B a n g l a d e s h

Bangladesh National Building Code (BNBC)

2006

Ministry of Works

Editorial note: According to the information provided by the national delegate, the codes have not been changed since 1993, and have enhanced in 2006 and will be revised in 2009. Earthquake Resistant Design part of the code is in the World List 2008.

Comments on Building Codes

1. General
   a. Name of Country: **Bangladesh**
   b. Name of Codes: **Bangladesh National Building Code (BNBC)**
   Earthquake Resistant Design part of the code is attached as pdf file.

   --- in English
   --- in Original Language

   c. Issued by: **Ministry of Works**
   d. Enforcement Year: **2006**

2. Structural Design Method
   a. Format: (please check)
   √☐ **Working Stress Design** : Allowable Stress ≧ Actual Stress
   √☐ **Ultimate Strength Design**: Ultimate Member Strength ≧ Required Member Strength
   ☐ Limit State Design : Ultimate Lateral Strength ≧ Required Lateral Strength
   ☐ Other Design Method :
   (comment)

   b. Material Strength (Concrete and Steel):

The working stress design for all loading limits the allowable stress of concrete to be 0.45 of the specified concrete strength, and that of steel to be approximately 0.5 of the specified yield strength.

   c. Strength Reduction Factors:
   Are used. See Equation 2.5.1 (Page 6-53) and Table 6.2.24 (Page 6-54).

   d. Load Factors for Gravity Loadings and Load Combination:
   These factors are provided in article 2.7.4 and 2.7.5 (Page 6-68 and 6-69)

   e. Typical Live Load Values:
   Office Buildings : 3.0 kN/m^2
   Residential Buildings: 2.0 kN/m^2

   f. Special Aspects of Structural Design Method

Details of Masonry structures are provided in Chapter 4
Details of reinforced concrete structures are provided in Chapter 8
Details of steel structures are provided in Chapter 10

BANGLADESH NATIONAL BUILDING CODE

1993

collected and analysed in relation with the geological and geotectonic characteristics of the country. A seismic zoning map of Bangladesh together with the seismic design provision was thus drawn up.

In order to ensure wide participation by and interaction with the vast number of professionals involved in the building construction field across the country, a three day workshop was held at the Institution of Engineers premises at Dhaka in May 1993, following submission of the preliminary draft code by the consultant. The workshop was organized into sixteen sessions of related disciplines and covered some fifty-seven hours of presentation, analysis and discussion. Some 185 professionals representing 125 organizations were invited to participate in the workshop. These organizations included professional societies, technological and general universities, all the Institutes of Technology, various government agencies and sector corporations, city development authorities, selected municipal organizations, research organizations, consulting firms, construction firms, producers and suppliers of building materials, non-government organizations involved in development projects, federation of chambers of commerce and industry, the legal profession, and concerned government ministries.

Another joint workshop of the Steering Committee and the editorial subcommittees was held on June 12, 1993, to ensure that comments, observations and suggestions received during the May workshop were being properly incorporated in the subsequent drafts of the code. A review meeting was also held at the Planning Commission on August 24, 1993, after submission of the draft code by the consultant. The meeting was attended by officials of the concerned departments and chaired by the honourable Minister for Planning. Following submission of the final draft by the consultant, a meeting of the Steering Committee was held on December 28, 1993, where the building code was approved for publication.

This document, the Bangladesh National Building Code, has been prepared in ten distinct parts comprising different aspects of building construction and services with cross references as necessary. Part 1 gives a general introduction to the code and lists the definitions and abbreviations of general terms used in the code. Part 2 outlines the administrative requirements necessary for enforcement of the code. It should be borne in mind that enforcement of the code is a continuous activity and requires a standing administrative structure for various jurisdictions in the country. Considering the difficulty of maintaining adequate technical personnel for enforcing and verifying compliance with diverse provision of the code, a professional practice-based certification, enforcement and administration structure has been prescribed.

General planning and architectural requirements of buildings, based on classifications in accordance with occupancy and fire resistance, are specified in Part 3. The specifications cover requirements within the premises of the building plot for all categories of buildings. As the area planning requirements involve parameters not within the control of individual building developer and are matters of interest to government planners, these are not included in the building code. The requirements have been set keeping in view the tropical climate of the country and the local architectural practices and tradition. In setting the minimum requirements for various types of buildings, the urgent need of providing accommodation to the lower income majority of the population was given due consideration. A separate occupancy class has been recommended for such housing, for which lower minimum standards for various parameters of planning and design have been specified. These lower requirements will, however, be applicable only for designated mass housing projects for the lower income people.

Arrangement for safety from fire in buildings is of paramount importance, particularly in built up or city areas. Part 4 specifies the requirements for fire prevention and protection measures in buildings. The measures are divided into three categories — precautionary measure to prevent or arrest propagation of fire in buildings, provision of life saving means of escape from the building in the event of fire, and provision of in-built fire fighting arrangements within buildings. Requirements for each of these types of protective measures are specified in this part of the code. These are followed by specific requirements for fire protection of various occupancy classes of buildings. The fire protection requirements of the code are based on the principle of providing reasonable protection within achievable means.

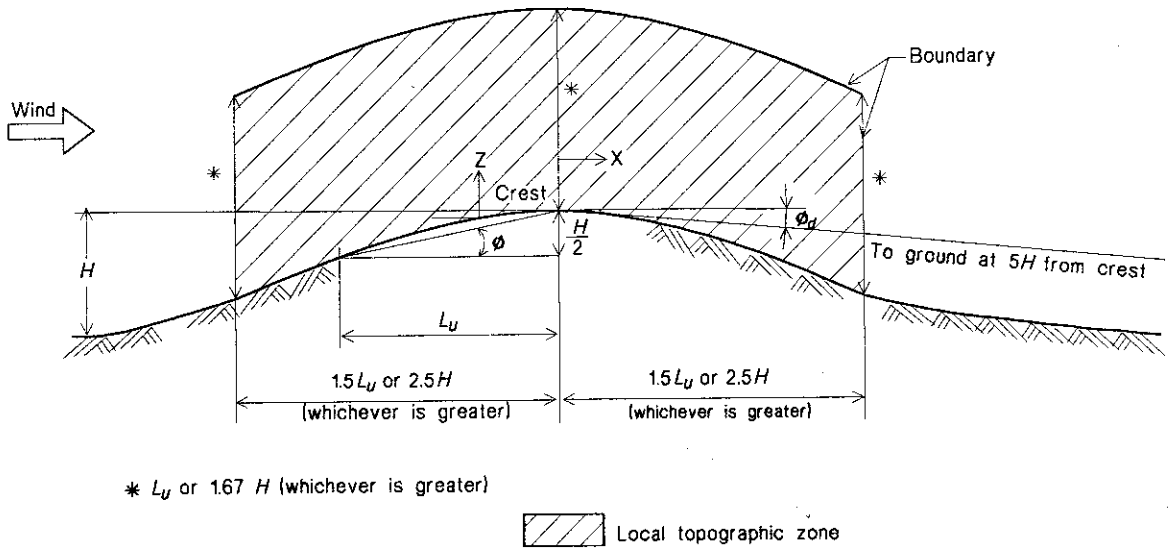
Part 5 sets the standards of materials to be used in building construction. Materials covered include all types of common construction materials as well as some indigenous building materials of the country. The requirements for materials provided in the building code are based on specifications of established standards issued by standards agencies. The agency responsible for issuing standards and ensuring compliance with these standards in Bangladesh is the Bangladesh Standards and Testing Institution. In general, the building code specifies compliance with relevant Bangladesh standards. Where Bangladesh standards are not available or are inadequate, the most applicable and widely used standards of other countries for the relevant materials have been specified. These will be replaced as more and more Bangladesh standards are available.

Requirements governing structural design that ensure safety and serviceability of buildings are specified in Part 6. The specifications cover the design of buildings in various structural materials — masonry, reinforced and prestressed concrete, steel, timber and ferrocement. The requirements for design of building foundations on various kinds of soil are also specified. Structural design is influenced by the loads that are put on the building both by the occupancy and by the forces of nature. Natural forces are purely a local phenomenon and have been worked out after a thorough study of the pertinent conditions of meteorology, geology and other features of the country. Data of many years for the cyclonic wind of the coastal region as well as the extreme wind data of other regions were collected from the Meteorological Department and other sources. Earthquake data of hundreds of years for the north-eastern region of the subcontinent were also collected from reliable sources. These data were statistically analysed for various return periods and duly considered together with the local natural features. The exercise resulted in the preparation of the first design wind speed map of Bangladesh and a revised seismic zoning map. These and the methods of and requirements for calculation of various loads acting on the building, including those due to wind and earthquake, are specified in this part of the code. The special requirements for earthquake resistant design and detailing of buildings made of masonry, concrete and steel are also specified. Ferrocement has emerged in the recent years as a promising alternative to common and traditional materials and is a strong and durable building material for low cost construction. The material has been given formal treatment in the code and specifications for design of buildings made of ferrocement have been incorporated.

Construction industry in Bangladesh is highly labour intensive and the success of a project lies to a great extent on proper site management and construction practices. Ensuring safety of life during construction and minimization of construction hazards are the concern of Part 7. Constructional responsibilities regarding planning and control of the construction as well as the protection of public, workers and property are specified in this part. The minimum requirements of on-site welfare measure for health and sanitation of the workers are also specified. The specifications additionally provide for the safe and scientific demolition of buildings, where necessary.

A building requires various services — electrical, mechanical, acoustic, sanitary, water supply, gas supply. The specifications of Part 8 set standards of minimum requirements for the various services required for proper functioning of the building. It should be noted that not all the services provided for in the code are essential requirements of a building, but the services when installed should satisfy the requirements of Part 8. The actual requirements of services for specific occupancy types are prescribed in Part 3.

2.4.6.8 Effect of Local Topography : If a structure or any portion thereof is located within a local topographic zone, such as regions around hills and ridges as shown in Fig 6.2.9, the sustained wind pressure obtained from Sec 2.4.6.2 shall be modified by multiplying by a local topographic coefficient, C_t. Value of the coefficient, C_t shall be obtained from Fig 6.2.9.



Local Topographic Coefficient, C_t at Crest

Upwind slope (tan ϕ)	Coefficient, C_t
0.05	1.19
0.1	1.39
0.2	1.85
$\geq$ 0.3	2.37

Legend:

tanϕ = the upwind slope, $\dfrac{H}{2L_u}$

tanϕ_d = the average downwind slope, measured from the crest of a hill or ridge or to the ground level at a distance of $5H$.

H = the height of the hill or ridge in meters

L_u = the horizontal distance upwind from the crest to a level half the height below the crest in meters.

Notes:
(1) For intermediate values of upwind slope, linear interpolation is permitted.
(2) C_t = 1.0 for a point at or out side the boundary of the local topographic zones as shown in the figure . For any point within the local topographic zone, value of the coefficient, C_t shall be obtained by interpolation from the value at crest given in the table and the value of C_t=1 at the boundary of the zone. The interpolation shall be linear with horizontal distance from the crest, and with height above the local ground level.

Fig 6.2.9 Local Topographic Coefficient, C_t for Hills and Ridges.

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 this section. For primary framing systems of buildings or structures, the design seismic lateral forces shall be calculated either by the Equivalent Static Force Method or by the Dynamic Response Method based on the criteria set forth in Sec 2.5.5.1. Overall design of buildings and structures to resist seismic ground motion and other forces shall comply with the applicable design requirements given in Chapter 1.

2.5.2 Definitions

The following definitions of terms shall be applicable only to the provisions of Sec 2.5 :

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 at the base of a structure.

BEARING WALL SYSTEM : A structural system without a complete vertical load carrying space frame, see Sec 1.3.2.

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 gravity loads, see Sec 1.3.2.

DIAPHRAGM : A horizontal or nearly horizontal system of structures acting to transmit lateral forces to the vertical resisting elements. The term "diaphragm" includes 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.

ECCENTRIC BRACED FRAME (EBF) : A steel braced frame designed in conformance with Sec 1.8.

ESSENTIAL FACILITIES : Buildings and structures which are necessary to remain functional during an emergency or a post disaster period.

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.

FLEXIBLY SUPPORTED EQUIPMENT : Non-rigid or flexibly supported equipment is a system having a fundamental period, including the equipment, greater than 0.06 second.

HORIZONTAL BRACING SYSTEM : A horizontal truss system that serves the same function as a floor or roof diaphragm.

INTERMEDIATE MOMENT RESISTING FRAME (IMRF) : A concrete or steel frame designed in accordance with Sec 8.3 or 10.5.17 respectively.

MOMENT RESISTING FRAME : A frame in which members and joints are capable of resisting forces primarily by flexure.

ORDINARY MOMENT RESISTING FRAME (OMRF) : 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.

RIGIDLY SUPPORTED EQUIPMENT : A rigid or rigidly supported equipment is a system having a fundamental period less than or equal to 0.06 second.

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).

SOFT STOREY : Storey in which the lateral stiffness is less than 70 per cent of the stiffness of the storey 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 RESISTING FRAME (SMRF) : 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.

