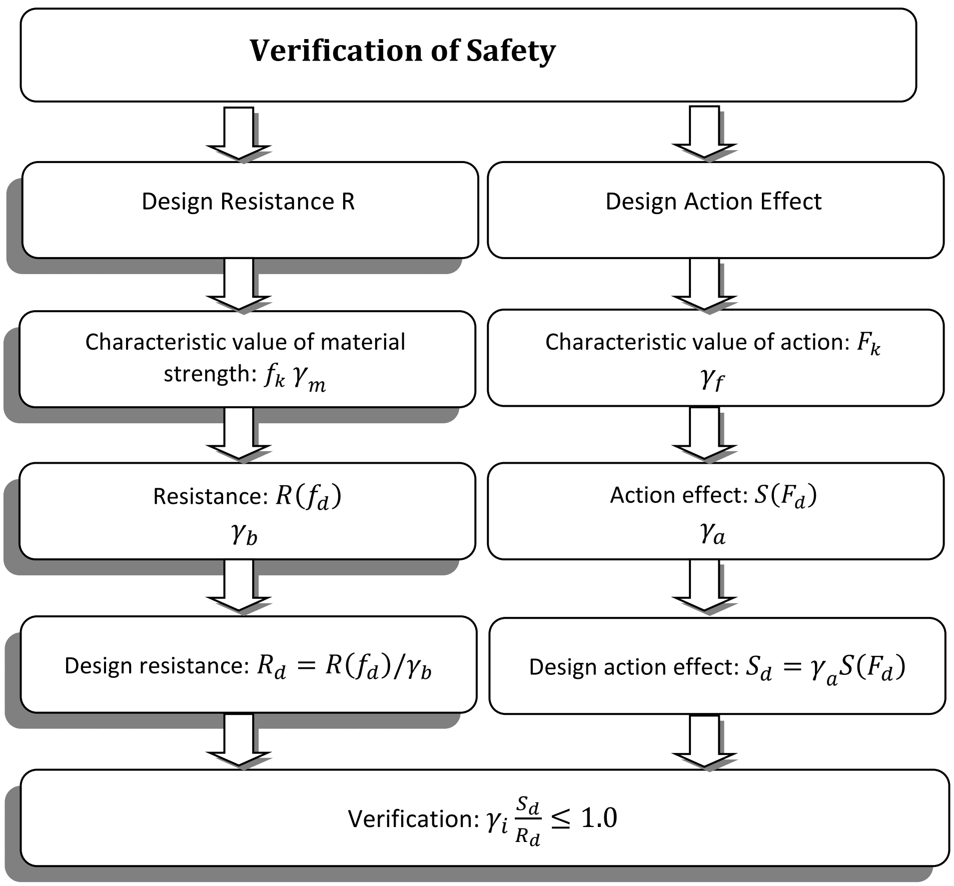


Figure 6.9.2 Flow chart explaining the concept of verification of safety

9.18.3 Partial Factors

(a) Partial factors shall be determined on the concept given (i) and (ii) below.

(i) The material factor, structural member factor, structural analysis factor, and action factor shall be determined in consideration of

- unfavorable deviations from characteristic values,
- uncertainties in computational accuracy, and
- discrepancies between design and practice with respect to actions or structures and materials.

Table 6.9.7 shows the standard values of partial factors.

(ii) The structural factor γ_i shall be determined according to structural importance and also the social and economic impact of the structure reaching its limit state.

Table 6.9.8 shows the standard values of structural factor γ_i for different performance items.

Table 6.9.7: Standard Values of Partial Factors

Performance Item	Action Factor, γ_f	Structural Analysis Factor, γ_a	Material Factor, γ_m	Structural Member Factor, γ_b
Structural safety	1.0 - 1.6	1.0 - 1.1	1.0 - 1.05	1.0 - 1.3
Serviceability (user comfort)	1.0	1.0	1.0 - 1.05	1.0
Durability (fatigue resistance)	1.0 - 1.1	1.0	1.0	1.0 - 1.1

Table 6.9.8: Standard Values of Structural Factors

Performance item	Structural factor γ_i
Structural safety	1.0 - 1.2
Serviceability (User comfort)	1.0
Durability (fatigue resistance)	1.0

Division B: Material and Construction (Sections 9.19 To 9.21)

9.19 Materials

9.19.1 Concrete Ingredients and Applicable ASTM Standards

Table 6.9.9 shows the list of commonly applicable standards for cement, coarse and fine aggregates, admixtures and mixing water.

Table 6.9.9: Applicable Standards for Cement, Coarse and Fine Aggregates, Admixtures and Water

Material	Designation of the Standard	Title of the Standard
Concrete	ASTM C39	Compression testing of cylindrical concrete specimens
Cement	BDS EN 197-1	Part 1: Composition, specifications and conformity criteria for common cements
Fine and Coarse aggregates	ASTM C136	Standard test method for sieve analysis of fine and coarse aggregates
	ASTM C40	Standard test method for organic impurities in fine aggregates for concrete
	ASTM C142	Clay lumps and friable particles
	ASTM C127	Specific gravity and absorption of coarse aggregate
	ASTM C128	Specific gravity and absorption of fine aggregate
	ASTM C131	Degradation of small-size coarse aggregate by L.A. abrasion test
	ASTM C29	Unit weights and voids in aggregates
	ASTM C70	Surface moisture in fine aggregate Soundness of aggregates by use of sodium sulfate or magnesium sulfate
	ASTM C88	Soundness of aggregates by use of sodium sulfate or magnesium sulfate
	ASTM C227	Alkali reactivity, potential of cement aggregate combinations

Material	Designation of the Standard	Title of the Standard
	ASTM C1260	Potential alkali reactivity of aggregates (Mortar-bar method)
	ASTM D2419	Sand equivalent value of soils and fine aggregate
Admixtures	ASTM C494	Type A – Water reducing Type B – Retarding Type C – Accelerating Type D – Water reducing and retarding Type E – Water reducing and accelerating Type F – Water reducing, high range Type G – Water reducing, high range and retarding Type S – Specific performance admixture
Mixing Water	ASTM C 1602/C1602M	Standard specification for mixing water used in the production of hydraulic cement concrete

9.19.2 Reinforcing Steel and Applicable Standards

Table 6.9.10 shows the types of reinforcing steel with the ASTM and BDS Designation standard specifications.

Table 6.9.10: List of Standards for the Reinforcing Steel

Material	Designation of the Standard	Title of the Standard
Reinforcing Steel	BDS ISO 6935-2	Bangladesh standard, Steel for the reinforcement of concrete, Part 2: Ribbed bars (1st revision)
	ASTM A615/A615M	Standard specifications for deformed and plain carbon steel bars for concrete reinforcement
	ASTM A706/A706M	Standard specifications for low-alloy steel deformed and plain carbon steel bars for concrete reinforcement
	A775/A775M	Standard Specification for Epoxy-Coated Steel Reinforcing Bars

Material	Designation of the Standard	Title of the Standard
	A884/A884M	Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Reinforcement
	A934/A934M	Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars
	ASTM A996/ A996M	Specification for Axle Steel Deformed and Plain Bars for Concrete Reinforcement
	ASTMA996/ A996M	Specification for Rail Steel Deformed and Plain Bars for Concrete Reinforcement" Including Supplementary Requirements S1

9.19.3 Prestressing Steel and Applicable ASTM Standards

Table 6.9.11 shows the types of high tensile prestressing steel and cold drawn wires used for prestressing, with the ASTM Designation standard specifications.

Table 6.9.11: List of Standards for the Prestressing Steel

Material	Designation of the Standard	Title of the Standard
Prestressing Steel	A416/A416M	Standard Specification for Steel Strand, Uncoated Seven-Wire for Prestressed Concrete
	A421/A421M	Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete
	ASTM A648	Standard specification for steel, wire, hard drawn for prestressing concrete pipe
	A722/A722M	Standard Specification for Uncoated High-Strength Steel Bars for Prestressing Concrete

9.20 Construction of Prestressed Concrete Structures

9.20.1 Corrosion Protection for Unbonded Tendons

9.20.1.1 Unbonded prestressing steel shall be encased with sheathing. The prestressing steel shall be completely coated and the sheathing around the prestressing steel filled with suitable material to inhibit corrosion.

9.20.1.2 Sheathing shall be watertight and continuous over entire length to be unbonded.

9.20.1.3 For applications in corrosive environments, the sheathing shall be connected to all stressing, intermediate and fixed anchorages in a water tight fashion.

9.20.1.4 Unbonded single-strand tendons shall be protected against corrosion in accordance with ACI 423.7.

9.20.2 Post-tensioning Ducts

9.20.2.1 Ducts for grouted tendons shall be mortar- tight and nonreactive with concrete, prestressing steel, grout, and corrosion inhibitor.

9.20.2.2 Ducts for grouted single-wire, single-strand, or single-bar tendons shall have an inside diameter at least 6 mm larger than the prestressing steel diameter.

9.20.2.3 Ducts for grouted multiple wire, multiple strand, or multiple bar tendons shall have an inside cross-sectional area at least two times the cross-sectional area of the prestressing steel.

9.20.2.4 Ducts shall be maintained free of ponded water if members to be grouted are exposed to temperatures below freezing prior to grouting.

9.20.3 Grout for Bonded Tendons

9.20.3.1 Grout shall consist of Portland cement and water; or Portland cement, sand, and water.

9.20.3.2 Materials for grout shall conform to Sections 9.20.3.3 to 9.20.3.5.

9.20.3.3 Portland cement shall conform to Sec 9.19.1.

9.20.3.4 Water shall conform to Sec 9.19.1.

9.20.3.5 Sand, if used, shall conform to Sec 9.19.1 except that gradation shall be permitted to be modified as necessary to obtain satisfactory workability.

9.20.3.6 Admixtures conforming to Sec 9.19.1 and known to have no injurious effects on grout, steel, or concrete shall be permitted. Calcium chloride shall not be used.

9.20.4 Selection of Grout Proportions

9.20.4.1 Proportions of materials for grout shall be based on either (a) or (b) below.

(a) Results of tests on fresh and hardened grout prior to beginning grouting operations; or

(b) Prior documented experience with similar materials and equipment and under comparable field conditions.

9.20.4.2 Cement used in the Work shall correspond to that on which selection of grout proportions was based.

9.20.4.3 Water content shall be minimum necessary for proper pumping of grout; however, water-cement ratio shall not exceed 0.45 by weight.

9.20.4.4 Water shall not be added to increase grout flowability that has been decreased by delayed use of the grout.

9.20.5 Mixing and Pumping of Grout

9.20.5.1 Grout shall be mixed in equipment capable of continuous mechanical mixing and agitation that will produce uniform distribution of materials, passed through screens, and pumped in a manner that will completely fill the ducts.

9.20.5.2 Temperature of members at time of grouting shall be above 2°C and shall be maintained above 2°C until field-cured 50 mm cubes of grout reach a minimum compressive strength of 5.5 N/mm^2.

9.20.5.3 Grout temperatures shall not be above 32°C during mixing and pumping.

9.20.6 Protection for Prestressing Steel During Welding

Burning or welding operations in the vicinity of prestressing steel shall be performed so that prestressing steel is not subject to excessive temperatures, welding sparks, or ground currents.

9.20.7 Application and Measurement of Prestressing Force

9.20.7.1 Prestressing force shall be determined by both of (a) and (b):

(a) Measurement of steel elongation. Required elongation shall be determined from average load-elongation curves for the prestressing steel used;

(b) Observation of jacking force on a calibrated gage or load cell or by use of a calibrated dynamometer.

