Government of NWFP Pakistan

SARHAD INTERIM SEISMIC BUILDING CODE

(SISBC – 02)

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(Version 02)

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SARHAD INTERIM SEISMIC BUILDING CODE

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Sarhad Interim Seismic Building Code

Document No. SISBC – 02

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Chapter 1 Introduction

General

It was on October 08, 2005, when one of the deadliest earthquakes of the sub-continent struck the Northern areas of Pakistan and parts of Pakistan administered Kashmir leaving behind more than 73,000 people killed, 69,000 injured, and 3.5 million homeless. The financial losses were estimated to exceed USD 5 billion.

This is a human tragedy of enormous dimensions that the poorest people in the least developed countries are hardest hit by natural disasters. The two earthquakes in California on 22nd December 2003 and in the Iranian city of Bam only four days later illustrate this very clearly. Measured on the Richter scale, the two earthquakes were of roughly the same strength – but the quake in Iran killed more than 26 000 people. In California, only two people died.

It is a well known fact that these are buildings and not the earthquakes that kill people. Therefore the devastating consequences of the earthquakes can be mitigated, if buildings are designed and constructed properly. Engineers play a significant role in achieving this goal because they are the persons responsible for developing technical regulations for the design and construction of earthquake resistant structures which can ensure safety of the occupants.

It was in this context when the Government of North-West Frontier Province (N-W.F.P) of Pakistan requested the Earthquake Engineering Center (EEC) at N-WFP University of Engineering & Technology (UET) Peshawar for technical assistance and guidance in developing a seismic design code for the earthquake-affected areas of N-W.F.P, both in the form of an enforceable set of rules for engineered structures and guidelines for vernacular buildings.

This document, named as “Sarhad Interim Seismic Building Code (SISBC)”, is actually a part of the set of six documents and instruction manuals and primarily addresses the earthquake-resistant design of engineered structures. Non-engineered buildings are dealt separately.

Following are the other documents:

1. **Construction Materials**

   This document contains standard specifications of materials to be used in the construction of buildings in the earthquake-affected areas. It is worth mentioning that conforming to provisions of Construction Materials document is mandatory, failing which, would not assure achievement of desired performance level as required and mentioned in various parts of SISBC.

2. **Modular Design Report**

   This document contains architectural and structural designs for model schools, basic health units, rural health centers, and residential buildings to be constructed in the earthquake-affected areas. These structures have been specially designed and detailed to resist seismic forces.

3. **Field Practicing Manual**

   Field Practicing Manual has been prepared in the Urdu language for providing guidelines to the skilled, semi-skilled and un-skilled persons working in the construction sector. The tips and methods presented in this manual are meant to convey good engineering practices in the simplest possible manner for common people lacking proper training. However, by no means the intent of this manual is to overrule, supersede, change or lower the standards given in SISBC.

4. **Repair and Retrofitting Manual**

   This manual contains illustrated guidelines on repair and retrofitting of the existing buildings. In no case shall this manual be treated as substitute for detailed testing of structures using specialized equipment of Non-destructive Evaluation (NDE).

5. **Modalities and Methodologies for Enforcement of SISBC**

   Code implementation, its updating, revisions and validating its conformity in built structures is daunting tasks and requires extensive planning, monitoring and evaluation at various hierarchal levels of government. This document named as “Modalities and Methodologies for Enforcement of Code” covers details which can help the regulatory authorities effectively implement the Sarhad Interim Seismic Building Code.

It is pertinent to mention that due to the absence of sufficient relevant research data and the urgency of the situation, the main theme and the body of SISBC have been taken from the already existing codes. Nevertheless, keeping in view the local construction practices, several fundamental changes and modifications have been introduced and incorporated in the code. The research work carried out at N-WFP UET in the last several years helped in transforming some of these basic design parameters.

It is acknowledged with gratitude that the following building codes have been referred to in the development of SISBC.

1. 1997 Uniform Building Code published by International Conference of Building Officials, Whittier, California, USA.

2. NEHRP 2003 Recommended Provisions for Seismic Regulations for New Buildings and other Structures, prepared by Building Seismic Safety Council, Washington D.C., USA.