SPECIAL STRUCTURAL SYSTEM : A structural system not listed in Table 6.2.24.

STOREY : The space between floor levels. 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.

STRUCTURE : An assemblage of framing members designed to support gravity loads and resist lateral forces. Structures may be categorized as building and non-building structures as defined in Sec 1.2.2.

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 per cent of that of the storey above.

2.5.3 Symbols and Notation

The following symbols and notation shall apply to the provisions of this section :

A_c	=	the combined effective area, in square metres of the shear walls in the first storey of the structure.
A_e	=	the effective horizontal cross-sectional area, in square metres of a shear wall in the first storey of the structure.
A_x	=	the torsion amplification factor at level-x.
C	=	numerical coefficient specified in Sec 2.5.6.1.
C'	=	numerical coefficient specified in Sec 2.5.8 and given in Table 6.2.26.
C_t	=	numerical coefficient given in Sec 2.5.6.2.
D_e	=	the length in metres of a shear wall element in the first storey in the direction parallel to the applied forces.
f_t	=	lateral force at level -i for use in Eq (2.5.5).
F_i, F_n, F_x	=	lateral force applied to level-i, -n, or -x respectively.
F'	=	lateral forces on an element or component or on equipment supports.
F_t	=	that portion of the base shear V, considered concentrated at the top of the structure in addition to F_n.
F'_x	=	force on floor- or roof-diaphragm.
g	=	acceleration due to gravity.
h_i, h_n, h_x	=	height in metres above the base to level i, -n or -x respectively.
I	=	structure importance coefficient given in Table 6.2.23.
I'	=	structure importance coefficient specified in Sec 2.5.8 for structural and non-structural components and equipment.
Level-i	=	level of the structure referred to by the subscript i, e.g., $i = 1$ designates the first level above the base.
Level-n	=	the uppermost level in the main portion of the structure.
Level-x	=	the level under consideration e.g., $x = 1$ designates the first level above the base.
R	=	response modification coefficient for structural systems given in Table 6.2.24.
S	=	site coefficient for soil characteristics given in Table 6.2.25.
T	=	fundamental period of vibration, in seconds, of the structure in the direction under consideration.
V	=	the total design lateral force or shear at the base
V_x	=	the design storey shear in storey x
W	=	the total seismic dead load defined in Sec 2.5.5.2
w_i, w_x	=	that portion of W which is located at or assigned to level -i or -x respectively
w'_x	=	the weight of the diaphragm and the elements tributary thereto at level-x, including applicable portions of other loads defined in Sec 2.5.5.2.

W'	=	the weight of an element or component
Z	=	seismic zone coefficient given in Table 6.2.22.
δ_i	=	horizontal displacement at level-i relative to the base due to applied lateral forces, in metre, for use in Eq (2.5.5).

2.5.4 **Seismic Zoning**

2.5.4.1 Seismic Zoning Map : The seismic zoning map of Bangladesh is provided in Fig 6.2.10. Based on the severity of the probable intensity of seismic ground motion and damages, Bangladesh has been divided into three seismic zones, i.e. Zone 1, Zone 2 and Zone 3 as shown in Fig 6.2.10 with Zone 3 being the most severe.

2.5.4.2 Selection of Seismic Zone and Zone Coefficient : Seismic zone for a building site shall be determined based on the location of the site on the Seismic Zoning Map provided in Fig 6.2.10. Each building or structure shall be assigned a Seismic Zone Coefficient, Z corresponding to the seismic zone of the site as set forth in Table 6.2.22.

2.5.5 **Design Earthquake Forces for Primary Framing Systems**

The design earthquake lateral forces on the primary framing systems of every building or structure shall be calculated based on the provisions set forth in this section. The design seismic forces shall be assumed to act nonconcurrently in the direction of each principal axis of the building or the structure, except otherwise required by the provisions of Sec 1.5.4 and 1.7.

2.5.5.1 Selection of Lateral Force Method : Seismic lateral forces on primary framing systems shall be determined by using either the Equivalent Static Force Method provided in Sec 2.5.6, or the Dynamic Response Method given in Sec 2.5.7 complying with the restrictions given below :

a) The Equivalent Static Force Method of Sec 2.5.6 may be used for the following structures :

i) All structures, regular or irregular, in Seismic Zone 1 and in Structure Importance Category IV in Seismic Zone 2, except case b(iv) below.
ii) Regular structures under 75 metres in height with lateral force resistance provided by structural systems listed in Table 6.2.24. except case b(iv) below.
iii) Irregular structures not more than 20 metres in height.
iv) A tower like building or structure having a flexible upper portion supported on a rigid lower portion where:

1) both portions of the structure considered separately can be classified as regular structures,
2) the average storey stiffness of the lower portion is at least ten times the average storey stiffness of the upper portion, and
3) the period of the entire structure is not greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

b) The Dynamic Response Method as given in Sec 2.5.7 may be used for all classes of structure, but shall be used for structures of the following types.

i) Structures 75 metres or more in height except as permitted by case a(i) above.
ii) Structures having a stiffness, weight or geometric vertical irregularity of Type I, II, or III as defined in Table 6.1.3. or structures having irregular features not described in either Table 6.1.3 or 6.1.4.
iii) Structures over 20 metres in height in Seismic Zone 3 not having the same structural system throughout their height except as permitted by Sec 1.6.4.
iv) Structures, regular or irregular, located on Soil Profile Type S_4 as given in Table 6.2.25, which have a period greater than 0.7 second. The analysis shall include the effects of the soils at the site and shall conform to Sec 2.5.7.1 (c).

2.5.5.2 Seismic Dead Load : Seismic dead load, W, is the total dead load of a building or a structure, including permanent partitions, and applicable portions of other loads listed below :

a) In storage and warehouse occupancies, a minimum of 25 per cent of the floor live load shall be applicable.

b) Where an allowance for partition load is included in the floor design in accordance with Sec 2.3.3.3, all such loads but not less than 0.6 kN/m^2 shall be applicable.

c) Total weight of permanent equipment shall be included.

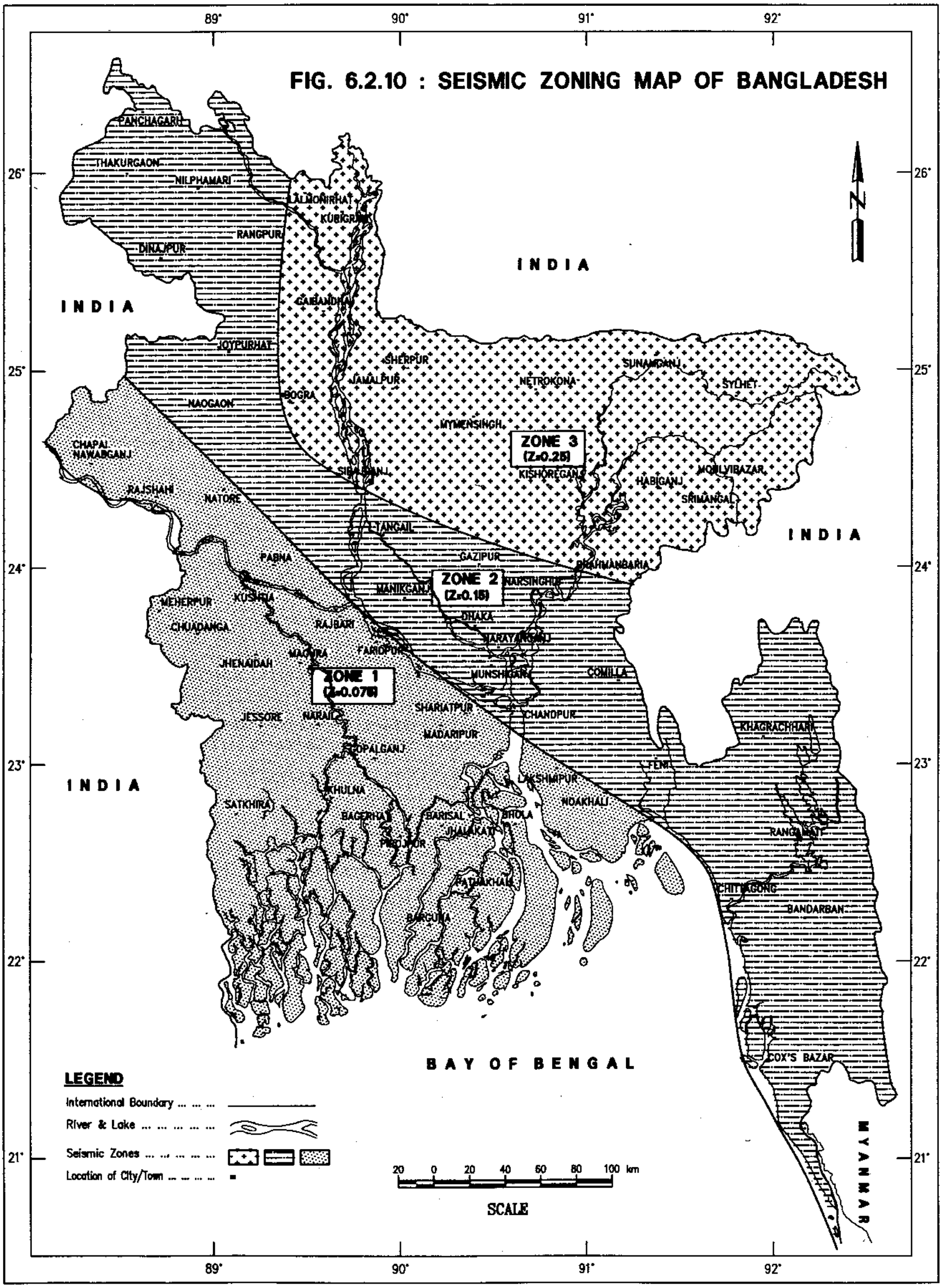


FIG. 6.2.10 : SEISMIC ZONING MAP OF BANGLADESH

2.5.6 Equivalent Static Force Method

This method may be used for calculation of seismic lateral forces for all structures specified in Sec 2.5.5.1(a)

2.5.6.1 Design Base Shear : The total design base shear in a given direction shall be determined from the following relation :

$$V = \frac{ZIC}{R} W \quad (2.5.1)$$

where,
- Z = Seismic zone coefficient given in Table 6.2.22
- I = Structure importance coefficient given in Table 6.2.23
- R = Response modification coefficient for structural systems given in Table 6.2.24
- W = The total seismic dead load defined in Sec 2.5.5.2
- C = Numerical coefficient given by the relation :

$$C = \frac{1.25S}{T^{2/3}} \quad (2.5.2)$$

- S = Site coefficient for soil characteristics as provided in Table 6.2.25
- T = Fundamental period of vibration in seconds, of the structure for the direction under consideration as determined by the provisions of Sec 2.5.6.2.

The value of C need not exceed 2.75 and this value may be used for any structure without regard to soil type or structure period. Except for those requirements where Code prescribed forces are scaled up by $0.375R$, the minimum value of the ratio C/R shall be 0.075.

Table 6.2.22
Seismic Zone Coefficients, Z

Seismic Zone (see Fig 6.2.10)	Zone Coefficient
1	0.075
2	0.15
3	0.25

Table 6.2.23
Structure Importance Coefficients *I*, *I′*

Structure Importance Category (see Table 6.1.1 for occupancy)	Structure Importance Coefficient	
	I	I'
I Essential facilities	1.25	1.50
II Hazardous facilities	1.25	1.50
III Special occupancy structures	1.00	1.00
IV Standard occupancy structures	1.00	1.00
V Low-risk Structures	1.00	1.00

2.5.6.2 Structure Period : The value of the fundamental period, T of the structure shall be determined from one of the following methods :

a) **Method A** : For all buildings the value of T may be approximated by the following formula :

$$T = C_t (h_n)^{3/4} \quad (2.5.3)$$

where,
- C_t = 0.083 for steel moment resisting frames
- = 0.073 for reinforced concrete moment resisting frames, and eccentric braced steel frames
- = 0.049 for all other structural systems
- h_n = Height in metres above the base to level n.

Alternatively, the value of C_t for buildings with concrete or masonry shear walls may be taken as $0.031/\sqrt{A_c}$. The value of A_c shall be obtained from the relation :

$$A_c = \sum A_e \left[0.2 + (D_e/h_n)^2\right] \quad (2.5.4)$$

where,
- A_c = The combined effective area, in square metres, of the shear walls in the first storey of the structure.
- A_e = The effective horizontal cross-sectional area, in square metres of a shear wall in the first storey of the structure.
- D_e = The length, in metre of a shear wall element in the first storey in the direction parallel to the applied forces.

The value of D_e/h_n for use in Eq (2.5.4) shall not exceed 0.9.