Cause of any difference in force determination between (a) and (b) that exceeds 5 percent for pretensioned elements or 7 percent for post-tensioned construction shall be ascertained and corrected.

9.20.7.2 Where the transfer of force from the bulk- heads of pretensioning bed to the concrete is accomplished by flame cutting prestressing steel, cutting points and cutting sequence shall be predetermined to avoid undesired temporary stresses.

9.20.7.3 Long lengths of exposed pretensioned strand shall be cut near the member to minimize shock to concrete.

9.20.7.4 Total loss of prestress due to unreplaced broken prestressing steel shall not exceed 2 percent of total prestress.

9.20.8 Post-tensioning Anchorages and Couplers

9.20.8.1 Anchorages and couplers for bonded and unbonded tendons shall develop at least 95 percent of the f_{pu} when tested in an unbonded condition, without exceeding anticipated set. For bonded tendons, anchorages and couplers shall be located so that 100 percent of f_{pu} shall be developed at critical sections after the prestressing steel is bonded in the member.

9.20.8.2 Couplers shall be placed in areas approved by the licensed design professional and enclosed in housing long enough to permit necessary movements.

9.20.8.3 In unbonded construction subject to repetitive loads, attention shall be given to the possibility of fatigue in anchorages and couplers.

9.20.8.4 Anchorages, couplers, and end fittings shall be permanently protected against corrosion.

9.21 Performance Requirement of Material

9.21.1 The fundamental performance requirement of materials forming the structure is that they should be able to resist actions such as the various loadings to which the structure is exposed.

9.21.2 Materials forming the structure should not reach unexpected limit states as a result of deterioration phenomena during the working life of the structure.

9.21.3 Materials-related energy consumption and CO_2 discharges should be minimized, while recyclability should be high.

Any materials that escape into the surrounding environment during construction and service should not have a strong impact on human beings, animals and plants.

Commentary:

Corresponding to design requirements, the materials should be evaluated to ensure that their properties are suitable with respect to strength (tensile, compressive and shear), deformation (e.g. elastic modulus), heat resistance and water tightness.

The characteristic values obtained from the tests, complying appropriate BDS, ASTM, BS, or equivalent standards, on such specimens should be converted to suit the design calculation models using appropriate conversion factors or functions. The characteristic value of material strength f_k is calculated from test results using Eq. 6.9.15.

$$f_k = f_m - k\sigma \qquad (6.9.15)$$

Where, f_m: mean of test values, σ : standard deviation of test values, and k: coefficient of variance. The coefficient k is determined from the probability of obtaining a test value less than the characteristic value and the probability distribution of test results. The 5% fractile value is often taken as the characteristic value. In this case, the value of k is 1.64 if the normal distribution is assumed for the test values.

At the structural design stage, verification shall be performed so that response value is less than or equal to the limit value of performance throughout both construction period and working life. At the end of construction stage, just completed structure shall fulfill the all required performances considered in its design.

Division C : Maintenance (Sections 9.22 To 9.27)

9.22 General

If the prestressed concrete structure is designed and constructed in accordance with the appropriate concepts described in Part I and II of this Chapter, based on which the durability is checked by verifying the performance requirements of the concrete and its constituent materials, it is not likely that structural deterioration would become so significant as to degrade the performance of the structure. On the other hand it is not easy to estimate the performance degradation process of the structure during its service life accurately. Also, it is difficult to completely avoid construction defects at all construction stages. Therefore, the new structure should be appropriately maintained by routine and regular inspections, based on an adequate maintenance plan formulated at the design stage.

For existing structures, deterioration may be evident in some cases, with the performance having been degraded. The defects of such structures should be accurately assessed and identified as initial defects, damage, or deteriorations. Major causes for such defects should be identified subsequently so that appropriate remedial actions can be selected. The initial defects and damage should be treated promptly and appropriately including emergency treatments. When the deterioration that would degrade the performance is evident, the deterioration mechanisms should be identified and appropriate maintenance, carried out based on the results of deterioration prediction and performance degradation evaluation.

9.23 Classification of Maintenance Action

Maintenance actions shall be classified into different categories depending on such factors as the importance of the structure, design life, impact on a third party, environmental conditions, ease of maintenance, and cost.

In the view of the above, four categories are recommended for the classifications of the maintenance actions:

9.23.1 Category A : Preventive Maintenance

Maintenance to prevent deterioration which would otherwise lead to unsatisfactory structural performance. Category A structures are those

- for which remedial actions are difficult to take after deterioration becomes apparent;
- of which deterioration must not be apparent;
- having a long design life.

Structures in this category generally have a high degree of importance which in many cases require monitoring.

9.23.2 Category B : Corrective Maintenance

Maintenance to restore the performance level and/or to reduce the rate of deterioration so as to maintain satisfactory structural performance. Category B structures are those for which

- remedial measures can be taken after deterioration becomes apparent;
- apparent deterioration causes no appreciable inconvenience.

9.23.3 Category C : Observational Maintenance

Maintenance in which visual inspection is necessary without any remedial action regardless of the deterioration level. Category C structures are those

- for use as long as they are usable;
- for which ensuring safety from threats posed to third parties is the only requirement.

9.23.4 Category D : Indirect Maintenance

Maintenance in which no direct inspection is necessary or possible. Category D structures are those for which direct inspection is extremely difficult. For these reasons, non-inspection maintenance after the initial inspection is carried out not as routine or regular inspection, but as extraordinary inspection following natural disasters, accidents, etc.

9.24 Maintenance Record

Records, drawings and related documents prepared during the time of planning, design and construction shall be referred to and made use of while developing an appropriate methodology for maintenance covering inspection and repairs.

Commentary:

A thorough study of the planning, design and construction related documents often provide insights into the inherent weaknesses of the structure which in turn often serve as pointers for further detailed inspection and/or repairs.

Furthermore, a clear record should be kept of the difficulties encountered, remedial actions taken and any deviation from the design drawings. These record also serve as a valuable reference in the design and construction of similar structures and their subsequent inspections.

9.25 Inspection

9.25.1 General

On the basis of the methods used in the frequency and timing, inspection shall be classified as initial inspection, routine inspection, regular inspection, detailed inspection, extraordinary inspection, and monitoring.

9.25.2 Initial Inspection

Initial inspection is intended to examine whether the structure is adequately constructed. It also allows the collection of basic data for initiating a maintenance program. Initial inspection shall also be carried out just after the completion of remedial actions.

Initial inspection should cover the external appearance of the structure, variation of concrete quality, existence of construction defects, construction errors on reinforcing and pretsressing bar arrangement, and so on.

9.25.3 Routine Inspections

It shall be carried out on a routine basis at certain intervals without making any specific effort to identify signs of deterioration, if any, and the time of their first appearance. The exact tools to be used and the frequency of such inspections may be decided on the basis of such factors as the likely mechanisms of such deterioration, environmental conditions, importance of the structure, and the maintenance action classification.

A routine inspection should cover the external appearance of the structure including cracks, spalling, delamination, color changes, rust stain from reinforcement, and isolation of free lime from concrete.

9.25.4 Regular Inspection

It shall be carried out at regular intervals using appropriate tools to identify signs of deterioration and the time of their first appearance. Efforts shall be made during a regular inspection to observe the structure closely to obtain details which will be difficult to gather during a routine inspection.

Visual inspection and/or hammering inspection are carried out mainly to obtain more details on the items inspected in a routine inspection. In addition, inspections by using appropriate non-destructive tests or taking concrete cores etc. can be effectively combined with the visual inspection.

9.25.5 Detailed Inspection

Detailed inspection shall be done when

(a) some signs of deterioration or a change in the performance level are observed during a routine and/or regular inspection;

(b) it is difficult to obtain reliable and accurate information during a routine and/or regular inspection;

(c) it is found that the structural integrity of the structure has been adversely affected by the extent of the deterioration;

(d) more detailed information is required before deciding on the necessity and scope for undertaking a major repair, rehabilitation or strengthening work.

9.25.6 Extraordinary Inspection

It shall be carried out after a structure has been subjected to an accidental load to assess the extent of the damage and the need for remedial actions. Such accidental loads may include those caused by an earthquake, storm, flood, fire, explosion, etc.

9.26 Monitoring

The deterioration and/or performance of the concerned structure as determined in 9.6.2, shall be monitored, through continuous recording of the appropriate data, together with routine and regular inspections, so that the appropriate remedial actions can be taken before the deterioration becomes detrimental to the appearance and other performance of the structure.

9.26.1 Deterioration Mechanism and Prediction

9.26.1.1 General

The prevailing state of the concerned structure shall be evaluated as properly as possible according to the inspection results, design and construction records, environmental conditions, and any other relevant information. Then when any deterioration is found, the possible causes of the deterioration and the corresponding mechanism can be appropriately estimated.

9.26.1.2 Identification of deterioration mechanisms

Deterioration of a structure is caused by the environmental actions and loading conditions. Environment-oriented deterioration includes carbonation-induced deterioration, chloride-induced deterioration, chemical attack, alkali-aggregate reaction, etc. On the other hand external force-oriented deterioration includes fatigue, excessive loading, and differential settlement of the support.

9.26.1.3 Deterioration factors

Deterioration factors may be classified into those

(a) external to structures such as temperature, humidity and any other environmental characteristics; and

(b) internal to the structure such as design parameters and quality control during construction.

Commentary:

Design factors include the geometry of the members/ segments, crack width specifications, concrete cover to reinforcing bar and prestressing steel/ducts, and design strength. Construction factors include material selection, mix proportions, transportation, placement, and curing methods.

9.26.1.4 Determination of deterioration levels and rates

The level of deterioration and/or performance shall be determined based on the results of inspections and simulations using appropriate models for the mechanisms of deterioration.

The following features appearing on the surface of the structure may be used for evaluating the degree of deterioration and the level of performance:

(a) crack pattern, length and width;

(b) the extent of delamination, peeling and spalling of concrete cover, and scaling and degradation areas;

(c) abnormal hammer tapping sound and the extent of abnormality;

(d) presence and degree of exudation of rust and efflorescence and water leakage.

9.26.2 Evaluation and Decision Making

9.26.2.1 General

In general, the deterioration and performance degradation of a structure progress monotonically. The decision, therefore, should be made based on the evaluation outcome of the performance of the structure at the time of inspection and at the end of its design life.

9.26.2.2 Threshold level

The threshold level of the structure's degraded performance shall be specified in accordance with the requirements of safety, functionality, appearance, societal friendliness and such other factors, taking into consideration the type, importance and maintenance level of the structure and the environmental conditions.

9.26.2.3 Evaluation of inspection results

The results from routine and regular inspections shall be evaluated and a decision shall be made whether a detailed inspection is required or otherwise.

The results from the detailed and/or extraordinary inspections shall be evaluated and a decision shall be made whether a remedial action is required or otherwise.

Immediate remedial actions shall be taken in cases where deterioration, damage and/or initial defects are found to be hazardous to third parties.

9.27 Remedial Action

9.27.1 General

A remedial action on a deteriorated structure shall be taken on the basis of the inspection results, importance of the structure, maintenance classification, and the threshold level of deterioration and/or performance.

Commentary:

Repair and strengthening are the main techniques of remedial actions of which details are described in Sections 9.2.7.3 and 9.2.7.4 respectively. The following measures are also included in the remedial actions.

Intensified inspection: inspection may be carried out by suitably increasing one or more of the following: frequency of inspection, number of inspection items, and the locations for inspection.

Usage restriction: suitable restriction shall be imposed on the maximum live load that the structure may carry, depending on the level of deterioration observed.