3. EN1998-1 Eurocode 8: Design of Structures for Earthquake Resistance, prepared by European Committee for Standardization, Brussels, Belgium.

4. Italian Seismic Code, OPTCM 3274.

5. Specification for Structures to be built in Disaster Areas, prepared by the Ministry of Public Works and Settlement, Government of Turkey.

6. Indian Seismic Code IS 1893, prepared by Indian Institute of Technology, Kanpur, India.

Scope

The Sarhad Interim Seismic Building Code is intended to be used for earthquake-resistant design of reinforced concrete, brick and concrete block masonry structures in the affected areas.

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Chapter 2 Terms & Definitions

2.0 General

This chapter covers the definitions of basic terms relating to both concrete and masonry work.

2.1 Definitions

1. **Base**
   Base is the level at which the earthquake ground motions are considered to be imparted to the structure or the level at which the structure as a dynamic vibrator is supported.

2. **Base Shear, V**
   Base shear, V is the total design lateral force or shear at the base of a structure.

3. **Bed Joint Reinforcement**
   Reinforcing steel that is prefabricated for building into a bed joint.

4. **Boundary Elements (Or Zones)**
   Boundary Elements (Or Zones) are portions along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement. Boundary elements do not necessarily require an increase in the thickness of the wall or diaphragm. Edges of openings within walls and diaphragms shall be provided with boundary elements if required by Section 5.4.5.1.

5. **Collector Elements**
   Collector Elements are elements that serve to transmit the inertial forces with the diaphragms to members of the lateral-force-resisting systems.

6. **Compressive Strength of Masonry**
   The strength of masonry in compression without the effects of platen restraint, slenderness or eccentricity of loading.

7. **Confined masonry**
   Masonry provided with reinforced concrete or reinforced masonry confining elements in the vertical and horizontal directions.

8. **Coupling Beams**
   Coupling Beams are a horizontal element in plane with and connec-

ting two shear walls.

9. Crosstie

Crosstie is a continuous reinforcing bar having a seismic hook at one end and a hook of not less than 90-degrees with at least six diameters at the other end. The hooks shall engage peripheral longitudinal bars. The 90-degree hooks of two successive crossties engaging the same longitudinal bar shall be alternated end for end.

10. Dead Loads

Dead loads consist of the weight of all materials and fixed equipment incorporated into the building or other structure.

11. Design Basis Ground Motion:

Design Basis Ground Motion is that ground motion that has a 10% chance of being exceeded in 50 years as determined by a site-specific hazard analysis or may be determined from a hazard map. A suite of ground motion time histories with the dynamic properties of the site characteristics shall be used to represent this ground motion. The dynamic effect of the Design Basis Ground Motion may be represented by the Design response Spectrum.

12. Design Response Spectrum

Design Response Spectrum is an elastic response spectrum for 5 percent equivalent viscous damping used to represent the dynamic effects of the Design Basis Ground Motion for the design of structures in accordance with Sections 4.2 and 4.3. This response spectrum may be either a site-specific spectrum based on geologic, tectonic, seismological and soil characteristics associated with a specific site or may be a spectrum constructed in accordance with the spectral shape in Figure 4.3-1 using the site-specific values of C_a and C_v and multiplied by the acceleration of gravity, 386.4 in./sec^2 (9.815 m/sec). See Section 4.3.2.

13. Design Seismic Force

Design Seismic Force is the minimum total strength design base shear, factored and distributed in accordance with Section 4.2.

14. Development Length for a Bar with a Standard Hook

Development length for a bar with a standard hook is the shortest distance between the critical section (where the strength of the bar is to be developed) and a tangent to the outer edge of the 90-degree hook.

15. Diaphragm

Diaphragm is a horizontal or nearly horizontal system acting to transmit lateral forces to the vertical-resisting elements. The term "diaphragm" includes horizontal bracing systems.

16. Diaphragm or Shear Wall Chord

Diaphragm or shear wall chord is the boundary element of diaphragm or shear wall that is assumed to take axial stresses analogous to the flanges of a beam.