Table 6.2.24
Response Modification Coefficient for Structural Systems, *R*

Basic Structural System[1]	Description of Lateral Force Resisting System	R [2]
a. Bearing Wall System	1. Light framed walls with shear panels	
	i) Plywood walls for structures, 3 storeys or less	8
	ii) All other light framed walls	6
	2. Shear walls	
	i) Concrete	6
	ii) Masonry	6
	3. Light steel framed bearing walls with tension only bracing	4
	4. Braced frames where bracing carries gravity loads	
	i) Steel	6
	ii) Concrete [3]	4
	iii) Heavy timber	4
b. Building Frame System	1. Steel eccentric braced frame (EBF)	10
	2. Light framed walls with shear panels	
	i) Plywood walls for structures 3-storeys or less	9
	ii) All other light framed walls	7
	3. Shear walls	
	i) Concrete	8
	ii) Masonry	8
	4. Concentric braced frames (CBF)	
	i) Steel	8
	ii) Concrete [3]	8
	iii) Heavy timber	8
c. Moment Resisting Frame System	1. Special moment resisting frames (SMRF)	
	i) Steel	12
	ii) Concrete	12
	2. Intermediate moment resisting frames (IMRF), concrete[4]	8
	3. Ordinary moment resisting frames (OMRF)	
	i) Steel	6
	ii) Concrete [5]	5
d. Dual System	1. Shear walls	
	i) Concrete with steel or concrete SMRF	12
	ii) Concrete with steel OMRF	6
	iii) Concrete with concrete IMRF [4]	9
	iv) Masonry with steel or concrete SMRF	8
	v) Masonry with steel OMRF	6
	vi) Masonry with concrete IMRF [3]	7
	2. Steel EBF	
	i) With steel SMRF	12
	ii) With steel OMRF	6
	3. Concentric braced frame (CBF)	
	i) Steel with steel SMRF	10
	ii) Steel with steel OMRF	6
	iii) Concrete with concrete SMRF [3]	9
	iv) Concrete with concrete IMRF [3]	6
e. Special Structural Systems	See Sec 1.3.2, 1.3.3, 1.3.5	

Notes : (1) Basic Structural Systems are defined in Sec 1.3.2, Chapter 1.
(2) See Sec 2.5.6.6 for combination of structural systems, and Sec 1.3.5 for system limitations.
(3) Prohibited in Seismic Zone 3.
(4) Prohibited in Seismic Zone 3 except as permitted in Sec 2.5.9.3.
(5) Prohibited in Seismic Zones 2 and 3. Sec 1.7.2.6.

Table 6.2.25
Site Coefficient, S for Seismic Lateral Forces [(1)]

Type	Site Soil Characteristics – Description	Coefficient, S
S_1	A soil profile with either : a) A rock-like material characterized by a shear-wave velocity greater than 762 m/s or by other suitable means of classification, or b) Stiff or dense soil condition where the soil depth is less than 61 metres	1.0
S_2	A soil profile with dense or stiff soil conditions, where the soil depth exceeds 61 metres	1.2
S_3	A soil profile 21 metres or more in depth and containing more than 6 metres of soft to medium stiff clay but not more than 12 metres of soft clay	1.5
S_4	A soil profile containing more than 12 metres of soft clay characterized by a shear wave velocity less than 152 m/s	2.0

Note : (1) The site coefficient shall be established from properly substantiated geotechnical data. In locations where the soil properties are not known in sufficient detail to determine the soil profile type, soil profile S_3 shall be used. Soil profile S_4 need not be assumed unless the building official determines that soil profile S_4 may be present at the site, or in the event that soil profile S_4 is established by geotechnical data.

b) **Method B** : The fundamental period T may be calculated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. This requirement may be satisfied by using the following formula :

$$T = 2\pi\sqrt{\sum_{i=1}^{n} w_i\delta_i^2 \Big/ g\sum_{i=1}^{n} f_i\delta_i} \qquad (2.5.5)$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of Eq (2.5.6), (2.5.7) and (2.5.8) or any other rational distribution. The elastic deflections, δ_i shall be calculated using the applied lateral forces, f_i. The value of T determined from Eq (2.5.5) shall not exceed that calculated using Eq (2.5.3) by more than 40%.

2.5.6.3 Vertical Distribution of Lateral Forces : In the absence of a more rigorous procedure, the total lateral force, which is the base shear V, shall be distributed along the height.of the structure in accordance with Eq (2.5.6), (2.5.7) and (2.5.8):

$$V = F_t + \sum_{i=1}^{n} F_i \qquad (2.5.6)$$

where, F_i = Lateral force applied at storey level -i and
F_t = Concentrated lateral force considered at the top of the building in addition to the force F_n.

The concentrated force, F_t acting at the top of the building shall be determined as follows:

$$F_t = 0.07\,TV \le 0.25\,V \quad \text{when } T > 0.7 \text{ second} \qquad (2.5.7a)$$
$$F_t = 0.0 \quad \text{when } T \le 0.7 \text{ second} \qquad (2.5.7b)$$

The remaining portion of the base shear $(V\text{-}F_t)$, shall be distributed over the height of the building, including level-n, according to the relation :

$$F_x = \frac{(V - F_t)w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (2.5.8)$$

At each storey level-x, the force F_x shall be applied over the area of the building in proportion to the mass distribution at that level.

2.5.6.4 Horizontal Distribution of Shear : The design storey shear V_x, in any storey x is the sum of the forces F_x and F_t above that storey. V_x shall be distributed to the various elements of the vertical lateral force resisting system in proportion to their rigidities, considering the rigidity of the floor or roof diaphragm. Allowance shall also be made for the increased shear arising due to any horizontal torsional moments as specified in Sec 2.5.6.5.

2.5.6.5 Horizontal Torsional Moments : Provision shall be made for the increased shears resulting from horizontal torsion where floor diaphragms are not flexible.

The torsional design moment at a given storey shall be the moment resulting from eccentricities between applied design lateral forces at levels above that storey and the vertical resisting elements in that storey plus an accidental torsional moment.

The accidental torsional moment in any storey shall be determined assuming the storey mass to be displaced from the calculated centre of mass in each direction a distance equal to 5% of the building dimension at that level perpendicular to the direction of the force under consideration.

Where torsional irregularity exists (Plan Irregularity Type I as defined in Table 6.1.4) the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor, A_x determined from the formula:

$$A_x = \left[\delta_{max}/(1.2\delta_{avg})\right]^2 \leq 3.0 \qquad (2.5.9)$$

where, δ_{max} = The maximum displacement at level-x.

δ_{avg} = The average of the displacements at extreme positions of the building at level-x.

The more severe loading for each element shall be considered for design.

2.5.6.6 Combination of Structural Systems : When structural systems defined in Sec 1.3.2 are combined to be incorporated into the same structure, the following requirements shall be satisfied:

a) Vertical Combinations: The value of the response modification coefficient, R used in the design of any storey for a given direction shall not be greater than that used for the storey above. However, this requirement need not apply to a storey where the dead load above that storey is less than 10 per cent of the total dead weight of the structure.

Structures may be designed using the procedures of Sec 2.5.6 under the following conditions:

i) The entire structure is designed using the lowest value of R for the lateral force resisting systems used, or

ii) The following procedure is used for structures conforming to Sec 2.5.5.1a(iv).

1. The flexible upper portion, shall be designed as a separate structure, supported laterally by the rigid lower portions using the appropriate value of R.

2. The rigid lower portion shall be designed as a separate structure using the appropriate value of R . The reactions from the upper portion shall be increased by the ratio of the R values of the two portions. These factored reactions shall be applied at the top of the rigid lower portion in addition to the forces determined for the lower portion itself.

b) Combinations Along Different Axes:

i) In Seismic Zone 3, where a structure has a Bearing Wall System in only one direction, the value of R used for the orthogonal direction shall not be greater than that used for the Bearing Wall System defined in Sec 1.3.2.

ii) Any combination of Building Frame Systems, Dual Systems, or Moment Resisting Frame Systems defined in Sec 1.3.2 may be used to resist design seismic forces in structures less than 50 m in height. Only combinations of Dual Systems and Special Moment Resisting Frames (SMRF) can be used to resist the design seismic forces in structures exceeding 50 m in height in Seismic Zone 3.

2.5.7 **Dynamic Response Method**

The Dynamic Response Method, where used, shall conform to the criteria established in this section. The analysis of the structure shall be based on an established principle of mechanics, using a mathematical model specified in Sec 1.2.6.1(a) and one of the dynamic analysis procedures given in Sec 2.5.7.2 and 2.5.7.3.

The mass and mass moments of inertia of various components of a structure, required for the dynamic analysis, shall be calculated based on the seismic dead load specified in Sec 2.5.5.2.

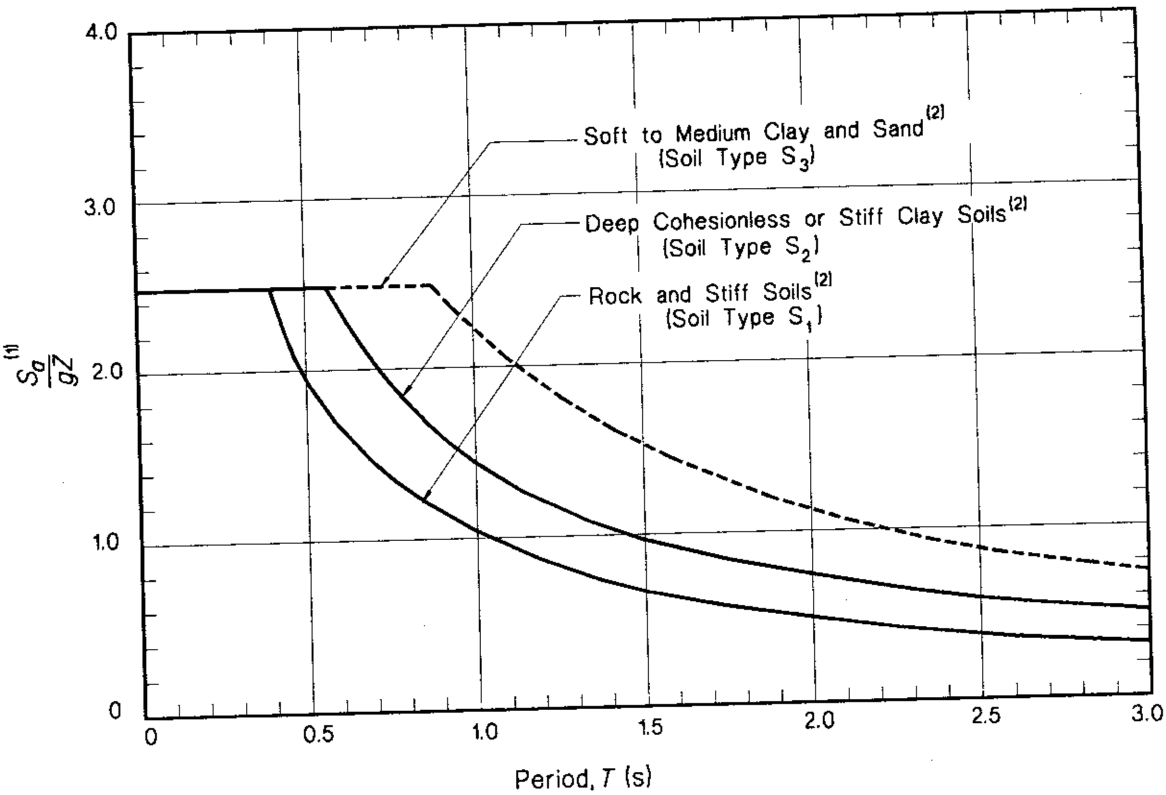
2.5.7.1 Ground Motion : The ground motion representation as set out in this section shall, as a minimum, be one having 20% probability of being exceeded in 50 years and may be one of the following:

a) Response Spectrum : The response spectrum to be used in the dynamic analysis shall be any one of the following:

   i) Site Specific Design Spectra : A site specific response spectra shall be developed based on the geologic, tectonic, seismologic, and soil characteristics associated with the specific site. The spectra shall be developed for a damping ratio of 0.05 unless a different value is found to be consistent with the expected structural behaviour at the intensity of vibration established for the site.

   ii) Normalized Response Spectra : In absence of a site-specific response spectrum, the normalized response spectra given in Fig 6.2.11 shall be used in the dynamic analysis procedure given in Sec 2.5.7.2.

b) Time History : Ground motion time history developed for the specific site shall be representative of actual earthquake motions for the directions under consideration. Response spectra from time history, either individually or in combination, shall approximate the site-specific design spectra conforming to paragraph a (i) above.



Note: (1) S_a: Spectral acceleration
g: Acceleration due to gravity
Z: Seismic zone coefficient.
(2) For structures on Soil Type S_4, refer to Sec 2.5.7.1(c) .

Fig. 6.2.11 Normalized Response Spectra for 5% Damping Ratio

c) Structures on Soil Profile Type S_4 : The following provisions shall apply when required by Sec 2.5.5.1 b (iv):

i) The ground motion representation shall be developed in accordance with paragraphs a (i) and b above.

ii) Possible amplification of building response due to soil-structure interaction and lengthening of building period caused by inelastic behaviour shall be considered.

iii) The base shear determined by these procedures may be reduced to a design base shear, V, by dividing by a factor not greater than the appropriate R value for the structure but shall not be less than that required by Sec 2.5.7.2c(i).

d) Vertical Component: The vertical component of ground motion may be defined by scaling the corresponding horizontal ground accelerations by a factor of two-thirds. Alternative factors may be used when substantiated by site-specific data.