Functional improvement or restoration: this may include an appearance improvement that beautifies a structure with suitably painting or placing additional concrete, and so on.

Dismantling and removal: in a case when the deterioration of a structure is too severe for its structural performance to be sufficiently restored, and dismantling or the removal is one of the choices as the remedial measures.

Special care for emergency: when a deteriorated structure poses an immediate threat to the environment, its users, or third parties, suitable emergency action shall be taken immediately.

9.27.2 Selection of Remedial Action

Selection of methods and materials suitable for the relevant deterioration mechanism and degree of performance degradation is particularly important for measures for which wide varieties of methods and materials are available. Care should be taken as the method of restoring the performance may vary depending on the deterioration mechanism, even if the level of performance is the same.

9.27.3 Repair

9.27.3.1 General

Repair of a structure refers to the remedial action taken to prevent or slow down its further deterioration and reduce the possibility of damage to its users or third parties.

Types of repair include (i) repair of defects such as cracking and peeling; (ii) removal of concrete damaged by deterioration due to carbonation and such like; (iii) surface coating to prevent re-intrusion of hazardous substances.

9.27.3.2 Preparation and execution

A complete plan for the repair work including methods of repair, materials to be used, and tests to ensure the quality of work, shall be developed before the repair work commences.

Repair works shall be carried out with minimum disturbances to the surrounding environment. Necessary tests to ensure the quality of the repair work shall be carried out. Detailed record of the repair work shall be maintained for future reference.

9.27.3.3 Methods and materials

Some current repair methods and associated materials are

- crack repair by injecting epoxy;
- section repair including patching using polymer cement mortar;
- surface protection by resin or mortar;
- cathodic protection;
- re-alkalization;
- de-salination, wherever required.

Commentary:

Development of a repair plan comprises the selection of a repair method suitable for the deterioration mechanism, establishment of the required repair level, and decisions on the repair policy, specifications for the repair materials, sectional dimensions after repair, and execution methods.

9.27.4 Strengthening

9.27.4.1 General

Strengthening of a structure refers to the remedial action taken to restore or improve its structural properties including load carrying capacity and stiffness, to a level which is equal to or higher than that of the original design.

Commentary:

Strengthening methods include (i) replacement of members; (ii) an increase in the cross-sectional area of concrete; (iii) addition of members; (iv) an increase of the support points; (v) addition of strengthening members; (vi) external prestressing, etc.

9.27.4.2 Preparation and execution

Strengthening of a structure shall be preceded by a thorough investigation of its deterioration considering such factors as the remaining design life, deterioration mechanism, possible causes and extent of deterioration, the remaining and desired load-carrying capacity or stiffness, importance of the structure, maintenance classification, and any remedial actions taken previously.

A complete plan for the strengthening work including design calculations, methods of strengthening, materials to be used, and tests to ensure quality of the work, shall be developed before work commences.

Strengthening work shall be carried out with minimum disturbance to the surrounding environment and the service condition of the structure.

9.27.4.3 Methods and materials

Some current methods and associated materials for strengthening are

- external bonding viz plate or sheet bonding and over or under-laying using steel or carbon sheets;
- external prestressing using additional tension cables;
- addition of girders, braces and/or supports;
- replacement of members;
- seismic isolation.

Commentary:

When selecting a strengthening method, it is necessary to consider effects of strengthening, constructability, cost-effectiveness, and impact on the community/ environment during execution. It is also important to consider the ease of maintenance after strengthening and any influence on the landscape.

9.27.5 Record

9.27.5.1 General

Records shall be kept and preserved for future reference. Such records shall include details concerning the design, inspection and evaluation procedures, plans and execution of any repair and/or strengthening work undertaken, and other such information.

9.27.5.2 Preservation

The maintenance records of a structure shall be preserved while the structure remains in service. It is also desirable that such records be preserved for an indefinite period as a useful reference for the construction and maintenance of other similar structures.

Commentary:

It is important to devise a format that makes it easy to understand the history of a structure by simply referring to records. The records should be made accessible at all times.

9.27.5.3 Method and item of recording

Records shall be kept in an easy-to-understand format.

The items to be recorded shall include references to concerned agencies, drawings, immediate and nearby environment, classification of structure, results of deterioration rate estimation, results of any inspections carried out, evaluation of the structure, and details of the plan and actual execution of remedial and other actions.

PART VI
Chapter 10
Steel Structures

10.1 General Provisions for Structural Steel Buildings and Structures

This Section states the scope of the Specification, summarizes referenced Specification, code, and standard documents, and provide requirements for materials and contract documents.

10.1.1 Scope

The specification contained in Chapter 10 Part 6 of this Code sets forth criteria for the design, fabrication, and erection of structural steel buildings and other structures, where other steel-structures are defined as those structures designed, fabricated, and erected in a manner similar to steel-buildings, with building-like vertical and lateral load resisting elements. Where conditions are not covered by this specification, designs are permitted to be based on tests or analysis, subject to the approval of the authority having jurisdiction. Alternate methods of analysis and design shall be permitted, provided such alternate methods or criteria are acceptable to the authority having jurisdiction.

10.1.1.1 Low-seismic applications

When the seismic response modification coefficient, R (as specified in Chapter 2 Part 6) is taken equal to or less than 3, the design, fabrication, and erection of structural-steel-framed buildings and other steel-structures shall comply with this specification except that such structures need not to comply with the specifications set forth in Sec 10.20 Seismic Provisions.

10.1.1.2 High-seismic applications

When the seismic response modification coefficient, R (as specified in Chapter 2 Part 6) is taken greater than 3, the design, fabrication and erection of structural-steel-framed buildings and other structures shall comply with the requirements in the Sec 10.20 Seismic Provisions, in addition to the provisions of other sections (whichever applicable) this specification.

10.1.2 Symbols, Glossary and Referenced Specifications, Codes and Standards

10.1.2.1 Symbols

The Section or Table number in the right-hand column refers to where the symbol is first used.

Symbol	Meaning	Section
A	Column cross-sectional area, mm^2	10.10.10.6
A	Total cross-sectional area of member, mm^2	10.5.7.2

Symbol	Meaning	Section
A_{BM}	Cross-sectional area of the base metal, mm^2	10.10.2.4
A_b	Nominal unthreaded body area of bolt or threaded part, mm^2	10.10.3.6
A_b	Cross-sectional area of a horizontal boundary element (HBE), mm^2	10.20.17.2.1
A_{bi}	Cross-sectional area of the overlapping branch, mm^2	10.11.2.3
A_{bj}	Cross-sectional area of the overlapped branch, mm^2	10.11.2.3
A_c	Cross-sectional area of a vertical boundary element (VBE),mm^2	10.20.17.2.1
A_D	Area of an upset rod based on the major thread diameter, mm^2	Table 6.10.10
A_e	Effective net area, mm^2	10.4.2
A_{eff}	Summation of the effective areas of the cross section based on the reduced effective width, b_e, mm^2	10.5.7.2
A_f	Flange area, mm^2	10.20.8
A_{fc}	Area of compression flange	10.7.3.1
A_{fg}	Gross tension flange area, mm^2	10.6.13.1
A_{fn}	Net tension flange area, mm^2	10.6.13.1
A_{ft}	Area of tension flange, mm^2	10.7.3.1
A_g	Gross area of member, mm^2	10.2.3.13
A_g	Gross area of section based on design wall thickness, mm^2	10.7.6
A_g	Chord gross area, mm^2	10.11.2.2
A_g	Gross area, mm^2	10.20.9
A_{gv}	Gross area subject to shear, mm^2	10.10.4.3
A_n	Net area of member, mm^2	10.2.3.13
A_{nt}	Net area subject to tension, mm^2	10.10.4.3
A_{nv}	Net area subject to shear, mm^2	10.10.4.2

Symbol	**Meaning**	**Section**
A_{pb}	Projected bearing area, mm^2	10.10.7
A_{sc}	Area of the yielding segment of steel core, mm^2	10.20.16
A_{sf}	Shear area on the failure path, mm^2	10.4.5.1
A_{st}	Stiffener area, mm^2	10.7.3.3
A_{st}	Area of link stiffener, mm^2	10.20.15
A_t	Net tensile area, mm^2	10.17.4
A_w	Web area, the overall depth times the web thickness, dt_w, mm^2	10.7.2.1
A_w	Effective area of the weld, mm^2	10.10.2.4
A_w	Link web area, mm^2	10.20.15
A_{wi}	Effective area of weld throat of any i^{th} weld element, mm^2	10.10.2.4
A_1	Area of steel concentrically bearing on a concrete support, mm^2	10.10.8
A_2	Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, mm^2.	10.10.8
B	Factor for lateral-torsional buckling in tees and double angles	10.6.9.2
B	Overall width of rectangular HSS member, measured 90° to the plane of connection, mm.	10.11.1.1, Table 6.10.2
B	Overall width of rectangular HSS main member, measured 90° to the plane of the connection,mm.	10.11.2.1, 10.11.3.1
B_b	Overall width of rectangular HSS branch member, measured 90° to the plane of the connection, mm.	10.11.2.1, 10.11.3.1
B_{bi}	Overall branch width of the overlapping branch	10.11.2.3
B_{bj}	Overall branch width of the overlapped branch	10.11.2.3
B_p	Width of plate, measure 90° to the plane of the connection, mm.	10.11.1.1
B_p	Width of plate, transverse to the axis of the main member,mm.	10.11.2.3

Symbol	Meaning	Section
B_1,B_2	Factors used in determining M_u for combined bending and axial forces when first-order analysis is employed	10.3.2.1
C	HSS torsional constant	10.8.3.1
C_a	Ratio of required strength to available strength	Table 6.10.8
C_b	Lateral-torsional buckling modification factor for nonuniform moment diagrams when both ends of the unsupported segment are braced	10.6.1
C_d	Coefficient relating relative brace stiffness and curvature	10.19.3.1, 10.20.9
C_d	Deflection amplification factor	P.2
C_f	Constant based on stress category, given in Table 6.10.14	10.17.3
C_m	Coefficient assuming no lateral translation of the frame	10.3.2.1
C_p	Ponding flexibility coefficient for primary member in a flat roof	10.16.1
C_r	Coefficient for web sidesway buckling	10.10.10.4
C_r	Parameter used for determining the approximate fundamental period	P.2
C_s	Ponding flexibility coefficient for secondary member in a flat roof	10.16.1
C_v	Web shear coefficient	10.7.2.1
C_w	Warping constant, mm^6	10.5.4
D	Nominal dead load	10.16.2
D	Dead load due to the weight of the structural elements and permanent features on the building, N.	10.20.9
D	Outside diameter of round HSS, mm.	Table 6.10.8
D	Outside diameter of round HSS member, mm.	Table 6.10.1
D	Outside diameter of round HSS member, mm.	10.11.1.1
D	Outside diameter of round HSS main member, mm.	10.11.2.1
D	Outside diameter of round HSS main member, mm.	10.11.3.1
D	Outside diameter, mm.	10.5.7.2