17. Dual System

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

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

18. Gross Area

The area of a cross-section through the unit without reduction for the area of holes, voids and re-entrants.

19. Hoop

Hoop is a closed tie or continuously wound tie. A closed tie can be made up of several reinforcing elements, each having seismic hooks at both ends. A continuously wound tie shall have a seismic hook at both ends.

20. Horizontal Bracing System

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

21. Infill Concrete

A concrete used to fill pre-formed cavities or voids in masonry.

22. Intermediate Moment-Resisting Frame (IMRF)

Intermediate moment-resisting frame (IMRF) is a moment-resisting frame meeting special detailing requirements of Section 5.5

23. Joints

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

24. Live Loads

Live loads are those loads produced by the use and occupancy of the building or other structure and do not include dead load, construction load, or environmental loads such as wind load, snow load, rain load, earthquake load or flood load.

25. Masonry

An assemblage of masonry units laid in a specified pattern and joined together with mortar.

26. Masonry Bond

Disposition of units in masonry in a regular pattern to achieve common action.

27. Masonry Unit

A preformed component, intended for use in masonry construction.

28. Modal Mass

Modal mass of a structure subjected to horizontal or vertical ground motion is a part of the total seismic mass of the structure that is effective in any mode of vibration.

29. Moment Resisting Frame

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

30. Ordinary Moment-Resisting Frame (OMRF)

Ordinary moment-resisting frame (OMRF) is a moment-resisting frame not meeting special detailing requirements for ductile behavior.

31. Over Strength

Over strength is characteristic of structures where the actual strength is larger than the design strength. The degree of over strength is both material and system-dependent.

32. P-Δ Effect

P-Δ effect is the secondary effect on shears, axial forces and moments of frame members induced by the vertical loads acting on the laterally displaced building system.

33. Reinforced Masonry

Masonry in which bars or mesh are embedded in mortar or concrete so that all the materials act together in resisting action effects.

34. Seismic Hook

Seismic Hook is a hook on a stirrup, hoop or crosstie having a bend not less than 135 degrees with a six-bar diameter (but not less than 3 in. (76 mm)), extension that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop.

35. Shear Strength of Masonry

The strength of masonry subjected to shear forces.

36. Shear Wall

Shear wall is a wall designed to resist lateral forces parallel to the plane of the wall (sometimes referred to as vertical diaphragm or structural wall).

37. Soft Story

Soft story is one in which the lateral stiffness is less than 70 % of the stiffness of the story above.

38. Special Moment Resisting Frame (SMRF)

Special Moment Resisting Frame (SMRF) is a moment-resisting frame specially detailed to provide ductile behavior.

39. Story

Story is the space between levels. Story x is the story below level x.

40. Story Drift

Story drift is the lateral displacement of one level relative to the level above or below.

41. Story Drift Ratio

Story drift ratio is the story drift divided by the story height.

42. Story Shear, V_x

Story shear, V_x is the summation of design lateral forces above the story under consideration.

43. Strength

Strength is the capacity of an element or a member to resist factored load.

44. Structure

Structure is an assemblage of framing members designed to support gravity loads and resist lateral forces. Structures may be categorized as building structures or non-building structures.

45. Wall Anchorage System

Wall anchorage system is the system of elements anchoraging the wall to the diaphragm and those elements within the diaphragm required to develop the anchorage forces, including sub-diaphragms and continuous ties.

46. Weak Story

Weak story is one in which the story strength is less than 80 % of the story above.

47. Un-reinforced Masonry

Masonry not containing sufficient reinforcement so as to be considered as reinforced masonry.

2.2 Notations

The following symbols and notations apply to the provisions of SISBC:

A_c = the combined effective area, in square feet (m^2), of the shear walls in the first story of the structure.
A_{ch} = cross-sectional area of a structural member measured out-to-out of transverse reinforcement, square inches (mm^2).
A_{cv} = net area of concrete section bounded by web thickness and length of section in the direction of shear force considered, square inches (mm^2).
A_e = the minimum cross-sectional area in any horizontal plane in the first story, in square feet (m^2) of a shear wall.
A_g = gross area of section, square inches (mm^2).
A_j = effective cross-sectional area within a joint (see Section 5.3.3.1) 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:

1. beam width plus the joint depth
2. twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side.