2.5.7.2 Response Spectrum Analysis : Where this procedure is used, an elastic dynamic analysis of a structure shall be performed based on the criteria set forth in this section with a mathematical model conforming to Sec 1.2.6.1(a) and using a response spectrum as specified in Sec 2.5.7.1(a). The analysis shall include the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal response shall be calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions shall be combined in a statistical manner to obtain an approximate total structural response.

a) Number of Modes : The requirement that all significant modes be included may be satisfied by demonstrating that, for the modes considered, at least 90 per cent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

b) Combination of Modes : The peak member forces, displacements, storey forces, storey shears, and base reactions for each mode shall be combined using established procedures in order to estimate resultant maximum values of these response parameters. When three dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maximum.

c) Scaling of Results : Where the base shear for a given direction, determined by this procedure, is different from the base shear obtained by using the procedure of Sec 2.5.6.1, it shall be adjusted as follows :

i) When the base shear is less than that determined from Sec 2.5.6.1, the following values shall be taken :

1. The value of the base shear as obtained from Sec 2.5.6.1, for irregular structures.

2. 90 per cent of the value from Sec 2.5.6.1 for regular structures except that the base shear shall not be less than 80 per cent of that determined using T from Sec 2.5.6.2(a).

ii) When the base shear is greater than that determined from Sec 2.5.6.1, the value need not exceed that required by c(i) above, except for structures required to conform to Sec 2.5.7.1(c)

All corresponding response parameters, including deflections, member forces and moments, shall be adjusted in proportion to the adjusted base shear.

d) Torsion : The analysis shall account for torsional effects, including accidental torsional effects as prescribed in Sec 2.5.6.5. Where three-dimensional models are used for analysis, effects of accidental torsion shall be accounted for by appropriate adjustments in the model such as adjustment of mass locations, or by the equivalent static procedure provided in Sec 2.5.6.5.

2.5.7.3 Time History Analysis : When this procedure is followed, an elastic or inelastic dynamic analysis of a structure shall be made using a mathematical model of the structure specified in Sec 1.2.6.1(a) and applying at its base or any other appropriate level, a ground motion time history as specified in Sec 2.5.7.1(b). The time-dependent dynamic response of the structure shall be obtained through numerical integration of its equations of motion.

2.5.8 **Seismic Lateral Forces on Components and Equipment Supported by Structures**

2.5.8.1 Lateral Forces on Structural and Non-structural Components, and Equipment : The minimum design seismic lateral forces on elements of structures, non-structural components, equipment and their attachments including anchorage and bracing to the main structural system shall be determined in accordance with the formula :

$$F' = ZI'C'W' \qquad (2.5.10)$$

where, F' = Total lateral seismic force
Z = Seismic zone coefficient as given in Table 6.2.22
I' = Structure Importance Coefficient for components as given in Table 6.2.23
C' = Horizontal force Coefficient as specified in Sec 2.5.8.2.
W' = Weight of an element, component or piece of equipment.

The total lateral seismic force, F' obtained from Eq (2.5.10) shall be distributed in proportion to the mass distribution of the element, component or piece of equipment. These forces shall be applied in the horizontal direction to cause the most critical loading for design. Friction resulting from gravity forces shall not be considered to provide resistance to seismic forces.

Seismic lateral forces on attachments for floor- or roof-mounted equipment weighing less than 1.8 kN and for furniture need not be determined for design purposes.

2.5.8.2 Horizontal Force Coefficient C' : The value of the coefficient C' shall be determined as follows :

a) For elements of structure and non-structural components, and for rigid or rigidly supported equipment supported by structures above grade, C' shall be taken as those given in Table 6.2.26.

b) For non-rigid or flexibly supported equipment, supported by a structure and located above grade on a structure, the seismic lateral force shall be determined considering the dynamic properties of both the equipment and those of the structure which supports it, but the value of C' shall not be less than that listed in Table 6.2.26. In the absence of an analysis or empirical data, the value of C' shall be taken as twice the value listed in Table 6.2.26 but it need not exceed 2.0.

For piping, ducting and conduit systems which are constructed of ductile materials and connections, the values of C' may be taken as those given in Table 6.2.26.

c) The value of C' for elements, or components and equipment laterally self-supported and located at or below ground level may be two-thirds of the value set forth in Table 6.2.26. However, the design lateral forces obtained from Eq (2.5.10) for these elements shall not be less than that as would be obtained using the provision of Sec 2.5.9.

2.5.8.3 Seismic Lateral Forces on Floor or Roof Diaphragms : Seismic lateral forces on floor and roof diaphragms and collector elements shall be determined in accordance with the following formula :

$$F'_x = \frac{(F_t + \sum_{i=x}^{n} F'_i)}{\sum_{i=x}^{n} w_i} w'_x \qquad (2.5.11)$$

a) The force F'_x determined from Eq (2.5.11) need not exceed 0.75 ZIw'_x but it shall not be less than 0.35 ZI.

b) When the diaphragm is required to transfer lateral forces from the vertical resisting elements above the diaphragm to other vertical resisting elements below the diaphragm due to offset in the placement of the elements or to changes in stiffness in the vertical elements, these forces shall be added to those determined from Eq (2.5.11).

2.5.9 Seismic Lateral Forces on Non-Building Structures

Non-building structures shall include all self-supporting structures other than buildings that carry gravity loads and resist the effects of earthquake and other lateral forces. Determination of seismic lateral forces for such structures shall be based on the following provisions:

2.5.9.1 Seismic Dead Load : For non-building structures, the seismic dead load, W shall include all loads defined for buildings in Sec 2.5.5.2. In addition, W' shall include all normal operating contents for structures such as tanks, vessels, bins and piping.

2.5.9.2 Fundamental Period : For structures with primary framing systems similar to buildings, the fundamental period T, shall be determined in accordance with Sec 2.5.6.2. For other structures, T shall be obtained by using a rational method such as Method B of Sec 2.5.6.2.

2.5.9.3 Structures Similar to Buildings : The seismic lateral forces on structures with primary framing systems similar to buildings (i.e. structural systems listed in Table 6.2.24) shall be determined in accordance with the provisions of Sec 2.5.5 through 2.5.8 with following modifications :

a) Intermediate moment resisting frames (IMRF) may be used in structures within Seismic Zone 3 and in structure importance categories III through *V*, if, (i) the structure is less than 15 m in height, and (ii) $R = 4.0$ is used in load calculations.

b) Seismic dead load and structure period shall be calculated in accordance with Sec 2.5.5.2 and 2.5.9.2 respectively.

Table 6.2.26
Horizontal Force Coefficient, C' for Elements, Components and Equipment

	Elements of Structures and Non-structural Components and Equipment[1]	Value of C'
I	Elements of Structures	
	1. Walls including the following:	
	a. Unbraced (cantilevered) parapets	2.00
	b. Other exterior walls above the ground floor[2,3]	0.75
	c. All interior bearing and nonbearing walls and partitions[3]	0.75
	d. Masonry or concrete fences over 1.8 m high	0.75
	2. Penthouse (except when framed by an extension of the structural frame)	0.75
	3. Connections for prefabricated structural elements other than walls, with force applied at centre of gravity [4]	0.75
	4. Diaphragms[3,5]	—
II.	Non-structural Components	
	1. Exterior and interior ornamentation and appendages	2.00
	2. Chimneys, stacks, trussed towers and tanks on legs:	
	a. Supported on or projecting as an unbraced cantilever above the roof more than one-half their total height	2.00
	b. All others, including those supported below the roof with unbraced projection above the roof less than one-half their height, or braced or guyed to the structural frame at or above their centres of mass	0.75
	3. Signs and billboards	2.00
	4. Storage racks (including contents)	0.75
	5. Anchorage for permanent floor-supported cabinets and book stacks more than 1.5 m in height (including contents)	0.75
	6. Anchorage for suspended ceilings and light fixtures[4, 6]	0.75
	7. Access floor systems[4, 7]	0.75
III.	Equipment	
	1. Tanks and vessels (including contents), together with support systems and anchorage	0.75
	2. Electrical, mechanical and plumbing equipment and associated conduit, ductwork and piping, and machinery [8]	0.75

Notes:
(1) See Sec 2.5.8.2 for items supported at or below grade.
(2) See Sec 1.7.2.3 and 2.5.8.2
(3) Where flexible diaphragms provide lateral support for walls and partitions, the value of C' for anchorage shall be increased 50 per cent for the centre one-half of the diaphragm span.
(4) Applies to Seismic Zones 2 and 3 only.
(5) See Sec 1.7.2.9 and 2.5.8.3.
(6) Ceiling weight shall include all light fixtures and other equipment or partitions which are laterally supported by the ceiling. For the purpose of determining the seismic force, a ceiling weight of not less than 0.2 kN/m^2 shall be used. Ceilings constructed of lath and plaster or gypsum board, screw or nail attached to suspended members that support a ceiling at one level extending from wall to wall need not be analysed provided the walls are not over 15 m apart.
(7) W' for access floor systems shall be the dead load of the access floor systems plus 25 per cent of the floor live load plus a 0.5 kN/m^2 partition load allowance.
(8) Equipment includes, but is not limited to , boilers, chillers, heat exchangers, pumps, air-handing units, cooling towers, control panels, motors, switchgear, transformers and life-safety equipment. It also includes major conduit, ducting and piping serving such equipment and fire sprinkler systems. See Sec 2.5.8.2 for additional requirements for determining C' for non-rigid or flexibly mounted equipment.

2.5.9.4 Rigid Structures : For rigid structures (i.e. those with period, $T \leq 0.06$ second) including their anchorage, the total lateral force, V shall be determined in accordance with the relation :

$$V = 0.5\, Z\, I\, W \qquad (2.5.12)$$

2.5.9.5 Flat-bottom Tanks at or Below Grade : Seismic forces for flat-bottom tanks or other tanks with supported bottoms, founded at or below grade, shall be calculated using the procedure of Sec 2.5.9.4 considering the entire weight of the tank and its contents. Alternatively, such forces may be determined using one of the following methods.

a) A response spectrum analysis, which includes consideration of the actual ground motion anticipated at the site and the inertial effects of the contained fluid.

b) A substantiated analysis prescribed for the particular type of tank provided that the seismic Zones and Structure Importance Categories are in conformance with Fig 6.2.10 and Sec 1.2.3 respectively.

2.5.9.6 Other Structures : For structures (other than buildings), which are not covered by Sec 2.5.9.3 through 2.5.9.5, the minimum seismic lateral forces shall be determined in accordance with the following provisions :

a) The total lateral seismic force, V shall be determined using the provisions of Sec 2.5.6 with the coefficient R taken from Table 6.2.27. However, the ratio C/R shall not be less than 0.5.

Table 6.2.27
Coefficient, R for Non-Building Structures

	Structure Type	Coefficient R
1.	Tanks, vessels or pressurized spheres on braced or unbraced legs	3
2.	Cast-in-place concrete silos and chimneys having walls continuous to the foundation	5
3.	Distributed mass cantilever structures such as stacks, chimneys, silos and skirt-supported vertical vessels	4
4.	Trussed towers (free standing or guyed), guyed stacks and chimneys	4
5.	Inverted pendulum-type structures	3
6.	Cooling towers	5
7.	Bins and hoppers on braced or unbraced legs	4
8.	Storage racks	5
9.	Signs and billboards	5
10.	Amusement structures and monuments	3
11.	All other self-supporting structures not otherwise covered	4

b) The vertical distribution of the total lateral seismic force, V, may be determined by one of the following procedures:

1. Using provisions of Sec 2.5.6.3.
2. Using procedures of Sec 2.5.7.

Exception:
For irregular structures assigned to Structure Importance Categories I and II, which cannot be modeled as a single mass, the procedures of Sec 2.5.7 shall be used.

c) When any other established standard or method is used as a basis for obtaining the seismic lateral forces for a particular type of non-building structure covered by this section, such a standard may be used subject to the following limitations:

i) The Seismic Zones and Structure Importance Categories shall be in conformance with the requirements of Sec 2.5.4 and 1.2.3 respectively.

ii) The values for total lateral force and total base overturning moment used in design shall not be less than 80% of the values which would be obtained using these provisions.

2.6.10 **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 unfavourable 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 2.7.4 or 2.7.5 whichever is applicable. However, once Sec 2.7.4 or 2.7.5 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 Sec 2.7.4 and 2.7.5 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 result 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 **Definitions**

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).

DESIGN STRENGTH : The product of the nominal strength and a resistance factor.

FACTORED LOAD : The product of the nominal load and a load factor.

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).

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.

NOMINAL LOADS : The magnitudes of the loads such as dead, live, wind, earthquake etc. specified in Sec 2.2 through 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.

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.

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).

WORKING STRESS DESIGN METHOD (WSD) : See ALLOWABLE STRESS DESIGN METHOD.

2.7.3 **Symbols and Notation**

D = dead load consisting 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.

E = earthquake load

E' = amplified earthquake load equal to $(0.375R)E$

F = loads due to fluids with well-defined pressures and maximum heights, including loads due to water pressure during flood and surge.