Symbol	Meaning	Section
D	Chord diameter, mm.	10.11.2.2
D_b	Outside diameter of round HSS branch member,mm.	10.11.2.1
D_b	Outside diameter of round HSS branch member,mm.	10.11.3.1
D_s	Factor used in Eq. 6.10.144, dependent on the type of transverse stiffeners used in a plate girder	10.7.3.3
D_u	In slip-critical connections, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension	10.10.3.8
E	Earthquake load	10.20.4
E	Effect of horizontal and vertical earthquake-induced loads	10.20.9
E	Modulus of elasticity of steel, E = 200,000 MPa	10.20.8
E	Eccentricity in a truss connection, positive being away from the branches, mm.	10.11.2.1
E	Modulus of elasticity of steel = 200000 MPa	Table 6.10.1
E_c	Modulus of elasticity of concrete, MPa.	10.18.2.3
E_{cm}	Modulus of elasticity of concrete at elevated temperature, MPa	
EI	Flexural elastic stiffness of the chord members of special segment, (N-mm^2)	10.20.12
E_m	Modulus of elasticity of steel at elevated temperature, MPa.	10.18.2.3
F_a	Available axial stress at the point of consideration, MPa.	10.8.2
F_{BM}	Nominal strength of the base metal per unit area, MPa.	10.10.2.4
F_{BW}	Available flexural stress at the point of consideration about the major axis, MPa.	10.8.2
F_{bz}	Available flexural stress at the point of consideration about the minor axis, MPa.	10.8.2
F_c	Available stress, MPa.	10.11.2.2
F_{cr}	Critical stress, MPa.	10.5.3
F_{cr}	Buckling stress for the section as determined by analysis, MPa.	10.6.12.2

Symbol	Meaning	Section
F_{cry}	Critical stress about the minor axis, MPa.	10.5.4
F_{crz}	Critical torsional buckling stress, MPa.	10.5.4
F_e	Elastic critical buckling stress, MPa.	10.3.1.3
F_{ex}	Elastic flexural buckling stress about the major axis, MPa.	10.5.4
F_{EXX}	Electrode classification number, MPa.	10.10.2.4
F_{ey}	Elastic flexural buckling stress about the minor axis, MPa.	10.5.4
F_{ez}	Elastic torsional buckling stress, MPa.	10.5.4
F_L	A calculated stress used in the calculation of nominal flexural strength, MPa.	Table 6.10.1
F_n	Nominal torsional strength	10.8.3.3
F_n	Nominal tensile stress F_{nt}, or shear stress, F_{nv}, from Table 6.10.10, MPa.	10.10.3.6
F_{nt}	Nominal tensile stress from Table 6.10.10, MPa.	10.10.3.7
F'_{nt}	Nominal tensile stress modified to include the effects of shearing stress, MPa.	10.10.3.7
F_{nv}	Nominal shear stress from Table 6.10.10, MPa.	10.10.3.7
F_{SR}	Design stress range, MPa.	10.17.3
F_{TH}	Threshold fatigue stress range, maximum stress range for indefinite design life from Table 6.10.14, MPa.	10.17.1
F_u	Specified minimum tensile strength of the type of steel being used, MPa.	10.4.2
F_u	Specified minimum tensile strength of the connected material, MPa.	10.10.3.10
F_u	Specified minimum tensile strength of HSS material, MPa.	10.11.1.1
F_u	Specified minimum tensile strength of HSS material, MPa.	10.11.2.1
F_u	Ultimate strength of HSS member, MPa.	10.11.3.1

Symbol	Meaning	Section
F_u	Specified minimum tensile strength, MPa.	10.20.6
F_{um}	Specified minimum tensile strength of the type of steel being used at elevated temperature, MPa.	10.18.2
F_w	Nominal strength of the weld metal per unit area, MPa.	10.10.2.4
F_{wi}	Nominal stress in any i^{th} weld element, MPa.	10.10.2.4
F_{wix}	x component of stress $F_{wi,}$ MPa.	10.10.2.4
F_{wiy}	y component of stress $F_{wi,}$ MPa.	10.10.2.4
F_y	Specified minimum yield stress of the type of steel being used, MPa. As used in this Specification, “yield stress” denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point	10.20.6, Table 6.10.1
F_y	Specified minimum yield stress of the compression flange, MPa.	10.15.3
F_y	Specified minimum yield stress of the column web, MPa.	10.10.10.6
F_y	Specified minimum yield stress of HSS member material, MPa.	10.11.1.1
F_y	Specified minimum yield stress of HSS main member material, MPa.	10.11.2.1
F_y	Specified minimum yield stress of HSS main member, MPa.	10.11.3.1
F_{yb}	Specified minimum yield stress of HSS branch member material, MPa.	10.11.2.1
F_{yb}	Specified minimum yield stress of HSS branch member, MPa.	10.11.3.1
F_{yb}	F_y of a beam, MPa.	10.20.9
F_{yc}	F_y of a column, MPa.	10.20.9
F_{ybi}	Specified minimum yield stress of the overlapping branch material, MPa.	10.11.2.3
F_{yf}	Specified minimum yield stress of the flange, MPa.	10.10.10.1
F_{ym}	Specified minimum yield stress of the type of steel used at elevated temperature, MPa.	10.18.2
F_{yp}	Specified minimum yield stress of plate, MPa.	10.11.1.1

Symbol	Meaning	Section
F_{ysc}	Specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, MPa.	10.20.16
F_{yst}	Specified minimum yield stress of the stiffener material, MPa.	10.7.3.3
F_{yw}	Specified minimum yield stress of the web, MPa.	10.10.10.2
G	Shear modulus of elasticity of steel = 77 200 MPa.	10.5.4
G	Gap between toes of branch members in a gapped K-connection, neglecting welds, mm.	10.11.2.1
H	Flexural constant	10.5.4
H	Overall height of rectangular HSS member, measured in the plane of connection, mm.	10.11.1.1
H	Overall height of rectangular HSS main member, measured in plane of connection, mm.	10.11.2.1
H	Overall height of rectangular HSS main member, measured in plane of connection, mm.	10.11.3.1
H	Overall height of rectangular HSS member, measured in the plane of connection, mm.	Table 6.10.2
H	The load length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width = N/B, where, $N = H_b/sin\theta$	10.11.2.1
H	Height of story, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between the top of floor slabs at each of the levels above and below, mm	10.20.8
H_b	Overall height of rectangular HSS branch member, measured in the plane of the connection, mm.	10.11.3.1
H_b	Overall height of rectangular HSS branch member, measured in the plane of the connection, mm.	10.11.2.1
H_{bi}	Overall depth of the overlapping branch	10.11.2.3
ΣH	Story shear produced by the lateral forces used to compute Δ_H, N.	10.3.2.1
I	Moment of inertia in the place of bending, mm^4.	10.3.2.1

Symbol	**Meaning**	**Section**
I	Moment of inertia about the axis of bending, mm^4.	10.14.3
I	Moment of inertia, mm^4	10.20.12
I_c	Moment of inertia of a vertical boundary element (VBE) taken perpendicular to the direction of the web plate line, mm^4	10.20.17
I_d	Moment of inertia of the steel deck supported on secondary members, mm^4	10.16.1
I_p	Moment of inertia of primary members, mm^4	10.16.1
I_s	Moment of inertia of secondary members, mm^4	10.16.1
I_x, I_y	Moment of inertia about the principal axes, mm^4	10.5.4
I_y	Out-of-plane moment of inertia, mm^4	10.19.2
I_z	Minor principal axis moment of inertia, mm^4	10.6.10.2
I_{yc}	Moment of inertia about y-axis referred to the compression flange, or if reverse curvature bending referred to smaller flange, mm^4	10.6.1
J	Torsional constant, mm^4	10.5.4
K	Effective length factor determined in accordance with Sec 10.3	10.3.1.2
K	Effective length factor for prismatic member	10.20.13
K_z	Effective length factor for torsional buckling	10.5.4
K_1	Effective length factor in plane of bending, calculated based on the assumption of no lateral translation set equal to 1.0 unless analysis indicates a smaller value to be used.	10.3.2.1
K_2	Effective length factor in the plane of bending, calculated based on a sidesway buckling analysis	10.3.2.1
L	Story height, mm.	10.3.2.1
L	Laterally unbraced length of a member, mm.	10.5.2
L	Length of member between work points at truss chord centerlines,mm.	10.5.5

Symbol	Meaning	Section
L	Length of the member, mm.	10.8.3
L	Actual length of end-loaded weld, mm.	10.10.2.2
L	Nominal occupancy live load	10.18.1.4
L	Span length, mm.	10.19.2
L	Span length of the truss, mm.	10.20.12
L	Distance between VBE centerlines, mm	10.20.17
L_b	Distance between braces, mm.	10.19.2
L_b	Length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, mm.	10.6.2, 10.20.13
L_c	Distance between plastic hinge locations, mm	10.20.9
L_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material,mm.	10.10.3.10
L_c	Link length, mm	10.20.15
L_{cf}	Clear distance between VBE flanges, mm	10.20.17
L_e	Total effective weld length of groove and fillet welds to rectangular HSS,mm.	10.11.2.3
L_p	Limiting laterally unbraced length for the limit state of yielding, mm.	10.6.2.2
L_p	Column spacing in direction of girder, m	10.16
L_p	Limiting laterally unbraced length for full plastic flexural strength, uniform moment case,mm.	10.20.12
L_{pd}	Limiting laterally unbraced length for plastic analysis, mm.	10.15.7
L_{pd}	Limiting laterally unbraced length for plastic analysis, mm	10.20.13
L_q	Maximum unbraced length forM_r(the required flexural strength),mm.	10.19.2
L_r	Limiting laterally unbraced length for limit state of inelastic lateral-torsional buckling, mm.	10.6.2.2

Symbol	Meaning	Section
L_s	Column spacing perpendicular to direction of girder, m	10.16.1
L_s	Length of the special segment, mm	10.20.12
L_v	Distance from maximum to zero shear force,mm.	10.7.6
M_A	Absolute value of moment at quarter point of the unbraced segment, N-mm.	10.6.1
M_a	Required flexural strength in chord, using ASD load combinations, N-mm.	10.11.2.2
M_a	Required flexural strength, using ASD load combinations, N-mm.	10.20.9
M_{av}	Additional moment due to shear amplification from the location of plastic hinge to the column centerline based on ASD load combinations, N-mm.	10.20.9
M_B	Absolute value of moment at centerline of the unbraced segment, N-mm.	10.6.1
M_{br}	Required bracing moment,N-mm.	10.19.2
M_C	Absolute value of moment at three-quarter point of unbraced segment, N-mm.	10.6.1
$M_{C(x,y)}$	Available flexural strength determined in accordance with Sec 10.6,N-mm.	10.8.1.1
M_{Cx}	Available flexural-torsional strength for strong axis flexure determined in accordance with Sec 10.6,N-mm.	10.8.1.3
M_e	Elastic lateral-torsional buckling moment, N-mm.	10.6.10.2
M_{lt}	First-order moment under LRFD or ASD load combinations caused by lateral translation of the frame only, N-mm.	10.3.2.1
M_{max}	Absolute value of maximum moment in the unbraced segment,N-mm.	10.6.1
M_n	Nominal flexural strength, N-mm.	10.6.1
M_n	Nominal flexural strength, N-mm.	10.20.11
M_{nc}	Nominal flexural strength of the chord member of special segment, N-mm.	10.20.12
M_{nt}	First-order moment using LRFD or ASD load combinations assuming there is no lateral translation of the frame, N-mm.	10.3.2.1

Symbol	**Meaning**	**Section**
M_p	Plastic bending moment,N-mm.	Table 6.10.1
M_p	Nominal plastic flexural strength, N-mm.	Table 6.10.8
M_{pa}	Nominal plastic flexural strength modified by axial load, N-mm.	10.20.15
M_{pb}	Nominal plastic flexural strength of the beam, N-mm.	10.20.9
M_{pc}	Nominal plastic flexural strength of the column, N-mm.	10.20.8
$M_{p,exp}$	Expected plastic moment, N-mm.	10.20.9
M_r	Required second-order flexural strength under LRFD or ASD load combinations, N-mm.	10.3.2.1
M_r	Required flexural strength using LRFD or ASD load combinations,N-mm.	10.8.1
M_r	Required flexural strength in chord, N-mm.	10.11.2.2
M_r	Expected flexural strength, N-mm.	10.20.9
M_{r-ip}	Required in-plane flexural strength in branch,N-mm.	10.11.3.2
M_{r-op}	Required out-of-plane flexural strength in branch,N-mm.	10.11.3.2
M_u	Required flexural strength, using LRFD load combinations, N-mm.	10.20.9
M_u	Required flexural strength in chord, using LRFD load combinations,N-mm.	10.11.2.2
M_{uv}	Additional moment due to shear amplification from the location of plastic hinge to the column centerline based on LRFD load combinations, N-mm.	10.20.9
$M_{u,exp}$	Expected required flexural strength, N-mm.	10.20.15
M_y	Yield moment about the axis of bending, N-mm.	Table 6.10.1
M_1	Smaller moment, calculated from a first-order analysis, at the ends of that portion of the member unbraced in the plane of bending under consideration, N-mm.	10.3.2.1
M_2	Larger moment, calculated from a first-order analysis, at the ends of that portion of the member unbraced in the plane of bending under consideration, N-mm.	10.3.2.1

Symbol	Meaning	Section
N	Length of bearing (not less than k for end beam reactions),mm.	10.10.10.2
N	Bearing length of the load, measured parallel to the axis of the HSS member, (or measured across the width of the HSS in the case of the loaded cap plates), mm.	10.11.1.1
N	Number of stress range fluctuations in design life	10.17.3
N_b	Number of bolts carrying the Applied tension	10.10.3.9
N_i	Additional lateral load	10.3.2.2
N_i	Notional lateral load Applied at level i, N.	10.14.3
N_s	Number of slip planes	10.10.3.8
O_v	Overlap connection coefficient	10.11.2.2
P	Pitch, mm per thread	10.17.4
P_a	Required axial strength of a column using ASD load combinations, N.	10.20.8
P_{ac}	Required compressive strength using ASD load combinations, N.	10.20.9
P_b	Required strength of lateral brace at ends of the link, N.	10.20.15
P_{br}	Required brace strength, N.	10.19.2
P_c	Available axial compressive strength, N.	10.8.1.1
P_c	Available tensile strength, N.	10.8.1.2
P_c	Available axial strength of a column, N.	10.20.9
P_{co}	Available compressive strength out of the plane of bending, N.	10.8.1.3
P_{e1},P_{e2}	Elastic critical buckling load for braced and unbraced frame, respectively, N.	10.3.2.1
P_{eL}	Euler buckling load, evaluated in the plane of bending, N.	10.14.3

Symbol	**Meaning**	**Section**
$P_{l(t,c)}$	First-order axial force using LRFD or ASD load combinations as a result of lateral translation of the frame only (tension or compression, N.	10.3.2.1
$P_{n(t,c)}$	First-order axial force using LRFD or ASD load combinations, assuming there is no lateral translation of the frame (tension or compression, N.	10.3.2.1
P_n	Nominal axial strength, N.	10.4.2
P_n	Nominal axial strength of a column, N.	10.20.8
P_{nc}	Nominal axial compressive strength of diagonal members of the special segment, N.	10.20.12
P_{nt}	Nominal axial tensile strength of diagonal members of special segment, N.	10.20.12
P_{rc}	Required compressive strength using ASD or LRFD load combinations, N.	10.20.9
P_r	Required second-order axial strength using LRFD or ASD load combinations, N.	10.3.2.1
P_r	Required axial compressive strength using LRFD or ASD load combinations, N.	10.3.2.2
P_r	Required tensile strength using LRFD or ASD load combinations, N.	10.8.1.2
P_r	Required strength, N.	10.10.10.6
P_r	Required axial strength in chord, N.	10.11.2.2
P_r	Required axial strength in branch, N.	10.11.3.2
P_r	Required compressive strength, N.	10.20.15
P_u	Required axial strength in compression, N.	10.15.4
P_u	Required axial strength of a column or a link in LRFD load combinations, N.	10.20.8
P_{uc}	Required compressive strength using LRFD load combinations, N.	10.20.9
P_y	Member yield strength, N.	10.3.2.2
P_y	Nominal axial yield strength of a member, equal to $F_y A_g$,N.	Table 6.10.8

Symbol	Meaning	Section
P_{ysc}	Axial yield strength of steel core, N.	10.20.16
Q	Full reduction factor for slender compression elements	10.5.7
Q_a	Reduction factor for slender stiffened compression elements	10.5.7.2
Q_b	Maximum unbalanced vertical load effect applied to a beam by the braces, N.	10.20.13
Q_1	Axial forces and moments generated by at least 1.25 times the expected nominal shear strength of the link	10.20.15
Q_f	Chord-stress interaction parameter	10.11.2.2
Q_s	Reduction factor for slender unstiffened compression elements	10.5.7.1
R	Seismic response modification coefficient	10.20.1
R	Seismic response modification coefficient	10.1.1.1
R	Nominal load due to rainwater or snow, exclusive of the ponding contribution, MPa.	10.16.2
R_a	Required strength (ASD)	10.2.3.4
R_{FIL}	Reduction factor for joints using a pair of transverse fillet welds only	10.17.3
R_m	Factor in Eq. 6.10.8 dependent on type of system	10.3.2.1
R_m	Cross-section monosymmetry parameter	10.6.1
R_n	Nominal strength, N.	10.2.3.3
R_n	Nominal strength, N.	10.20.6
R_n	Nominal slip resistance, N.	10.10.3.8
R_{pc}	Web plastification factor	10.6.4.1
R_{PJP}	Reduction factor for reinforced or unreinforced transverse partial-joint-penetration (PJP) groove welds	10.17.3
R_{pt}	Web plastification factor corresponding to the tension flange yielding limit state	10.6.4.4

Symbol	**Meaning**	**Section**
R_t	Ratio of the expected tensile strength to the specified minimum tensile strength F_u, as related to overstrength in material yield stress R_y	10.20.6
R_u	Required strength (LRFD)	10.2.3.3
R_u	Required strength	10.20.9
R_v	Panel zone nominal shear strength	10.20.9
R_{wl}	Total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table 6.10.8	10.10.2.4
R_{wt}	Total nominal strength of transversely loaded fillet welds, as determined in accordance with Table 6.10.8 without the alternate in Sec 10.10.2.4 (a)	10.10.2.4
R_y	Ratio of the expected yield stress to the specified minimum yield stress, F_y	10.20.6
S	Elastic section modulus of round HSS, mm^3	10.6.8.2
S	Lowest elastic section modulus relative to the axis of bending, mm^3	10.6.12
S	Chord elastic section modulus, mm^3	10.11.2.2
S	Spacing of secondary members, m	10.16.1
S_c	Elastic section modulus to toe in compression relative to axis of bending, mm^3.	10.6.10.3
S_{eff}	Effective section modulus about major axis, mm^3	10.6.7.2
S_{xt}, S_{xc}	Elastic section modulus referred tension and compression flanges, respectively, mm^3	Table 6.10.1
S_x, S_y	Elastic section modulus taken about the principal axes, mm^3	10.6.2.2, F6
S_y	For channels, taken as the minimum section modulus	10.6.6
T	Nominal forces and deformations due to design-basis fire defined in Sec 4.2.1	10.18.1.4
T_a	Tension force due to ASD load combinations, kN.	10.10.3.9

Symbol	**Meaning**	**Section**
T_b	Minimum fastener tension given in Table 6.10.9, kN.	10.10.3.8
T_c	Available torsional strength, N-mm.	10.8.3.2
T_n	Nominal torsional strength, N-mm.	10.8.3.1
T_r	Required torsional strength, N-mm.	10.8.3.2
T_u	Tension force due to LRFD load combinations, kN.	10.10.3.9
U	Shear lag factor	10.4.3.3
U	Utilization ratio	10.11.2.2
U_{bs}	Reduction coefficient, used in calculating block shear rupture strength	10.10.4.3
U_p	Stress index	10.16.2
U_s	Stress index	10.16.2
V_a	Required shear strength using ASD load combinations, N.	10.20.9
V_c	Available shear strength, N.	10.7.3.3
V_n	Nominal shear strength, N.	10.7.1
V_n	Nominal shear strength of a member, N.	10.20.15
V_p	Nominal shear strength of an active link, N.	Table 6.10.8
V_{pa}	Nominal shear strength of an active link modified by axial load magnitude, N.	10.20.15
V_{ne}	Expected vertical shear strength of the special segment, N.	10.20.12
V_r	Required shear strength at the location of the stiffener, N.	10.7.3.3
V_r	Required shear strength using LRFD or ASD load combinations, N.	10.8.3.2
V_u	Required shear strength using LRFD load combinations, N.	10.20.10
Y_i	Gravity load from the LRFD load combination or 1.6 times the ASD load combination Applied at level i, N.	10.3.2.2
Y_t	Hole reduction coefficient, N.	10.6.13.1

Symbol	Meaning	Section
Z	Plastic section modulus about the axis of bending, mm^3	10.6.7.1
Z	Plastic section modulus of a member, mm^3.	10.20.9
Z_b	Branch plastic section modulus about the correct axis of bending, mm^3	10.11.3.3
Z_b	Plastic section modulus of the beam, mm^3.	10.20.9
Z_c	Plastic section modulus of the column, mm^3.	10.20.9
Z_x	Plastic section modulus x-axis, mm^3.	10.20.8
Z_{RBS}	Minimum plastic section modulus at the reduced beam section, mm^3.	10.20.9
$Z_{x,y}$	Plastic section modulus about the principal axes, mm^3	10.6.2, F6.1
a	Shortest distance from edge of pin hole to edge of member measured parallel to direction of force, mm.	10.4.5.1
a	Distance between connectors in a built-up member, mm.	10.5.6.1
a	Clear distance between transverse stiffeners,mm.	10.6.13.2
a	Half the length of the non-welded root face in the direction of the thickness of the tension-loaded plate, mm.	10.17.3
a	Angle that diagonal members make with the horizontal	10.20.12
a_w	Ratio of two times the web area in compression due to Application of major axis bending moment alone to the area of the compression flange components	10.6.4.2
b	Width of unstiffened compression element; for flanges of I-shaped members and tees, the width b is half the full-flange width, b_f; for legs of angles and flanges of channels and zees, the width b is the full nominal dimension; for plates, the width b is the distance from free edge to the first row of fasteners or line of welds, or the distance between adjacent lines of fasteners or lines of welds; for rectangular HSS, width b is the clear distance between the webs less the inside corner radius on each side, mm.	10.2.4.1, 10.2.4.2

Symbol	Meaning	Section
b	Full width of longest angle leg, mm.	10.5.7.1
b	Outside width of leg in compression, mm.	10.6.10.3
b	Width of the angle leg resisting the shear force, mm.	10.7.4
b	Width of compression element as defined in Specification Sec 10.2.4.1, mm.	Table 6.10.8
b_{cf}	Width of column flange, mm.	10.10.10.6
b_{cf}	Width of column flange, mm.	10.20.9
b_e	Reduced effective width,mm.	10.5.7.2
b_{eff}	Effective edge distance; the distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force,mm.	10.4.5.1
b_{eoi}	Effective width of the branch face welded to the chord	10.11.2.3
b_{eov}	Effective width of the branch face welded to the overlapped brace.	10.11.2.3
b_f	Flange width, mm.	10.2.4.1
b_f	Flange width, mm.	10.20.9
b_{fc}	Compression flange width, mm.	10.6.4.2
b_{ft}	Width of tension flange, mm.	10.7.3.1
b_l	Longer leg of angle, mm.	10.5.5
b_s	Shorter leg of angle, mm.	10.5.5
b_s	Stiffener width for one-sided stiffeners, mm.	10.19.2
d	Full nominal depth of section, mm.	10.2.4.1
d	Pin diameter,mm.	10.4.5.1
d	Full nominal depth of tee, mm.	10.5.7.1
d	Depth of rectangular bar, mm.	10.6.11.2