H = loads due to weight and lateral pressure of soil and water in soil
L = $L_f + (L_r$ or $P)$
L_f = 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. L_f includes any permissible reduction. If resistance to impact loads is taken into account in design, such effects shall be included with the live loads L_f.
L_r = roof live loads
P = loads due to initial rainwater ponding
R = seismic coefficient defined in Sec 2.5.3
S = self-straining forces and effects arising from contraction or expansion resulting from temperature changes, shrinkage, moisture changes, creep in component materials, movement due to differential settlement, or combinations thereof.
W = wind load

2.7.4 Combinations of Loads and Stress Increase for Allowable Stress Design Method

2.7.4.1 Combination of Loads : Provisions of this section shall apply to all construction materials permitting their use in proportioning structural members by allowable stress 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 unfavourable effect shall be used in design.

1. D
2. $D + L$
3. $D + S$
4. D + (W or E)
5. $0.9D$ + (W or E)
6. D + (H or F)
7. $D + L$ + (H or F)
8. $D + S + L$
9. $D + S$ + (W or E)
10. $D + L$ + (W or E)
11. $D + L$ + (H or F) + (W or E)
12. $D + S + L$ + (H or F) + (W or E)

2.7.4.2 Stress Increase : Except as specified in Sec 1.5.5.(b) and elsewhere in this Code, the maximum permissible increase in the allowable stresses of all materials and soil bearing capacities specified in this Code for working (or allowable) stress design method, when load combinations (7) through (11) in Sec 2.7.4.1 above is used, shall be 33%.

2.7.5 Combinations of Loads 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 :

2.7.5.1 Load Combinations for Reinforced Concrete and Masonry Structures

1. $1.4D$
2. $1.4D + 1.7L$
3. $1.4D + 1.4S$
4. $0.9D$ + 1.3 (W or $1.1E$)
5. $0.9D$ + 1.7 (H or F)
6. $1.4D + 1.7L$ + 1.7 (H or F)
7. 0.75 [$1.4D + 1.4S + 1.7L$]
8. 0.75 [$1.4D + 1.4S$ + 1.7 (W or $1.1E$)]
9. 0.75 [$1.4D + 1.7L + 1.7W$]
10. 0.75 [$1.4D + 1.7L$ + 1.7 (H or F) + 1.7 (W or $1.1E$)]
11. 0.75 [$1.4D + 1.4S + 1.7L$ + 1.7 (H or F) + 1.7 (W or $1.1E$)]
12. 1.4 (D+L+E)

2.7.5.2 Load Combinations for Steel Structures

1. $1.4D$
2. $1.2D + 1.6L_f$ + 0.5 (L_r or P)
3. $1.2D$ + 1.6 (L_r or P) + ($0.5L_f$ or $0.8W$)
4. $1.2D + 1.3W + 0.5L_f$ + 0.5 (L_r or P)
5. $1.2D + 1.5E + (0.5L_f)$
6. $0.9D$ + ($1.3W$ or $1.5E$)

Exception :
The load factor on L_f in combinations (3), (4) and (5) shall be equal to 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load exceeds 5.0 kN/m^2.

When the structural effects of F, H, or S are significant, their factored values shall be considered as $1.3F$, $1.6H$, and $1.2S$ and included with the above combinations to obtain the most unfavourable effect.

Also for buildings in Seismic Zone 3 and in Seismic Zone 2 having an Structural Importance Coefficient, I greater than 1.0, the following additional load combinations shall be considered :

7. $1.2D + 0.5L + E'$
8. $0.9D + E'$

2.7.5.3 Load Combinations for Design using Other Materials : When structural members are designed using the strength design method and using a construction material not covered in Sec 2.7.5.1 and 2.7.5.2, any other code or standard having load combinations applicable for that construction material may be used provided that other requirements of Sec 2.7 are satisfied.

Related Appendix

Appendix A	Conversion of Expressions from SI to FPS Units

4.7.2.5 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 6.4.14.

Table 6.4.14
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 (4.7.11).

$$V_n = V_m + V_s \tag{4.7.11}$$

where :

$$V_m = 0.083 C_d A_{mv}\sqrt{(f'_m)} \tag{4.7.12}$$

The value of C_d in Eq (4.7.12) is given as :

$$C_d = 2.4 \quad \text{for}\ \frac{M}{Vd} \le 0.25$$

$$= 1.2 \quad \text{for}\ \frac{M}{Vd} \ge 1.0$$

and

$$V_s = A_{mv}\rho_n f_y \tag{4.7.13}$$

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 (4.7.14)

$$V_n = A_{mv}\rho_n f_y \tag{4.7.14}$$

The required shear strength for this region shall be calculated at a distance $L_w/2$ 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 (4.7.11).

4.7.2.6 Reinforcement : Reinforcement shall be in accordance with the following :

i) Minimum reinforcement shall be provided in accordance with Sec 4.8.5.1 for all seismic areas using this method of analysis.

ii) 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 (4.7.7).

iii) All continuous reinforcement shall be anchored or spliced in accordance with Sec 4.6.6.4 with $f_s = 0.5f_y$

iv) Vertical reinforcement shall not be less than 50 per cent of the horizontal reinforcement.

v) Spacing of horizontal reinforcement within the region defined in Sec 4.7.2.5(c) shall not exceed three times the nominal wall thickness or 600 mm, whichever is smaller.

4.7.2.7 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 $0.2 f'_m$. The boundary members may be discontinued where the calculated compressive stresses are less than $0.15 f'_m$. 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 $0.4 f'_m$.

d) The minimum length of the boundary member shall be 3 times the thickness of the wall.

e) 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.

4.8 EARTHQUAKE RESISTANT DESIGN

4.8.1 General

All masonry structures constructed in the Seismic Zones 2 and 3 shown in Fig 6.2.10 shall be designed in accordance with the provisions of this Section.

4.8.2 Loads

Seismic forces on masonry structures shall be determined in accordance with the provisions of Sec 2.5 of this Part.

4.8.3 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.

4.8.4 Provisions for Seismic Zone 2

4.8.4.1 Wall Reinforcement : Vertical reinforcement of at least 12 mm ϕ 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 ϕ 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.

4.8.4.2 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.

4.8.4.3 Columns : Columns shall be reinforced as specified in Sec 4.6.6.1.

4.8.5 Provisions for Seismic Zone 3

All masonry structures built in Seismic Zone 3 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.

4.8.5.1 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 4.6.6.7 shall be considered as continuous reinforcement.

4.8.5.2 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.

4.8.5.3 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 $0.0007bt$.

4.8.5.4 Minimum Dimension

i) 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.

ii) Columns : The least nominal dimension of a reinforced masonry column shall be 375 mm except that if the allowable stresses are reduced to 50 per cent of the values given in Sec 4.3, the minimum nominal dimension shall be 250 mm.

4.8.5.5 Shear Wall

i) 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.

ii) 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.

iii) 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.

4.8.5.6 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.

4.8.5.7 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.

4.8.6 Additional Requirements

4.8.6.1 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 equal to at least one-eighth of the height of the opening for Seismic Zone 2 and one-fourth of the height for Seismic Zone 3.

d) The horizontal distance between two openings shall not be less than one-fourth of the height of the shorter opening for Seismic Zone 2 and one-half of the height for Seismic Zone 3.

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 4.8.6.5.

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 an opening is tall say, for the full height of wall, dividing the wall into two portions, these portions 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.

j) The use of arches to span over the openings is a source of weakness and shall be avoided unless steel ties are provided.

4.8.6.2 Strengthening Arrangements : All masonry buildings shall be strengthened by the methods specified in Table 6.4.15.

Table 6.4.15
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	Up to 2 with pitched roof	a) Masonry mortar shall not be leaner than M_3 b) By lintel and roof band (Sec 4.8.6.3) c) By vertical reinforcement at corners and junctions of walls (Sec 4.8.6.4) 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 4.8.6.3) c) By vertical reinforcement at corners and junctions of walls (Sec 4.8.6.4) d) Vertical reinforcement at jambs of openings (Sec 4.8.6.5) e) Bracing in plan at tie level for pitched roof*
3	Up to 4	a) Masonry mortar shall not be leaner than M_3 b) By lintel and roof band (Sec 4.8.6.3) c) By vertical reinforcement at corners and junctions of walls (Sec 4.8.6.4) d) Vertical reinforcement at jambs of openings (Sec 4.8.6.5) 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.

4.8.6.3 Bands : Roof band need not be provided underneath reinforced concrete or brickwork slabs resting on bearing walls, provided 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 6.4.16. 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 6.4.16
Band Reinforcement

Seismic Zones	Reinforcement		Links
	Plain Mild Steel Bars	High Strength Deformed Bars	
2	2 - 12 mm ϕ, one on each face of the wall with suitable cover	2 - 10 mm ϕ, one on each face of the wall with suitable cover	6 mm dia, 150 mm c/c
3	2 - 16 mm ϕ, one on each face of the wall with suitable cover	2 - 12 mm ϕ, one on each face of the wall with suitable cover	6 mm dia, 150 mm c/c

4.8.6.4 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 6.4.17. 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 Fig 6.4.1 and Fig 6.4.2.

b) Details of vertical reinforcement given in Table 6.4.17 are applicable to brick masonry and hollow block masonry.

Table 6.4.17
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		Diameter of Single Bar or Equivalent Area of High Strength Deformed Bar to be Provided	
		Zone 2 (mm)	Zone 3 (mm)	Zone 2 (mm)	Zone 3 (mm)
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

4.8.6.5 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 Fig 6.4.3.

4.8.6.6 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.

4.9 PROVISIONS FOR HIGH WIND REGIONS

4.9.1 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.

4.9.2 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 :

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 Sec 8.2.15.2 through 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, 8.2.14.2, and where applicable to 8.2.15.2(d) or 8.2.15.2(e) below, where the bar stress 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 compression members, if lateral ties are used having an area of at least 0.0015 hs, lap splice length may 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) If spiral reinforcement confines the splice, the lengths required may 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 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 Sec 8.2.12 through Sec 8.2.15 above.

8.2.17 **Mechanical Anchorage**

8.2.17.1 Any mechanical device capable of developing the strength of reinforcement without damage to concrete is allowed as anchorage.

8.2.17.2 Mechanical device may be used only when its adequacy can be proven by test results to the satisfaction of the engineer.

8.2.17.3 Development of reinforcement may consist of a combination of mechanical anchorage plus additional embedment length of reinforcement between the point of maximum bar stress and the mechanical anchorage.

8.3 SPECIAL PROVISION FOR SEISMIC DESIGN

8.3.1 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 (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
b = effective compressive flange width of a structural member, mm
b_w = web width or diameter of circular section, mm
d_b = bar diameter, mm
E = load effects of earthquake or related internal moments and forces

f'_c = specified compressive strength of concrete
f_y = specified yield strength of reinforcement
f_{yh} = specified yield strength of transverse reinforcement
h_c = cross-sectional dimension of column core measured centre to centre of confining reinforcement
h_w = height of entire wall (diaphragm) or of the segment of wall (diaphragm) considered
ℓ_d = development length for a straight bar
ℓ_{dh} = development length for a bar with a standard hook
ℓ_o = minimum length, measured from joint face along axis of structural member, over which 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
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
M_s = portion of slab moment balanced by support moment
s = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, mm
s_o = maximum spacing of transverse reinforcement, mm
V_c = nominal shear strength provided by concrete
V_e = design shear force
V_n = nominal shear strength
V_u = factored shear force at section
α_c = coefficient defining the relative contribution of concrete strength to wall strength
ρ = ratio of tension reinforcement = A_s / bd
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)
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.

8.3.2 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 deg with at least a six diameter extension at one end but not less than 75 mm, and a hook not less than 90 deg with at least a six diameter extension at the other end. The hooks shall engage peripheral longitudinal bars. The 90 deg hooks of two successive cross ties engaging the same longitudinal bars shall be alternated end for end.

DEVELOPMENT LENGTH FOR A BAR WITH A STANDARD HOOK : The shortest distance between the critical section and a tangent to the outer edge of the 90 deg 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.

8.3.3 General Requirements

8.3.3.1 Scope

a) 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.

b) The provisions of Chapter 6, shall apply except as modified by the provisions of this section.

c) In regions of moderate seismic risk, Zone 2 (see Chapter 2), reinforced concrete frames resisting forces induced by earthquake motions shall be built to satisfy the requirements of Sec 8.3.10 in addition to the requirements of Chapter 6.

d) In regions of high seismic risk, Zone 3 (see Chapter 2), all reinforced concrete structures shall satisfy the requirements of Sec 8.3.3 through 8.3.9 in addition to the requirements of Chapter 6.

8.3.3.2 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.

8.3.3.3 Strength Reduction Factors : Strength reduction factors shall be in accordance with Sec 6.1.4.

8.3.3.4 Concrete in Members Resisting Earthquake Induced Forces : Compressive strength f'_c of the concrete shall be not less than 20 N/mm^2.