Symbol	Meaning	Section
d	Nominal fastener diameter, mm.	10.10.3.3
d	Diameter, mm.	10.10.7
d	Roller diameter,mm.	10.10.7
d	Nominal fastener diameter, mm.	10.20.7
d	Overall beam depth, mm.	10.20.15
d_b	Beam depth,mm.	10.10.10.6
d_b	Nominal diameter (body or shank diameter), mm.	10.17.4
d_c	Column depth, mm.	10.10.10.6
d_c	Overall column depth, mm.	10.20.9
d_z	Overall panel zone depth between continuity plates, mm.	10.20.9
e	Eccentricity in a truss connection, positive being away from thebranches, mm.	10.11.2.1
e	EBF link length, mm.	10.20.15
f_a	Required axial stress at point of consideration of LRFD or ASD load combinations, MPa.	10.8.2
$f_{b(w,z)}$	Required flexural stress at the point of consideration (major axis, minor axis) using LRFD or ASD load combinations, MPa.	10.8.2
f'_{cm}	Specified minimum compressive strength of concrete at elevated temperatures, MPa.	10.18.2
f_o	Stress due to D + R (the nominal dead load + the nominal load due to rainwater or snow exclusive of the ponding contribution, MPa.	10.16.2
f_v	Required shear strength per unit area, MPa.	10.10.3.7
g	Transverse center-to-center spacing (gage) between fastener gage lines, mm.	10.2.3.13
g	Gap between toes of branch members in a gapped K-connection, neglecting welds, mm.	10.11.2.1
h	Clear distance between flanges less the fillet or corner radius for rolled shapes; for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; for tees, the overall depth; for rectangular HSS, the clear distance between the flanges less the inside corner radius on each side, mm.	10.2.4.2, Table 6.10.8

Symbol	Meaning	Section
h	Distance between centroids of individual components perpendicular to the member axis of buckling,mm.	10.5.6.1
h	Distance between horizontal boundary element centerlines, mm.	10.20.17
h_c	Twice the distance from the centroid to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of fasteners at the compression flange or the inside faces of the compression flange when welds are used, for built-up sections, mm.	10.2.4.2
h_o	Distance between flange centroids, mm.	10.6.2.2
h_o	Distance between flange centroids, mm	10.20.9
h_p	Twice the distance from plastic neutral axis to the nearest line of fasteners at the compression flange or inside face of compression flange when welds are used, mm	10.2.4.2
h_{sc}	Hole factor	10.10.3.8
j	Factor defined by Eq. 6.10.141 for minimum moment of inertia for a transverse stiffener	10.7.2.2
k	Distance from outer face of flange to the web toe of fillet, mm.	10.10.10.2
k	Outside corner radius of HSS, which is permitted to be taken as 1.5t if unknown, mm.	10.11.1.3
k_c	Coefficient for slender unstiffened elements, mm.	Table 6.10.1
k_s	Slip-critical combined tension and shear coefficient	10.10.3.9
k_v	Web plate buckling coefficient	10.7.2.1
l	Largest laterally unbraced length along either flange at the point of load, mm.	10.10.10.4
l	Length of bearing,mm.	10.10.7
l	Length of connection in the direction of loading,mm.	Table 6.10.2
l	Unbraced length between stitches of built-up bracing members,mm.	10.20.13
l	Unbraced length of compression or bracing member, mm.	10.20.13

Symbol	**Meaning**	**Section**
n	Number of nodal braced points within the span	10.19.2
n	Threads per mm.	10.17.4
p	Ratio of element *i* deformation to its deformation at maximum stress	10.10.2.4
p	Projected length of the overlapping branch on the chord	10.11.2.2
q	Overlap length measured along the connecting face of the chord beneath the two branches	10.11.2.2
r	Governing radius of gyration, mm.	10.5.2
r	Governing radius of gyration, mm.	10.20.13
r_{crit}	Distance from instantaneous center of rotation to weld element with minimum $\frac{\Delta_u}{r_i}$ ratio, mm.	10.10.2.4
r_i	Minimum radius of gyration of individual component in a built-up member, mm.	10.5.6.1
r_{ib}	Radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, mm.	10.5.6.1
$\overline{r_o}$	Polar radius of gyration about the shear center,mm.	10.5.4
r_t	Radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone	10.6.4.2
r_{ts}	Effective radius of gyration used in the determination of L_r for the lateral-torsional buckling limit state for major axis bending of doubly symmetric compact I-shaped members and channels	10.6.2.2
r_x	Radius of gyration about geometric axis parallel to connected leg, mm.	10.5.5
r_y	Radius of gyration about y-axis, mm.	10.5.4
r_y	Radius of gyration about *y*-axis, mm.	10.20.9
r_z	Radius of gyration for the minor principal axis, mm.	10.5.5

Symbol	**Meaning**	**Section**
s	Longitudinal center-to-center spacing (pitch) of any two consecutive holes, mm.	10.2.3.13
t	Thickness of element, mm.	10.2.4.2
t	Thickness of element, mm.	Table 6.10.8
t	Wall thickness, mm.	10.5.7.2
t	Angle leg thickness, mm.	10.6.10.2
t	Width of rectangular bar parallel to axis of bending, mm.	10.6.11.2
t	Thickness of connected material, mm.	10.10.3.10
t	Thickness of plate, mm.	10.4.5.1
t	Design wall thickness for HSS equal to 0.93 times the nominal wall thickness for ERW HSS and equal to the nominal wall thickness for SAW HSS, mm.	10.2.3.12
t	Total thickness of fillers, mm.	10.10.5
t	Thickness of connected part, mm.	10.20.7
t	Thickness of column web or doubler plate, mm.	10.20.9
t	Design wall thickness of HSS main member, mm.	10.11.3.1
t	Design wall thickness of HSS main member, mm.	10.11.2.1
t	Design wall thickness of HSS member, mm.	10.11.1.1
t_b	Design wall thickness of HSS branch member, mm.	10.11.2.1
t_b	Design wall thickness of HSS branch member, mm.	10.11.3.1
t_{bf}	Thickness of beam flange, mm.	10.20.9
t_{bj}	Thickness of the overlapped branch,mm.	10.11.2.3
t_{cf}	Thickness of the column flange,mm.	10.10.10.6

Symbol	**Meaning**	**Section**
t_{cf}	Thickness of column flange, mm.	10.20.9
t_f	Thickness of the loaded flange, mm.	10.10.10.1
t_f	Thickness of flange, mm.	10.20.17
t_{fc}	Compression flange thickness, mm.	10.6.4.2
t_p	Thickness of plate, mm.	10.11.1.1
t_p	Thickness of the attached transverse plate, mm.	10.11.2.3
t_p	Thickness of tension loaded plate, mm.	10.17.3
t_p	Thickness of panel zone including doubler plates, mm.	10.20.9
t_s	Web stiffener thickness, mm.	10.19.2
t_w	Web thickness, mm.	Table 6.10.1
t_w	Thickness of web, mm.	Table6.10.8
t_w	Thickness of element, mm.	10.5.7.1
t_w	Column web thickness, mm.	10.10.10.6
t_w	Beam web thickness, mm.	10.19.3
w	Width of cover plate, mm.	10.6.13.3
w	Weld leg size, mm.	10.10.2.2
w	Plate width, mm.	Table 6.10.2
w	Leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, mm.	10.17.3
w_z	Width of panel zone between column flanges, mm.	10.20.9
x	Parameter used for determining the approximate fundamental period	Appendix P.2
x_o,y_o	Coordinates of the shear center with respect to the centroid, mm.	10.5.4

Symbol	**Meaning**	**Section**
$\bar{x}$	Connection eccentricity,mm.	Table 6.10.2
y	Subscript relating symbol to weak axis	
z	Subscript relating symbol to minor principal axis bending	
Z_b	Minimum plastic section modulus at the reduced beam section, mm^3	10.20.9
α	Factor used in Eq. 6.10.2.2	10.3.2.1
α	Separation ratio for built-up compression members $=h/(2r_{ib})$	10.5.6.1
α	Angle of diagonal members with the horizontal	10.20.12
α	Angle of web yielding in radians, as measured relative to the vertical	10.20.17
β	Reduction factor given by Eq. 6.10.159	10.10.2.2
β	The width ratio; the ratio of branch diameter to chord diameter $= D_b/D$ for round HSS; the ratio of overall branch width to chord width $= B_b/B$ for rectangular HSS	10.11.2.1, 10.11.3.1
β	Compression strength adjustment factor	10.20.16
β_T	Brace stiffness requirement excluding web distortion, N-mm/radian.	10.19.2
β_{br}	Required brace stiffness	10.19.2
β_{eff}	Effective width ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width	10.11.2.1
β_{eop}	Effective outside punching parameter	10.11.2.3
β_{sec}	Web distortional stiffness, including the effect of web transverse stiffeners, if any, N-mm/radian.	10.19.2
β_w	Section property for unequal leg angles, positive for short legs in compression and negative for long legs in compression	10.6.10.2
Δ	First-order interstory drift due to the design loads,mm.	10.3.2.2

Symbol	Meaning	Section
Δ	Design story drift	10.20.15
Δ_b	Deformation quantity used to control loading of test specimen (total brace end rotation for the sub-assemblage test specimen; total brace axial deformation for the brace test specimen)	Appendix R.2
Δ_{bm}	Value of deformation quantity, Δ_b, corresponding to the design story drift	Appendix R.6
Δ_{by}	Value of deformation quantity, Δ_b, at first significant yield of test specimen	Appendix R.6
Δ_h	First-order interstory drift due to lateral forces,mm.	10.3.2.1
Δ_i	Deformation of weld elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from instantaneous center of rotation,r_i,mm.	10.10.2.4
Δ_m	Deformation of weld element at maximum stress,mm.	10.10.2.4
Δ_u	Deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, mm.	10.10.2.4
δ	Deformation quantity used to control loading of test specimen	Appendix Q.6
δ_y	Value of deformation quantity δ at first significant yield of test specimen	Appendix Q.6
ρ'	Ratio of required axial force P_u to required shear strength V_u of a link	10.20.15
γ	The chord slenderness ratio; the ratio of one-half the diameter to the wall thickness $=D/(2t)$ for round HSS; the ratio of one-half the width to wall thickness $= B/(2t)$ for rectangular HSS	10.11.2.1, 10.11.3.1
ξ	The gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord = g/ B for rectangular HSS	10.11.2.1