8.3.3.5 Reinforcement in Members Resisting Earthquake Induced Forces : Reinforcement resisting earthquake induced flexural and axial forces in frames and wall boundary members shall comply with ASTM A706, ASTM A615 and BDS 1313. Reinforcement with f_y = 275 N/mm^2 and f_y = 410 N/mm^2 are allowed in these members if (a) the actual yield strength based on mill tests does not exceed the specified yield strength by more than 125 N/mm^2 (retests shall not exceed this value by more than an additional 20 N/mm^2), and (b) the ratio of the actual ultimate tensile strength to the actual tensile yield strength is not less than 1.25.

8.3.3.6 Reinforcement required by factored load combinations which include earthquake effect shall not be welded except as specified in Sec 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 Frames

8.3.4.1 Scope : Requirements of this section shall apply to frame members, (i) resisting earthquake induced forces, and (ii) proportioned primarily to resist flexure. These frame members shall also satisfy the following conditions:

a) Factored axial compressive force on frame member shall not exceed $0.1A_g f'_c$.

b) Clear span for the member 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 not exceeding three-fourths of the depth of the flexural member.

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 $1.38b_w d/f_y$ and the reinforcement ratio, ρ shall not exceed 0.025. At least two bars shall be provided continuously both top and bottom.

b) 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. 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.

c) 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 $d/4$ 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.

d) Welded splices and mechanical connections conforming to Sec 8.2.12.3(a) through 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.

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.

ii) Over lengths equal to twice the member depth, 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. Maximum spacing of the hoops shall not exceed (i) $d/4$, (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 Sec 8.1.10.4(c), and where hoops are not required, stirrups shall be spaced not more than $d/2$ throughout the length of the member.

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 deg with 6 diameter but not less than 75 mm extension anchored in the confined core and a cross tie to make a closed hoop. Consecutive cross ties engaging the same longitudinal bar shall have their 90 deg 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 deg hooks of the cross ties shall all be placed on that side.

8.3.5 **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:

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.

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 :

$$\sum M_e \geq 1.2 \sum M_g \qquad (8.3.1)$$

where

$\sum M_e$ = sum of moments, at the centre of the joint, corresponding to the design flexural strength of the columns framing into that joint. The lowest flexural strength of the columns, calculated for the factored axial force, consistent with the direction of the lateral forces considered, shall be used.

$\sum M_g$ = sum of moments, at the centre of the joint, corresponding to the design flexural strengths of the girders framing into that joint.

Flexural strengths shall be summed such that the column moments oppose the beam moments. Eq (8.3.1) 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

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 Sec 8.2.12.3(a) through 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.

8.3.5.4 Transverse Reinforcement

a) Transverse reinforcement shall be provided as specified below 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 :

$$\rho_s = \frac{0.12 f'_c}{f_{yh}} \tag{8.3.2}$$

and shall not be less than that required by Eq (6.3.3).

ii) The total cross-sectional area of rectangular hoop reinforcement shall not be less than that given by the following equations :

$$A_{sh} = 0.3\left(s h_c f'_c / f_{yh}\right)\left[\left(A_g / A_{ch}\right) - 1\right] \tag{8.3.3}$$

$$A_{sh} = \frac{0.09 s h_c f'_c}{f_{yh}} \tag{8.3.4}$$

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 (8.3.3) and (6.3.3) need not be satisfied.

b) Transverse reinforcement shall not be spaced more than one-quarter of the minimum member dimension nor 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 $0.1 A_g f'_c$ 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.

8.3.6 **Structural Walls and Diaphragms**

8.3.6.1 Scope : The requirements of this section apply to structural walls serving as parts of the earthquake force resisting systems as well as to diaphragms, struts, ties, chords and collector members which transmit forces induced by earthquake.

8.3.6.2 Reinforcement

a) The reinforcement ratio, ρ_v for structural walls shall not be less than 0.0025 along the longitudinal and transverse directions. Reinforcement spacing each way shall not exceed 450 mm. Reinforcement provided for shear strength shall be continuous and shall be distributed across the shear plane. If the design shear force does not exceed $0.083A_{cv}\sqrt{f'_c}$, the shear reinforcement may conform to Sec 6.9.7.

b) At least two layers of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds $0.17A_{cv}\sqrt{f'_c}$.

c) Structural truss members, struts, ties, and collector members with compressive stresses exceeding $0.2f'_c$ shall have special transverse reinforcement, as specified in Sec 8.3.5.4 over the total length of the member. The special transverse reinforcement is allowed to be discontinued at a section where the calculated compressive stress is less than $0.15f'_c$. Stresses shall be calculated for the factored forces using a linear elastic model and gross section properties of the members considered.

d) All continuous reinforcement in structural walls, diaphragms, trusses, struts, ties, chords, and collector members shall be anchored or spliced in accordance with the provisions for reinforcement in tension as specified in Sec 8.3.7.4.

8.3.6.3 Boundary Members for Structural Walls and Diaphragms

a) Boundary members shall be provided at boundaries and edges around openings of structural walls and diaphragms for which the maximum extreme fibre stress exceeds $0.2f'_c$ unless the entire wall or diaphragm member is reinforced to satisfy Sec 8.3.5.4(a) through 8.3.5.4(c). The boundary members may 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.

b) Where required, boundary members shall have transverse reinforcement as specified in Sec 8.3.5.4(a) through 8.3.5.4(c).

c) Boundary members of structural walls shall be designed to carry all factored gravity loads on the wall, including tributary loads and self weight, as well as the vertical force required to resist overturning moment calculated from factored forces related to earthquake effect.

d) Boundary members of structural diaphragms shall be proportioned to resist the sum of the factored axial force acting in the plane of the diaphragm and the force obtained from dividing the factored moment at the section by the distance between the edges of the diaphragm at that section.

e) Transverse reinforcement in walls with boundary members shall be anchored within the confined core of the boundary member to develop the tensile yield stress.

f) Transverse 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 the U-stirrups having the same size and spacing as, and spliced to, the transverse reinforcement, except when V_u in the plane of the wall is less than $0.083A_{cv}\sqrt{f'_c}$.

8.3.6.4 Construction Joints : All construction joints in walls and diaphragms shall conform to Sec 5.16.4 and contact surfaces shall be roughened as specified in Sec 6.13.3.15(j).

8.3.6.5 Discontinuous Walls : Columns supporting discontinuous walls shall be reinforced in accordance with Sec 8.3.5.4(e).

8.3.7 **Joints of 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 1.25 f_y in the reinforcement.

b) Joint strength shall be calculated by the appropriate strength reduction factors specified in Sec 6.1.4.

c) Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored in tension according to Sec 8.3.7.4 below and in compression according to Sec 8.2.

8.3.7.2 Transverse Reinforcement

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.

8.3.7.3 Shear Strength : The nominal shear strength for the joint shall be taken not greater than the forces specified below :

$1.66\sqrt{f'_c}A_j$	for joints confined on all four faces
$1.24\sqrt{f'_c}A_j$	for joints confined on three faces or on two opposite faces
$1.0\sqrt{f'_c}A_j$	for others

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 a bar with a standard 90° hook shall be not less than (i) $8d_b$, (ii) 150 mm, and (iii) the length required by Eq (8.3.5).

$$\ell_{dh} = \frac{0.185 f_y d_b}{\sqrt{f'_c}} \qquad (8.3.5)$$

for bar sizes 10 mm ϕ through 35 mm ϕ.

b) For bar sizes 10 mm ϕ through 35 mm ϕ, 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 statical forces on the portion of the member between faces of the joints. 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 Sec 8.3.4.3, 8.3.5.4 and 8.3.7.2.

8.3.8.3 Shear Strength of 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

$$V_n = A_{cv}\left(0.17\sqrt{f_c'} + \rho_n f_y\right) \tag{8.3.6}$$

c) For walls and wall segments having a ratio of (h_w/ℓ_w) less than 2.0, nominal shear strength of wall and diaphragm shall be determined from

$$V_n = A_{cv}\left(\alpha_c\sqrt{f_c'} + \rho_n f_y\right) \tag{8.3.7}$$

where the coefficient α_c varies linearly from 0.25 for $(h_w/\ell_w)=1.5$ to 0.17 for $(h_w/\ell_w)=2.0$.

d) Value of ratio (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 (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 **Frame Members not Proportioned to Resist Forces Induced by Earthquake Motion**

8.3.9.1 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 $0.1A_g f_c'$ shall satisfy Sec 8.3.4.2(a) and 8.3.8.1(a) and members with factored gravity axial forces exceeding $0.1A_g f_c'$ shall satisfy Sec 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 All frame members with factored axial compressive forces exceeding $0.1A_g f_c'$ 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_o over a length ℓ_o measured from the joint face. The spacing s_o 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_o 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 $0.5s_o$ from the face of the joint.

d) The tie spacing shall not exceed $2s_o$ in any part of the column.

8.3.10 **Requirements for Frames in Regions of Moderate Seismic Risk, Zone 2**

8.3.10.1 In regions of moderate seismic risk, 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 details in a frame member shall satisfy Sec 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 (6.3.3). 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 Design shear strength of beams, columns, and two-way slabs resisting earthquake effect shall not be less than either (a) the sum of the shear associated with development of nominal moment strengths of the member at each restrained end of the clear span and the shear calculated for factored gravity loads, or (b) the maximum shear obtained from design load combinations which include earthquake effect.

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. 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. 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) $d/4$, (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 $d/2$ throughout the length of the member.

8.3.10.5 Columns

a) Maximum tie spacing shall not exceed s_o over a length ℓ_o measured from the joint face. The spacing s_o 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_o 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 $s_o/2$ from the joint face.

c) Joint reinforcement shall conform to Sec 6.3.8.

d) Tie spacing shall not exceed $2s_o$ throughout the length of the member.

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 6.4.2.2.

b) The fractional part of the column strip moment shall be resisted by reinforcement placed within the effective width specified in Sec 6.4.4.5(b).

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 specified in Sec 6.4.4.5(b).

d) Not less than one-quarter of the top steel at the support in the column strip shall be continuous throughout the span.

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.

g) At discontinuous edges of the slab all top and bottom reinforcement at the support shall be developed at the face of the support.

Related Appendix

Appendix A	Conversion of Expressions from SI to FPS Units

Table 6.4.16
Band Reinforcement

Seismic Zones	Reinforcement		Links
	Plain Mild Steel Bars	High Strength Deformed Bars	
2	2 - 12 mm ϕ, one on each face of the wall with suitable cover	2 - 10 mm ϕ, one on each face of the wall with suitable cover	6 mm dia, 150 mm c/c
3	2 - 16 mm ϕ, one on each face of the wall with suitable cover	2 - 12 mm ϕ, one on each face of the wall with suitable cover	6 mm dia, 150 mm c/c

4.8.6.4 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 6.4.17. 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 Fig 6.4.1 and Fig 6.4.2.

b) Details of vertical reinforcement given in Table 6.4.17 are applicable to brick masonry and hollow block masonry.

Table 6.4.17
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		Diameter of Single Bar or Equivalent Area of High Strength Deformed Bar to be Provided	
		Zone 2 (mm)	Zone 3 (mm)	Zone 2 (mm)	Zone 3 (mm)
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

4.8.6.5 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 Fig 6.4.3.

4.8.6.6 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.

4.9 PROVISIONS FOR HIGH WIND REGIONS

4.9.1 **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.

4.9.2 **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 :

ii) If the loaded flange is not restrained against rotation and $\left(d_c/t_w\right)/\left(l/b_f\right)$ is less than 1.7 :

$$R_n = \frac{83t_w^3}{h}\left[0.4\left(\frac{d_c/t_w}{l/b_f}\right)^3\right] \qquad (10.8.133)$$

where

$d_c = d - 2k$ = web depth clear of fillets, mm.

Eq (10.8.132) and (10.8.133) need not be checked provided $\left(d_c/t_w\right)/\left(l/b_f\right)$ exceeds 2.3 or 1.7 respectively, or for webs subject to distributed load.

If a concentrated load is located at a point where the web flexural stress due to factored loads is below yielding, 165 shall be used in lieu of 83 in Eq (10.8.132) and (10.8.133).

f) Compression Buckling of the Web : For unstiffened portions of webs of members under concentrated loads to both flanges, the design compressive strength shall be ϕR_n, where

ϕ = 0.90

$$R_n = \frac{10.76t_w^3\sqrt{F_{yw}}}{d_c} \qquad (10.8.134)$$

R_n may be exceeded provided that a transverse stiffener or pair of stiffeners is attached to the web to satisfy Sec 10.8.6.3.

g) Compression Members with Web Panels Subjected to High Shears : For compression members subjected to high shear stress in the web, the design web shear strength shall be ϕR_v, where ϕ =0.90 and R_v is determined as follows :

i) For $P_u \leq 0.75P_n$:

$$R_v = 0.0007F_y d_c t_w \qquad (10.8.135)$$

ii) For $P_u > 0.75P_n$:

$$R_v = 0.0007F_y d_c t_w\left[1.9 - 1.2\left(P_u/P_n\right)\right] \qquad (10.8.136)$$

h) Stiffener Requirements for Concentrated Loads : When required, stiffeners shall be placed in pairs at unframed ends of beams and girders. They shall be placed in pairs at points of concentrated load on the interior of beams, girders or column if the load exceeds the nominal strength ϕR_n as determined from (b) through (f) above as applicable.