Symbol	Meaning	Section
η	The load length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width $= N/B$, where $N = H_b/\sin\theta$	10.11.2.1, 10.11.3.1
λ	Slenderness parameter	10.6.3
λ_p	Limiting slenderness parameter for compact element	10.2.4
λ_{pf}	Limiting slenderness parameter for compact flange	10.6.3
λ_{pw}	Limiting slenderness parameter for compact web	10.6.4
λ_r	Limiting slenderness parameter for noncompact element	10.2.4
λ_{rf}	Limiting slenderness parameter for noncompact flange	10.6.3
λ_{rw}	Limiting slenderness parameter for noncompact web	10.6.4
λ_p, λ_{ps}	Limiting slenderness parameter for compact element	10.20.8
μ	Mean slip coefficient for class A or B surfaces, as Applicable, or as established by tests	10.10.3.8
ϕ	Resistance factor	10.2.3.3
ϕ	Resistance factor	10.20.6
ϕ_b	Resistance factor for flexure	10.6.1
ϕ_b	Resistance factor for flexure	10.20.8
ϕ_c	Resistance factor for compression	10.5.1
ϕ_c	Resistance factor for compression	10.20.8
ϕ_{sf}	Resistance factor for shear on the failure path	10.4.5.1
ϕ_T	Resistance factor for torsion	10.8.3.1
ϕ_t	Resistance factor for tension	10.4.2
ϕ_v	Resistance factor for shear	10.7.1

Symbol	**Meaning**	**Section**
ϕ_v	Resistance factor for shear strength of panel zone of beam-to-column connections	10.20.9
ϕ_v	Resistance factor for shear	10.20.15
ϕ_{total}	Link rotation angle	Appendix Q.2
Ω	Safety factor	10.2.3.4
Ω	Safety factor	10.20.6
Ω_b	Safety factor for flexure	10.6.1
Ω_b	Safety factor for flexure = 1.67	10.20.8
Ω_c	Safety factor for compression	10.5.1
Ω_c	Safety factor for compression = 1.67	10.20.8
Ω_{sf}	Safety factor for shear on the failure path	10.4.5.1
Ω_T	Safety factor for torsion	10.8.3.1
Ω_t	Safety factor for tension	10.4.2
Ω_v	Safety factor for shear	10.7.1
Ω_v	Safety factor for shear strength of panel zone of beam-to-column connections	10.20.9
Ω_o	Horizontal seismic overstrength factor	10.20.4
θ	Angle of loading measured from the weld longitudinal axis, degrees	10.10.2.4
θ	Acute angle between the branch and chord, degrees	10.11.2.1
θ	Acute angle between the branch and chord, degrees	10.11.3.1
θ	Interstory drift angle, radians	Appendix Q.3
ω	Strain hardening adjustment factor	10.20.16
ε_{cu}	Strain corresponding to compressive strength of concrete, f_c'	10.18.2
τ_b	Parameter for reduced flexural stiffness using the direct analysis method	10.14.3

Symbol	Meaning	Section
ΣM^*_{pc}	Moment at beam and column centerline determined by projecting the sum of the nominal column plastic moment strength, reduced by the axial stress P_{uc}/A_g, from the top and bottom of the beam moment connection	10.20.9
ΣM^*_{pb}	Moment at the intersection of the beam and column centerlines determined by projecting the beam maximum developed moments from the column face. Maximum developed moments shall be determined from test results	10.20.9

10.1.2.2 **Definitions**

ACTIVE FIRE PROTECTION	Building materials and systems that are activated by a fire to mitigate adverse effects or to notify people to take some action to mitigate adverse effects.
ADJUSTED BRACE STRENGTH	Strength of a brace in a buckling-restrained braced frame at deformations corresponding to 2.0 times the design story drift.
ALLOWABLE STRENGTH*	Nominal strength divided by the safety factor, R_n/Ω.
ALLOWABLE STRESS	Allowable strength divided by the appropriate section property, such as section modulus or cross-section area.
AMPLIFICATION FACTOR	Multiplier of the results of first-order analysis to reflect second-order effects.
AMPLIFIED SEISMIC LOAD	Horizontal component of earthquake load E multiplied by Ω_o, where E and the horizontal component of E are specified in the Code.
APPLICABLE BUILDING CODE	Building Code under which the structure is designed.
ASD(ALLOWABLE STRENGTH DESIGN)	Method of proportioning structural components such that the allowable strength equals or exceeds the required strength of the component under the action of the ASD load combinations.
ASD LOAD COMBINATION	Load combination in the Code intended for allowable strength design (allowable stress design).

AUTHORITY HAVING JURISDICTION	Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of the Code.
AVAILABLE STRENGTH*	Design strength or allowable strength, as appropriate.
AVAILABLE STRESS*	Design stress or allowable stress, as appropriate.
AVERAGE RIB WIDTH	Average width of the rib of a corrugation in a formed steel deck.
AUTHORITY HAVING JURISDICTION (AHJ)	Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this Code.
BATTEN PLATE	Plate rigidly connected to two parallel components of a built-up column or beam designed to transmit shear between the components.
BEAM	Structural member that has the primary function of resisting bending moments. Beam-column. Structural member that resists both axial force and bending moment. Bearing. In a bolted connection, limit state of shear forces transmitted by the bolt to the connection elements.
BEARING (LOCAL COMPRESSIVE YIELDING)	Limit state of local compressive yielding due to the action of a member bearing against another member or surface.
BEARING-TYPE CONNECTION	Bolted connection where shear forces are transmitted by the bolt bearing against the connection elements.
BLOCK SHEAR RUPTURE	In a connection, limit state of tension fracture along one path and shear yielding or shear fracture along another path.
BRACED FRAME	An essentially vertical truss system that provides resistance to lateral forces and provides stability for the structural system.
BRANCH FACE	Wall of HSS branch member.
BRANCH MEMBER	For HSS connections, member that terminates at a chord member or main member.
BUCKLING	Limit state of sudden change in the geometry of a structure or any of its elements under a critical loading condition.

BUCKLING STRENGTH	Nominal strength for buckling or instability limit states.
BUCKLING-RESTRAINED BRACED FRAME (BRBF)	Diagonally braced frame safisfying the requirements of Section16 in which all members of the bracing system are subjected primarily to axial forces and in which the limit state of compression buckling of braces is precluded at forces and deformations corresponding to 2.0 times the design story drift.
BUCKLING-RESTRAINING SYSTEM	System of restraints that limits buckling of the steel core in BRBF. This system includes the casing on the steel core and structural elements adjoining its connections. The buckling-restraining system is intended to permit the transverse expansion and longitudinal contraction of the steel core for deformations corresponding to 2.0 times the design story drift.
BUILT-UP MEMBER, CROSS-SECTION, SECTION, SHAPE	Member, cross-section, section or shape fabricated from structural steel elements that are welded or bolted together.
CAMBER	Curvature fabricated into a beam or truss so as to compensate for deflection induced by loads.
CASING	Element that resists forces transverse to the axis of the brace there by restraining buckling of the core. The casing requires a means of delivering this force to the remainder of the buckling-restraining system. The casing resists little or no force in the axis of the brace.
CHARPY V-NOTCH IMPACT TEST	Standard dynamic test measuring notch toughness of a specimen.
CHORD MEMBER	For HSS, primary member that extends through a truss connection.
CLADDING	Exterior covering of structure.
COLD-FORMED STEEL STRUCTURAL MEMBER	Shape manufactured by press-braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold-or hot-rolled coils or sheets; both forming operations being performed at ambient room temperature, that is, without manifest addition of heat such as would be required for hot forming.
COLUMN	Structural member that has the primary function of resisting axial force.

COLUMN BASE	Assemblage of plates, connectors, bolts, and rods at the base of a column used to transmit forces between the steel superstructure and the foundation.
COMBINED SYSTEM	Structure comprised of two or more lateral load-resisting systems of different type.
COMPACT SECTION	Section capable of developing a fully plastic stress distribution and possessing a rotation capacity of approximately three before the onset of local buckling.
COMPARTMENTATION	The enclosure of a building space with elements that have a specific fire endurance.
COMPLETE-JOINT-PENETRATION GROOVE WELD (CJP)	Groove weld in which weld metal extends through the joint thickness, except as permitted for HSS connections.
COMPOSITE	Condition in which steel and concrete elements and members work as a unit in the distribution of internal forces.
CONCRETE CRUSHING	Limit state of compressive failure in concrete having reached the ultimate strain.
CONCRETE HAUNCH	Section of solid concrete that results from stopping the deck on each side of the girder in a composite floor system constructed using a formed steel deck.
CONCRETE-ENCASED BEAM	Beam totally encased in concrete cast integrally with the slab.
CONNECTION	Combination of structural elements and joints used to transmit forces between two or more members.
CONVECTIVE HEAT TRANSFER	The transfer of thermal energy from a point of higher temperature to a point of lower temperature through the motion of an intervening medium.
CONTINUITY PLATES	Column stiffeners at the top and bottom of the panel zone; also known as transverse stiffeners.
CONTRACTOR	Fabricator or erector, as applicable.
COPE	Cutout made in a structural member to remove a flange and conform to the shape of an intersecting member.
COVER PLATE	Plate welded or bolted to the flange of a member to increase cross-sectional area, section modulus or moment of inertia.

CROSS CONNECTION	HSS connection in which forces in branch members or connecting elements transverse to the main member are primarily equilibrated by forces in other branch members or connecting elements on the opposite side of the main member.
DEMANDCRITICAL WELD	Weld so designated by these Provisions.
DESIGNEARTHQUAKE	The earthquake represented by the design response spectrum as specified in the Code.
DESIGNSTORYDRIFT	Amplified story drift (drift under the design earthquake, including the effects of inelastic action), determined as specified in the Code.
DESIGN-BASIS FIRE	A set of conditions that define the development of a fire and the spread of combustion products throughout a building or portion thereof.
DESIGN LOAD*	Applied load determined in accordance with either LRFD load combinations or ASD load combinations, whichever is applicable.
DESIGN STRENGTH*	Resistance factor multiplied by the nominal strength, ϕR_n.
DESIGN STRESS RANGE	Magnitude of change in stress due to the repeated application and removal of service live loads. For locations subject to stress reversal it is the algebraic difference of the peak stresses.
DESIGN STRESS*	Design strength divided by the appropriate section property, such as section modulus or cross section area.
DESIGN WALL THICKNESS	HSS wall thickness assumed in the determination of section properties.
DIAGONAL BRACING	Inclined structural member carrying primarily axial force in a braced frame.
DIAGONAL STIFFENER	Web stiffener at column panel zone oriented diagonally to the flanges, on one or both sides of the web.
DIAPHRAGM PLATE	Plate possessing in-plane shear stiffness and strength, used to transfer forces to the supporting elements.
DIAPHRAGM	Roof, floor or other membrane or bracing system that transfers in-plane forces to the lateral force resisting system.

DIRECT ANALYSIS METHOD	Design method for stability that captures the effects of residual stresses and initial out-of-plumbness of frames by reducing stiffness and applying notional loads in a second-order analysis.
DIRECT BOND INTERACTION	Mechanism by which force is transferred between steel and concrete in a composite section by bond stress.
DISTORTIONAL FAILURE	Limit state of an HSS truss connection based on distortion of a rectangular HSS chord member into a rhomboidal shape.
DISTORTIONAL STIFFNESS	Out-of-plane flexural stiffness of web.
DOUBLE CURVATURE	Deformed shape of a beam with one or more inflection points within the span.
DOUBLE-CONCENTRATED FORCES	Two equal and opposite forces that form a couple on the same side of the loaded member.
DOUBLER	Plate added to, and parallel with, a beam or column web to increase resistance to concentrated forces.
DRIFT	Lateral deflection of structure.
DUAL SYSTEM	Structural system with the following features (1) an essentially complete space frame that provides support for gravity loads; (2) resistance to lateral load provided by moment frames (SMF, IMF or OMF) that are capable of resisting atleast 25 percent of the base shear, and concrete or steel shearwalls, or steel braced frames (EBF,S CBF or OCBF); and (3) each system designed to resist the total lateral load in proportion to its relative rigidity.
DUCTILE LIMIT STATE	Ductile limit states include member and connection yielding, bearing deformation at bolt holes, as well as buckling of members that conform to the width-thickness limitations of Table 6.10.18. Fracture of a member or of a connection, or buckling of a connection element, is not a ductile limit state.
ECCENTRICALLY BRACED FRAME (EBF)	Diagonally braced frame meeting there quirements of Section 15 that has at least one end of each bracing member connected to a beam a short distance from another beam-to-brace connection or a beam-to-column connection.
EFFECTIVE LENGTH FACTOR, K	Ratio between the effective length and the unbraced length of the member.

EFFECTIVE LENGTH	Length of an otherwise identical column with the same strength when analyzed with pinned end conditions.
EFFECTIVE NET AREA	Net area modified to account for the effect of shear lag.
EFFECTIVE SECTION MODULUS	Section modulus reduced to account for buckling of slender compression elements.
EFFECTIVE WIDTH	Reduced width of a plate or slab with an assumed uniform stress distribution which produces the same effect on the behavior of a structural member as the actual plate or slab width with its nonuniform stress distribution.
ELASTIC ANALYSIS	Structural analysis based on the assumption that the structure returns to its original geometry on removal of the load.
ELEVATED TEMPERATURES	Heating conditions experienced by building elements or structures as a result of fire, which are in excess of the anticipated ambient conditions.
ENCASED COMPOSITE COLUMN	Composite column consisting of a structural concrete column and one or more embedded steel shapes.
END PANEL	Web panel with an adjacent panel on one side only.
ENGINEER OF RECORD	Engineer having authority or license from government approved Authority to sign and seal engineering and contract documents.
END RETURN	Length of fillet weld that continues around a corner in the same plane. Engineer of record. Licensed professional responsible for sealing the contract documents. Expansion rocker. Support with curved surface on which a member bears that can tilt to accommodate expansion.
EXPANSION ROLLER	Round steel bar on which a member bears that can roll to accommodate expansion.
EXEMPTED COLUMN	Column not meeting the requirements of Eq. 6.10.300 for SMF.
EXPECTED TENSILE STRENGTH *	Tensile strength of a member, equal to the specified minimum tensile strength, F_u, multiplied by R_t.

EXPECTED YIELD STRENGTH	Yield strength in tension of a member, equal to the expected yield stress multiplied by A_g.
EXPECTED YIELD STRESS	Yield stress of the material, equal to the specified minimum yield stress, F_y,multiplied by R_y.
EYEBAR	Pin-connected tension member of uniform thickness, with forged or thermally cut head of greater width than the body, proportioned to provide approximately equal strength in the head and body.
FACTORED LOAD	Product of a load factor and the nominal load. Fastener. Generic term for bolts, rivets, or other connecting devices.
FATIGUE	Limit state of crack initiation and growth resulting from repeated application of live loads.
FAYING SURFACE	Contact surface of connection elements transmitting a shear force.
FILLED COMPOSITE COLUMN	Composite column consisting of a shell of HSS or pipe filled with structural concrete.
FILLER METAL	Metal or alloy to be added in making a welded joint. Filler. Plate used to build up the thickness of one component. Fillet weld reinforcement. Fillet welds added to groove welds.
FILLET WELD	Weld of generally triangular cross section made between intersecting surfaces of elements.
FIRE	Destructive burning, as manifested by any or all of the following: light, flame, heat, or smoke.
FIRE BARRIER	Element of construction formed of fire-resisting materials and tested in accordance with ASTM Standard E119, or other approved standard fire resistance test, to demonstrate compliance with the Building Code.
FIRE ENDURANCE	A measure of the elapsed time during which a material or assembly continues to exhibit fire resistance.
FIRE RESISTANCE	That property of assemblies that prevents or retards the pas- sage of excessive heat, hot gases or flames under conditions of use and enables them to continue to perform a stipulated function.

FIRE RESISTANCE RATING	The period of time a building element, component or assembly maintains the ability to contain a fire, continues to perform a given structural function, or both, as determined by test or methods based on tests.
FIRST-ORDER ANALYSIS	Structural analysis in which equilibrium conditions are formulated on the undeformed structure; second-order effects are neglected.
FITTED BEARING STIFFENER	Stiffener used at a support or concentrated load that fits tightly against one or both flanges of a beam so as to transmit load through bearing.
FLASHOVER	The rapid transition to a state of total surface involvement in a fire of combustible materials within an enclosure.
FLARE BEVEL GROOVE WELD	Weld in a groove formed by a member with a curved surface in contact with a planar member.
FLARE V-GROOVE WELD	Weld in a groove formed by two members with curved surfaces.
FLAT WIDTH	Nominal width of rectangular HSS minus twice the outside corner radius. In absence of knowledge of the corner radius, the flat width may be taken as the total section width minus three times the thickness.
FLEXURAL BUCKLING	Buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape.
FLEXURAL-TORSIONAL BUCKLING	Buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.
FORCE	Resultant of distribution of stress over a prescribed area.
FORMED SECTION	See cold-formed steel structural member.
FORMED STEEL DECK	In composite construction, steel cold formed into a decking profile used as a permanent concrete form.
FULLY RESTRAINED MOMENT CONNECTION	Connection capable of transferring moment with negligible rotation between connected members.
GAGE	Transverse center-to-center spacing of fasteners.
GAP CONNECTION	HSS truss connection with a gap or space on the chord face between intersecting branch members.

GENERAL COLLAPSE	Limit state of chord plastification of opposing sides of a round HSSchord member at a cross-connection.
GEOMETRIC AXIS	Axis parallel to web, flange or angle leg.
GIRDER FILLER	Narrow piece of sheet steel used as a fill between edge of a deck sheet and flange of a girder in a composite floor system constructed using a formed steel deck.
GIRDER	See Beam.
GIRT	Horizontal structural member that supports wall panels and is primarily subjected to bending under horizontal loads, such as wind load.
GOUGE	Relatively smooth surface groove or cavity resulting from plastic deformation or removal of material.
GRAVITY AXIS	Axis through the center of gravity of a member along its length.
GRAVITY FRAME	Portion of the framing system not included in the lateral load resisting system.
GRAVITY LOAD	Load, such as that produced by dead and live loads, acting in the downward direction.
GRIP (OF BOLT)	Thickness of material through which a bolt passes.
GROOVE WELD	Weld in a groove between connection elements. See also AWS D1.1.
GUSSET PLATE	Plate element connecting truss members or a strut or brace to a beam or column.
HEAT FLUX	Radiant energy per unit surface area.
HEAT RELEASE RATE	The rate at which thermal energy is generated by a burning material.
HORIZONTAL SHEAR	Force at the interface between steel and concrete surfaces in a composite beam.
HSS	Square, rectangular or round hollow structural steel section produced in accordance with a pipe or tubing product specification.
INELASTIC ANALYSIS	Structural analysis that takes into account inelastic material behavior, including plastic analysis.

IN-PLANE INSTABILITY	Limit state of a beam-column bent about its major axis while lateral buckling or lateral-torsional buckling is prevented by lateral bracing.
INSTABILITY	Limit state reached in the loading of a structural component, frame or structure in which a slight disturbance in the loads or geometry produces large displacements.
INTERMEDIATE MOMENT FRAME (IMF)	Moment frame system that meets the requirements of Sec 10.20.10.
INTERSTORY DRIFT ANGLE	Interstory displacement divided by story height, radians.
INVERTED-V-BRACED FRAME	See V-braced frame.
JOINT ECCENTRICITY	For HSS truss connection, perpendicular distance from chord member center of gravity to intersection of branch member work points.
JOINT	Area where two or more ends, surfaces, or edges are attached. Categorized by type of fastener or weld used and method of force transfer.
K-AREA	The *k-area* is the region of the web that extends from the tangent point of the web and the flange-web fillet (AISC "*k*" dimension) a distance of 38 mm in to the web beyond the "*k*" dimension.
K-BRACED FRAME	A bracing configuration in which braces connect to a column at a location with no diaphragm or other out-of-plane support.
K-CONNECTION	HSS connection in which forces in branch members or connecting elements transverse to the main member are primarily equilibrated by forces in other branch members or connecting elements on the same side of the main member.
KSI	Kip per square inch, a US customary unit of stress.
LOWEST ANTICIPATED SERVICE TEMPERATURE (LAST)	The lowest1-hour average temperature with a 100-year mean recurrence interval.
LRFD (LOAD AND RESISTANCE FACTOR DESIGN)	Method of proportioning structural components such that the design strength equals or exceeds the required strength of the component under the action of the LRFD load combinations.

LRFD LOAD COMBINATION	Load combination in the Code intended for strength design (load and resistance factor design).
LACING	Plate, angle or other steel shape, in a lattice configuration, that connects two steel shapes together.
LAP JOINT	Joint between two overlapping connection elements in parallel planes.
LATERAL BRACING	Diagonal bracing, shear walls or equivalent means for providing in-plane lateral stability.
LATERAL BRACING MEMBER	Member that is designed to inhibit lateral buckling or lateral- torsional buckling of primary framing members.
LATERAL LOAD RESISTING SYSTEM	Structural system designed to resist lateral loads and provide stability for the structure as a whole.
LATERAL LOAD	Load that produced by wind or earthquake effects, acting in a lateral direction.
LATERAL-TORSIONAL BUCKLING	Buckling mode of a flexural member involving deflection normal to the plane of bending occurring simultaneously with twist about shear center of the cross-section.
LEANING COLUMN	Column designed to carry gravity loads only, with connections that are not intended to provide resistance to lateral loads.
LENGTH EFFECTS	Consideration of the reduction in strength of a member based on its unbraced length.
LIMIT STATE	Condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to have reached its ultimate load-carrying capacity (strength limit state).
LINK	In EBF, the segment of a beam that is located between the ends of two diagonal braces or between the end of a diagonal brace and a column. The length of the link is defined as the clear distance between the end soft wodiagonal braces or between the diagonal brace and the column face.
LINK INTERMEDIATE WEB STIFFENERS	Vertical web stiffeners placed within the link in EBF.

LINK ROTATION ANGLE	Inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift.
LINK SHEAR DESIGN STRENGTH	Lesser of the available shear strength of the link developed from the moment or shear strength of the link.
LOAD	Force or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes.
LOAD EFFECT	Forces, stresses and deformations produced in a structural component by the applied loads.
LOAD FACTOR	Factor that accounts for deviations of the nominal load from the actual load, for uncertainties in the analysis that transforms the load into a load effect and for the probability that more than one extreme load will occur simultaneously.
LOCAL BENDING**	Limit state of large deformation of a flange under a concentrated tensile force.
LOCAL BUCKLING**	Limit state of buckling of a compression element within a cross section.
LOCAL CRIPPLING**	Limit state of local failure of web plate in the immediate vicinity of a concentrated load or reaction.
LOCAL YIELDING**	Yielding that occurs in a local area of an element.
LRFD (LOAD AND RESISTANCE FACTOR DESIGN)	Method of proportioning structural components such that the design strength equals or exceeds the required strength of the component under the action of the LRFD load combinations.
LRFD LOAD COMBINATION	Load combination in the Code intended for strength design (load and resistance factor design.
MAIN MEMBER	For HSS connections, chord member, column or other HSS member to which branch members or other connecting elements are attached.
MEASURED FLEXURAL RESISTANCE	Bending moment measured in a beam at the face of the column, for a beam-to-column test specimen tested in accordance with Appendix S.
MECHANISM	Structural system that includes a sufficient number of real hinges, plastic hinges or both, so as to be able to articulate in one or more rigid body modes.