If the concentrated load, tension or compression exceeds the criteria for ϕR_n of (b) or (c) above respectively, stiffeners need not be extended more than one-half the depth of the web, except as follows :

If concentrated compressive loads are applied to the members and if the load exceeds the compressive strength of the web ϕR_n given in (d) or (f) above, the stiffener shall be designed as axially compressed members (columns) in accordance with the requirements of Sec 10.8.5.2 with an effective length equal to $0.75h$, a cross-section composed of two stiffeners and a strip of the web having a width of $25t_w$ at interior stiffeners and $12t_w$ at the ends of members.

When the load normal to the flange is tensile, the stiffeners shall be welded to the loaded flange. When the load normal to the flange is compressive, the stiffeners shall either bear on or be welded to the loaded flange.

10.8.11.2 Ponding : The roof system shall be investigated for ponding in accordance with the provisions of Sec 10.7.11.2.

10.8.11.3 Torsion : For limiting values of normal and shear stresses, due to torsion and other loading, the provisions of Sec 10.8.8.2 shall apply. Some constrained local yielding may be permitted.

10.8.12 **Seismic Design Provisions**

10.8.12.1 Scope : This section specifies special seismic design provisions for the design and construction of structural steel members and connections in buildings for which the design forces resulting from earthquake motions have been determined on the basis of energy dissipation in the nonlinear range of response. These special seismic requirements are to be applied in conjunction with the Load Factor Design method. The special seismic design provisions apply only to buildings in Zone 3 and to buildings in Zone 2 having an importance factor I greater than 1.0.

10.8.12.2 Seismic Zoning : Seismic zoning provisions shall be as stipulated in Chapter 2, Loads.

10.8.12.3 Loads and Load Combinations : The design loads shall be the minimum factored loads and their combinations specified in Chapter 2, Loads.

10.8.12.4 Storey Drift and Building Separations : Storey drift shall be calculated using the appropriate effects consistent with the structural system and the method of analysis. Limits on storey drift shall be in accordance with Sec 1.5.6, Chapter 1, General Design Requirements.

10.8.12.5 Materials : Steel used in seismic force resisting systems shall be as listed in Sec 10.3.1. For buildings over one storey in height, the steel used in seismic resisting systems described in Sec 10.8.14.7, 10.8.14.8 and 10.8.14.9 shall be limited to the following ASTM Specifications : A36, A441, A500 (Grades B and C), A501, A572 (Grades 42 and 50), and A588. For base plates, ASTM A283 Grade D can be used in lieu of the other plate materials listed above.

10.8.12.6 Column Requirements

a) Columns in earthquake resisting frames shall comply with the requirements of Sec 10.8.5 and 10.8.8. When $P_u/\phi P_n > 0.5$ column axial design strength shall be limited by the following requirements.

i) Axial compression loads :

$$1.2P_D + 0.5P_L + 1.0P_{E'} \le \phi P_n \qquad (10.8.137)$$

Exception :
The load factor on L used to determine P_L in Eq (10.8.137) shall be equal to 1.0 for garages, areas occupied as places of public assembly and all areas where the live load is greater than 5.0 kN/m^2.

ii) Axial tension loads :

$$0.9P_D + 1.0P_{E'} \le \phi P_n \qquad (10.8.138)$$

iii) The required axial strengths in Eq (10.8.137) and (10.8.138) need not exceed either of the following:

A. The maximum loads that can be transferred to the column, considering 1.25 times the design strengths of the connecting beam or brace elements of the structure.

B. The limit as determined by the foundation capacity to resist overturning uplift.

b) Column Splices : Column splices shall have sufficient strength to develop the column axial loads given in (i), (ii) and (iii) above as well as the load combinations specified in Sec 2.7.5.2 of Chapter 2, Loads.

i) In column splices using either complete or partial penetration welded joints, changes in thickness and width of flanges and webs are permitted without providing bevelled transitions.

ii) Splices using partial penetration welded joints shall not be within 900 mm of the beam to column connection. Column splices that are subjected to net tension forces shall comply with the more critical of the following:

A. The lesser design strength of $\phi_w F_w A_w$ or $\phi_w F_{BM} A_w$ (see Sec 10.9.2.4) for partial penetration welded joints shall be at least 150 per cent of the required strength.

B. The design strength of welds shall not be less than $0.5F_{yc}A_f$ where F_{yc} is the yield strength of the column material and A_f is the flange area of the smaller column connected.

10.8.12.7 Requirements for Ordinary Moment Frames (OMF)

a) Design Strength : Ordinary moment frames (OMF), where permitted according to the provisions of this Code, shall have the design strength to resist the load combinations specified in Sec 2.7.5.2 of Chapter 2, Loads. The design strength of such members shall be determined using the LFD method.

b) Joint Requirements : All beam to column connections in OMF which resist earthquake forces shall meet one of the following requirements :

i) FR (fully restrained) connections conforming with Sec 10.8.12.8(b).

ii) FR connections with design strengths of the connections meeting the requirements of (a) above using the load combinations 7 and 8 specified in Sec 2.7.5.2 of Chapter 2, Loads.

iii) Either FR or PR (partially restrained) connections are permitted provided :

A. The design strengths of the members and connections meet the requirements of (a) above.

B. The connections have been demonstrated by cyclic tests to have adequate rotation capacity at a storey drift calculated at a horizontal load of E′.

C. The additional drift due to PR connections shall be considered in design.

FR and PR connections are described in detail in Sec 10.4.

10.8.12.8 Requirements for Special Moment Frames (SMF)

a) Scope : Special moment frames (SMF), shall be used when required by the provisions of this Code. For buildings in Seismic Zone 2 having an importance factor I greater than 1.0, only the requirements of (b), (c), (g), and (h) below shall be applicable.

b) Beam to Column Joints

i) The required flexural strength M_u of each beam to column joint shall be the lesser of the following quantities :

A. The plastic bending moment M_p of the beam.

B. The moment resulting from the panel zone nominal shear strength V_n as determined using Eq (10.8.139).

The joint need not develop either of the strengths defined above if it can be shown that under an amplified frame deformation of E′/E times that produced by load combinations 5 and 6 specified in Sec 2.7.5.2 of Chapter 2, Loads, the design strength of the members at the connection are adequate to support the vertical loads, and the required lateral force resistance is provided by other means.

ii) The required shear strength V_u of a beam to column joint shall be determined using the load combination $1.2D+0.5L$ plus the shear resulting from M_u, as defined in (i) above, on at least one end of the beam. The required shear strength need not exceed the shear resulting from the load combination 7 specified in Sec 2.7.5.2 of Chapter 2, Loads.

iii) The design strength ϕR_n of a beam to column joint can be considered adequate to develop the required flexural strength M_u of the beam if it conforms to the following :

A. The beam flanges are welded to the column using complete penetration welded joints.

B. The beam web joint shall have a design shear strength ϕV_n greater than the required shear V_u and conform to either :

1. Where the nominal flexural strength of the beam M_n considering only the flanges is greater than 70 per cent of the nominal flexural strength of the entire beam section i.e., $\left[10^{-6} b_f t_f \left(d - t_f\right) F_{yf} \geq 0.7 M_p\right]$, the web joint may be made by means of welding or slip-critical high strength bolting.

2. Where a slip-critical high strength bolted joint in a beam does not meet the flexural criteria in (i) above the required strength of added web welding shall be at least 20 per cent of the nominal flexural strength of the beam web. The required beam shear strength shall be developed by further welding or by slip-critical high strength bolting.

iv) Alternate joint configurations : Joint configurations utilizing welds or high strength bolts, but not conforming to (iii) above, may be used if shown by test or calculations to meet the criteria of (i) above. Where conformance is shown by calculation, the design strength of the joint shall be 125 per cent of the design strengths of the connecting elements.

c) Panel Zone of Beam to Column Connections (Beam Web Parallel to Column Web)

i) Shear strength : The required shear strength V_u of the panel zone shall be based on the beam bending moments determined from the load combinations 5 and 6 specified in Sec 2.7.5.2 of Chapter 2, Loads. V_u need not exceed that determined from $0.9\sum\phi_b M_p$ of the beams framing into the column flanges at the connection. The design shear strength $\phi_v V_n$ of the panel zone shall be determined by the following formula :

$$\phi_v V_n = 0.55\times10^{-3}\,\phi_v F_y d_c t_p\left[1+\frac{3b_{cf}t_{cf}^2}{d_b d_c t_p}\right],\ \text{where for this case } \phi_v = 0.8 \qquad (10.8.139)$$

ii) Panel zone thickness : The panel zone thickness t_z shall conform to the following :

$$t_z \geq (d_z + w_z)/90 \qquad (10.8.140)$$

For this purpose t_z shall not include any doubler plate thickness unless the doubler plate is connected to the web with plug welds adequate to prevent local buckling of the plate.

iii) Panel zone doubler plates : Doubler plates provided to increase the design strength of the panel zone or to reduce the web depth-thickness ratio shall be placed close to the column web and welded across the plate width top and bottom with a minimum fillet weld as specified in Table (6.10.9). The doubler plates shall be fastened to the column flanges using either butt or fillet welded joints to develop the design shear strength of the doubler plate.

d) Beam Limitations

i) Beam Flange Area : Abrupt changes in beam flange areas are not permitted within possible plastic hinge regions.

ii) Width-thickness Ratios : Beams shall comply with the limiting width thickness ratio stipulated in Table 6.10.5 in lieu of those in Table 6.10.4.

e) Continuity Plates : Continuity plates shall be provided if the nominal column local flange bending strength as given by Eq (10.8.127) is less than $1.8\times10^{-3}F_{yb}b_f t_{bf}$. Continuity plates shall be fastened by welds to both the flanges and webs or doubler plates of columns.

f) Column-Beam Moment Ratio : At any beam to column connection, one of the following relationships shall be satisfied :

Table 6.10.5
Limiting Width-Thickness Ratios for Compression Elements

Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratios
Flanges of I-shaped nonhybrid sections and channels in flexure Flanges of I-shaped hybrid beams in flexure	b/t	$136/\sqrt{F_y}$
Webs in combined flexural and axial compression	h_c/t_w	For $P_u/\phi_b P_y \leq 0.125$ $\frac{1365}{\sqrt{F_y}}\left[1-\frac{1.54P_u}{\phi_b P_y}\right]$
		For $P_u/\phi_b P_y > 0.125$ $\frac{500}{\sqrt{F_y}}\left[2.33-\frac{P_u}{\phi_b P_y}\right] \geq \frac{665}{\sqrt{F_y}}$

$$\frac{\sum Z_c\left(F_{yc}-P_{uc}/A_g\right)}{\sum Z_b F_{yb}} \geq 1.0, \text{ where } P_{uc} \text{ (in compression)} \geq 0 \qquad (10.8.141)$$

$$\frac{\sum Z_c\left(F_{yc}-P_{uc}/A_g\right)}{1000 V_n d_b\left(H/\left[H-d_b\right]\right)} \geq 1.0, \text{ where } P_{uc} \text{ (in compression)} \geq 0 \qquad (10.8.142)$$

In Eq (10.8.142), V_n is the nominal strength of the panel zone as determined from Eq (10.8.139), d_b is the average overall depth of the beams framing into the connection, and H is the average of the storey heights above and below the connection.

These requirements need not apply in the following cases, provided that the columns conform to the requirements of (d) above:

i) For columns with $P_{uc} < 0.3 \times 10^{-3} F_y A_g$.

ii) For columns in any storey that has a total design lateral shear strength 50 per cent greater than that of the storey above.

iii) For any column not included in the design to resist the required seismic shears, although the column is included in the design to resist axial overturning forces.

g) Beam to Column Connection Restraint

i) Restrained Connections :

A. Column flanges at a beam to column connection require lateral support only at the level of the top flanges of the beams when a column can be shown to remain outside of the panel zone. This can be satisfied by one of the following conditions :

1. The ratios given by Eq (10.8.141) or (10.8.142) are greater than 1.25.

2. The column remains elastic for load combination 7 specified in Sec 2.7.5.2 of Chapter 2, Loads.

B. When a column cannot be shown to remain elastic outside of the panel zone, the following provisions apply :

1. The column flanges shall be laterally supported at the levels of both top and bottom beam flanges.

2. Each column flange lateral support shall be designed for a required strength equal to 1.5 per cent of the nominal beam flange strength $\left(F_y b_f t_f\right)$.

3. Column flanges shall be laterally supported either directly or indirectly by means of the column web or beam flanges.

ii) Unrestrained Connections : A column containing a beam to column connection with no lateral support transverse to the seismic frame at the connection shall be designed using the distance between adjacent lateral supports as the column height and conform to Sec 10.8.8.1, except that :

A. The required column strength shall be determined from the load combination 5 specified in Sec 2.7.5.2 of Chapter 2, where E is the least of :

1. The amplified earthquake force E′.
2. 125 per cent of the frame design strength based on either beam or panel zone design strengths.

B. The nominal column axial strength P_n shall be based on a pin ended column.

C. The *L/r* for these columns shall not exceed 60.

D. The required column moment M_{uy} shall include that caused by the beam flange force specified in i(B-2) above plus the added P-Delta moment due to the resulting column flange displacement.

h) Lateral Support of Beams : Both flanges of beams shall be laterally supported directly or indirectly. The unbraced length between lateral supports shall not exceed $17237 r_y / F_y$. In addition, lateral supports shall be placed at concentrated loads where a hinge may form.

10.8.12.9 Requirements for Concentrically Braced Frames (CBF)

a) Scope : The provisions of this section apply to all braced frames except Eccentric Braced Frames (EBF) designed in accordance with Sec 10.8.12.10. Those members which resist seismic forces totally or partially by shear and flexure shall be designed in accordance with Sec 10.8.12.8.

b) Bracing Members

i) Slenderness : Bracing members shall have an $L/r \le 1890/\sqrt{F_y}$ in Seismic Zone 3 except as permitted in (e) below.

ii) Compressive Design Strength : The design strength of a bracing member in axial compression shall be determined by $0.8\phi_c P_n$.

iii) Lateral Force Distribution : In Seismic Zone 3, the summation of the horizontal components of lateral seismic force in tension or alternatively in compression along any line of bracing shall not exceed 70% of the total force on that line, unless the nominal strength P_n of the member in compression is larger than the required strength P_u resulting from the application of the load combinations 7 and 8 specified in Sec 2.7.5.2 of Chapter 2, Loads. A line of bracing, for the purpose of this provision, is defined as a single line or parallel lines whose offset is within 10 per cent of the building dimension perpendicular to the line of bracing.

iv) Width-thickness Ratios : Width-thickness ratios of stiffened and unstiffened compression elements in braces shall comply with Sec 10.8.3. Braces shall be compact or noncompact, but not slender members. Circular hollow sections shall have an outside diameter to wall thickness ratio not exceeding $8965/F_y$. Rectangular tubes shall have the flat width to wall thickness not exceeding $290/\sqrt{F_y}$, unless the tube walls are stiffened.

v) Built-up Member Stitches : For all built-up braces, the first bolted or welded stitch on each side of the mid-length of a built-up member shall be designed to transmit a force equal to 50% of the nominal strength of one element to the adjacent element. Not less than two stitches shall be equally spaced about the member centre line.

c) Bracing Connections

i) Forces : The required strength of bracing joints (including beam to column joints if part of the bracing system) shall be the least of the following :

A. The design axial tension strength of the bracing member.

B. The force in the brace resulting from the load combinations 7 and 8 specified in Sec 2.7.5.2 of Chapter 2, Loads.

C. The maximum force that can be transferred to the brace by the system.

ii) Net Area : In bolted brace joints, the minimum ratio of effective net section area to gross section area shall be limited by :

$$A_e / A_g = \frac{1.2\alpha P_u^*}{\phi_t P_n}$$

where

α = Fraction of the member force from (i) above, that is transferred across a particular net section.

P_u^* = Required axial strength of the brace as determined in (i) above.

ϕ_t = Resistance factor for tension = 0.75.

iii) Gusset Plates

A. For braces that can buckle in the plane of the gusset plate, the gusset and other parts of the connection shall have a design strength equal to or greater than the nominal in-plane bending strength of the brace.

B. For braces which can buckle out-of-plane of the gusset plate, the brace shall terminate on the gusset a minimum of two times the gusset thickness from a line about which the gusset plate can bend unrestrained by the column or beam joints. The gusset plate shall be designed to carry the compressive design strength of the brace member without local buckling of the gusset plate. For braces designed for axial load only, the bolts or welds shall be designed to transmit the brace forces along the centroids of the brace elements.

d) Special Bracing Configuration Requirements

i) V Bracing

A. The design strength of V brace members shall be at least 1.5 times the required strength using load combinations 5 and 6 specified in Sec 2.7.5.2 of Chapter 2, Loads.

B. A beam intersected by V braces shall be continuous between columns.

C. A beam intersected by V braces shall be capable of supporting all tributary dead and live loads assuming the bracing is not present.

D. The top and bottom flanges of the beam at the point of intersection of V braces shall be designed to support a lateral force equal to 1.5 per cent of the nominal beam flange strength $(F_y b_f t_f)$.

ii) K Bracing

A. In Seismic Zone 3, K bracing shall be prohibited except for those framing systems meeting the requirements of (e) below.

B. In Seismic Zone 2, with importance factor I greater than 1.0, K bracing shall meet the requirements for V bracing.

e) Low Buildings : Braced frames not meeting the requirements of (b) through (d) above may be used in buildings not over two storeys and in roof structures if load combinations 7 and 8 specified in Sec 2.7.5.2 of Chapter 2, Loads, are used for determining the required strength of the members and connections.

10.8.12.10 Requirements for Eccentrically Braced Frames (EBF)

a) Scope : Eccentrically braced frames shall be designed so that under earthquake loading, yielding will occur primarily in the links. The diagonal braces, the columns, and the beam segments outside of the links shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain hardened links. In EBF, plastic hinges shall not develop in columns at floor beam levels up to an amplified frame displacement of E'/E times that produced by E.

b) Links

i) Links shall comply with the width-thickness ratios in Table 6.10.5.

ii) The specified minimum yield stress of steel used for links shall not exceed $F_y = 345$ N/mm^2.

iii) The web of a link shall be single thickness without doubler plate reinforcement and without openings.

iv) The required shear strength of the link V_u shall not exceed the design shear strength of the link $\phi_v V_n$ defined as the lesser of $\phi_v V_y$ or $2000\phi_b M_p/e$, where $V_y = 0.6\times10^{-3} F_y d t_w$, $\phi_b = \phi_v = 0.9$ and e = link length, except as limited by (vi) below.

v) If the required axial strength P_u in a link is less than or equal to $0.15P_y$, where $P_y = 10^{-3} A_g F_y$, the effect of axial force on the link design shear strength need not be considered.

vi) If the required axial strength P_u in a link exceeds $0.15P_y$, the following additional limitations shall apply :

A. The link design shear strength shall be the lesser of $\phi_v V_{ya}$ or $2000\phi_b M_{pa}/e$,

where

$$V_{ya} = V_y\sqrt{1-\left(P_u/P_y\right)^2}$$

$$M_{pa} = 1.18 M_p \left[1 - \left(P_u / P_y\right)\right]$$

and $\phi_b = \phi_v = 0.9$

B. The length of the link in mm, shall not exceed :

$\left[1.15 - 0.5\left(P_u/V_y\right)\left(A_w/A_g\right)\right] 1600 M_p/V_y$ for $\left(P_u/V_y\right)\left(A_w/A_g\right) \geq 0.3$ and

$1600 M_p/V_y$ *for* $\left(P_u/V_y\right)\left(A_w/A_g\right) < 0.3$

where $A_w = dt_w$.

vii) The link rotation angle is the plastic angle between the link and the beam outside of the link when the total storey drift is E'/E times the drift determined using the specified base shear V. Except as noted in d(iii) below, the link rotation angle shall not exceed the following values :

A. 0.09 radian for links of length $1600 M_p/V_y$ or less, provided the fundamental period of the EBF is equal to or greater than 1.0 second; otherwise the link rotation angle shall not exceed 0.08 radian.

B. 0.02 radian for links of length $2600 M_p/V_y$ or greater.

C. Linear interpolation shall be used for links of length between $1600 M_p/V_y$ and $2600 M_p/V_y$.

c) Link Stiffeners

i) Full depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $\left(b_f - 2t_w\right)$ and a thickness not less than 0.75 t_w nor 10 mm, whichever is larger, where b_f and t_w are the link flange width and link web thickness, respectively.

ii) Links shall be provided with intermediate web stiffeners as follows :

A. Links of lengths $1600 M_p/V_y$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - d/5)$ for a link rotation angle of 0.09 radian or $(52t_w - d/5)$ for link rotation angles of 0.03 radian or less. Linear interpolation shall be used for values between 0.03 and 0.09 radians.

B. Links of length greater than $2600 M_p/V_y$ and less than $5000 M_p/V_y$ shall be provided with intermediate web stiffeners placed at a distance of $1.5 b_f$ from each end of the link.

C. Links of length between $1600 M_p/V_y$ and $2600 M_p/V_y$ shall be provided with intermediate web stiffeners meeting the requirements of (A) and (B) above.

D. No intermediate web stiffeners are required in links of lengths greater than $5000 M_p/V_y$.

E. Intermediate link web stiffeners shall be full depth. For links less than 625 mm in depth, stiffeners are required on only one side of the link web. The thickness of one sided stiffeners shall not be less than t_w nor 10 mm, and the width shall be not less than $\left[\left(b_f/2\right) - t_w\right]$. For links 625 mm in depth or greater, similar intermediate stiffeners are required on both sides of the web.

iii) Fillet welds connecting link stiffener to the link web shall have a design strength adequate to resist a force of $A_{st}F_y$, where A_{st} equals the area of the stiffener. The design strength of fillet welds fastening the stiffener to the flanges shall be adequate to resist a force of $A_{st}F_y/4$.

10.9.1.3 Moment Connections : End connections of restrained beams, girders and trusses shall be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connections.

10.9.1.4 Compression Members with Bearing Joints : When columns bear on bearing plates or are finished to bear at splices, there shall be sufficient connectors to hold all parts securely in place.

When other compression members are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be designed for 50% of the strength of the member in Working Stress Design and for 50% of the factored strength of the member in Load Factor Design.

In Working Stress Design compression joints shall be proportioned to resist any tension developed by the specified lateral loads acting in conjunction with 75% of the calculated dead load stress and no live loads.

In Load Factor Design compression joints shall be proportioned to resist any tension developed by the factored load combination 6 specified in Sec 2.7.5.2 of Chapter 2, Loads.

10.9.1.5 Minimum Connections : Except for lacing, sagbars and girts, all connections shall be proportioned to support a service load not less than 27 kN in Working Stress Design while in Load Factor Design the connection shall be proportioned to support a factored load not less than 45 kN.

10.9.1.6 Splices in Heavy Sections : The following requirement applies to ASTM A6 Group 4 and 5 rolled shapes, or shapes built up by welding plates more than 50 mm thick together to form the cross-section, and where the cross-section is to be spliced and subjected to primary tensile stresses due to tension or flexure. When the individual elements of the cross section are spliced prior to being joined to form cross-section in accordance with Sec 3.4.6 of AWS D1.1, the applicable provisions of AWS D1.1 shall apply in lieu of the requirements of this section.

When tensile forces in these sections are to be transmitted through splices by full penetration groove welds, weld access hole details as given in Sec 10.9.1.7, welding preheat requirements as given in Sec 10.9.2.6 and thermal cut surface preparation and inspection requirements as given in Sec 10.11.2.2 shall be applicable.

At tension splices in these sections, weld tabs and backing shall be removed and the surfaces ground smooth.

When splicing these sections, and where the section is to be used as a primary compression member, all weld access holes required to facilitate groove welding operations shall satisfy the requirements of Sec 10.9.1.7.

Alternatively, splicing of such members subjected to compression, including members which are subjected to tension due to wind or seismic loads, may be accomplished using splice details which do not induce large weld shrinkage strains such as partial-penetration flange groove welds with fillet-welded surface lap plate splices on the web, or with bolted or combination of bolted and fillet welded lap plate splices.

10.9.1.7 Beam Copes and Weld Access Holes : All weld access holes required to facilitate welding operations shall have a length from the toe of the weld preparation not less than $1\frac{1}{2}$ times the thickness of the material in which the hole is made. The height of the access hole shall be adequate for deposition of sound weld metal in the adjacent plates and provide clearance for weld tabs. In hot rolled shapes and built-up shapes, all beam copes and weld access holes shall be shaped free of notches or sharp reentrant corners except that, when fillet web to flange welds are used in built-up shapes, access holes are permitted to terminate perpendicular to the flange.

For Group 4 and 5 shapes and built-up shapes of material more than 50 mm thick, the thermally cut surfaces of beam copes and weld access holes shall be ground to bright metal and inspected by either magnetic particle or dye penetrant methods. If the curved transition portion of weld access holes and beam copes are formed by predrilled or sawed holes, that portion of the access hole or cope need not be ground. Weld access holes and beam copes in other shapes need not be ground nor inspected by dye penetrant or magnetic particle.

10.9.1.8 Placement of Welds, Bolts and Rivets : Groups of welds, bolts or rivets at the ends of any member which transmit axial stress into that member shall be sized so that the centre of gravity of the group coincides with the centre of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of statically loaded single-angle, double-angle and similar members. Eccentricity between the gravity axes of such members and the gauge lines for their riveted or bolted end connections may be neglected in statically loaded members.

10.9.1.9 Bolts in Combination with Welds : In new work, A307 bolts or high strength bolts used in bearing type connections shall not be considered as sharing the stress in combination with welds. Welds, if used, shall be provided to carry the entire stress in the connection. High strength bolts designed for slip-critical connections may be considered as sharing the stress with the welds.