A_s = area of tension reinforcement, square in. (mm^2).
A_{sh} = total cross-sectional area of transverse reinforcement (including crossties) within spacing, s, and perpendicular to dimension, h_c.
$A_{s,min}$ = minimum amount of flexural reinforcement, square in. (mm^2).
A_x = the torsional amplification factor at Level x.
a_p = numerical coefficient specified in Section 4.4 and set forth in Table 4-H.
b = width of section, in. (mm).
b_w = web width, or diameter of circular section, inches (mm).
C_a = seismic coefficient, as set forth in Table 4-J.
C_t = numerical coefficient given in Section 4.2.2.2.
C_v = seismic coefficient, as set forth in Table 4-K.
D = dead load on a structural element.
D_e = the length, in feet (m), of a shear wall in the first story in the direction parallel to the applied forces.
d = effective depth of section.
d_b = bar diameter.
E = load effects of earthquake set forth in Chapter 4, or related internal moments and forces.
F = load due to fluids.
F_i, F_n, F_x = Design Seismic Force applied to Level I, n, or x. respectively.
F_p = Design Seismic Forces on a part of the structure.
F_{px} = Design Seismic Force on a diaphragm.
F_t = that portion of the base shear, V, considered concentrated at the top of the structure in addition to F_n.
f_i = lateral force at Level i for use in Formula (4.2-10).
f'_c = specified compressive strength of concrete, psi (MPa).
f_y = specified yield strength of reinforcement, psi (MPa).
f_{yh} = specified yield strength of transverse reinforcement, psi (MPa).

g = acceleration due to gravity.
H = load due to lateral pressure of soil and water in soil.
h_c = cross-sectional dimension of a column core or shear wall boundary zone measured center-to-center of confining reinforcement.
h_i, h_n, h_x = height in feet (m) above the base to Level i, n, or x respectively.
h_w = height of entire wall (diaphragm) or of the segment of wall (diaphragm) considered.
I = importance factor given in Table 4-D.
I_p = importance factor specified in Table 4-D.
L = live load on a structural element including any permitted live load reduction.
Level *i* = level of the structure referred to by the subscript i. "*i* = 1" designates the first level above the base.
Level n = that level that is uppermost in the main portion of the structure.
Level x = that level that is under design consideration. "x = 1" designates the first level above the base.
l_d = development length for a straight bar.
l_{dh} = development length for a bar with a standard hook as defined in Formula (5-5).
l_o = minimum length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided, inches (mm).
l_u = unsupported length of compression member.
l_w = length of entire wall (diaphragm) or of segment of wall (diaphragm) considered in direction of shear force.
M = maximum moment magnitude.
M_{pr} = probable flexural strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile strength in the longitudinal bars of at least 1.25 f_y and a strength reduction factor ϕ of 1.0.
N_a = near-source factor used in the determination of C_a in Seismic Zone 4 related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in Tables 4-L and 4-N.
N_v = near-source factor used in the determination of C_v in Seismic Zone 4 related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in Tables 4-M and 4-N.
P = ponding load.
PI = plasticity index of soil determined in accordance.
R = numerical coefficient representative of the inherent overstrength and global ductility capacity of lateral force-resisting systems, as set forth in Table 4-G or 4-I.
S = snow load.
S_A, S_B,
S_C, S_D,
S_E, S_F = soil profile types as set forth in Table 4-C.

s = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, inches (mm).
s_o = maximum spacing of transverse reinforcement, inches (mm).
T = self-straining force and effects arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement, or combinations thereof.
T = elastic 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 given by Formulas (4.2-4), (4.2-5), (4.2-6), (4.2-7), and (4.2-11).
V_c = nominal shear strength provided by concrete.
V_e = design shear force determined from Section 5.1.4.1 or 5.2.5.1.
V_n = nominal shear strength.
V_x = the design story shear in Story x.
V_u = factored shear force at section.
W = load due to wind pressure.
W = the total seismic dead load defined in Section 4.2.2.
w_i, w_x = that portion of W located at or assigned to Level i or x respectively.
W_p = the weight of an element or component.
w_{px} = the weight of the diaphragm and the element tributary thereto at Level x, including applicable portions of other loads defined in Section 4.2.2.
Z = seismic zone factor as given in Table 4-B.
Δ_M = Maximum Inelastic Response Displacement, which is the total drift or total story drift that occurs when the structure is subjected to the Design Basis Ground Motion, including estimated elastic and inelastic contributions to the total deformation defined in Section 4.2.9.
Δ_S = Design Level Response Displacement, which is the total drift or total story drift that occurs when the structure is subjected to the design seismic forces.
α_c = coefficient defining the relative contribution of concrete strength to wall strength.
δ_i = horizontal displacement at Level i relative to the base due to applied lateral forces, f, for use in Formula (4.2-10).
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).
$\rho_v = A_{sv}/A_{cv}$; where A_{sv} is the projection on A_{cv} of area of distributed shear reinforcement crossing the plane of A_{cv}.

Chapter 3 Structural Loads for Design of Buildings

3.0 General

Buildings shall be designed for all loads as specified in this chapter.

3.1 Scope

This chapter prescribes different loads to be used for the design of buildings.

3.2 Dead Loads

3.2.1 General

Dead loads shall be as defined in chapter 2 and this section.

3.2.2 Partition Loads

Floors in office buildings and other buildings where partition locations are subject to change shall be designed to support, in addition to al1 other loads, a uniformly distributed dead load equal to 20 pounds per square foot (psf) (0.96 kN/m^2) of floor area.

EXCEPTION: Access floor systems shall be designed to support, in addition to all other loads, a uniformly distributed dead load not less than 10 psf (0.48 kN/m^2) of floor area.

3.3 Live Loads

3.3.1 General

Live loads shal1 be as defined in chapter 2 and shall be the maximum loads expected by the intended use or occupancy but in no case shal1 be less than the loads required by this section.

3.3.2 Minimum Live Load

Buildings shall be designed for the unit live loads as set forth in Table 3-A. These loads shal1 be taken as the minimum live loads in pounds per square foot of horizontal projection to be used in the design of buildings for the occupancies listed, and loads at least equal shal1 be assumed for uses not listed in this section but that create or accommodate similar loadings.

Where it can be determined in designing floors that the actual live load will be greater than the value shown in Table 3-A, the actual live load shal1 be used in the design of such buildings or portions thereof. Special provisions shal1 be made for machine and apparatus loads.

3.3.3 Distribution of uniform floor loads

Structural members shall be designed using the loading conditions, which would cause maximum shear and bending moments. Maximum negative moments at a support is obtained if the live load is placed on adjacent spans. Maximum positive moment at a span is obtained if the live load is placed on that span but not on adjacent spans (alternate). In both cases the dead load is placed on all the spans.

3.3.4 Concentrated loads

Provision shall be made in designing floors for a concentrated load, L, as set forth in Table 3-A placed upon any space 2½ feet (762 mm) square, wherever this load upon an otherwise unloaded floor would produce stresses greater than those caused by the uniform load required.

Provision shal1 be made in areas where vehicles are used or stored for concentrated loads, L, consisting of two or more loads spaced 5 feet (1,524 mm) nominal1y on center without uniform live loads. Each load shal1 be 40 percent of the gross weight of the maximum-size vehicle to be accommodated. The condition of concentrated or uniform live load, combined in accordance with Section 4.1.1.1 or 4.1.1.2 as appropriate, producing the greatest stresses shal1 govern.

3.3.5 Live loads posted

The live loads for which each floor or portion thereof of a commercial or industrial building is or has been designed shal1 have such design live loads conspicuously posted by the owner in that part of each story in which they apply, using durable metal signs, and it shall be unlawful to remove or deface such notices. The occupants of the building shal1 be responsible for keeping the actual load below the allowable limits.

3.3.6 Reduction of Live Loads

The design live load determined using the unit live loads as set forth in Table 3-A for floors may be reduced on any member supporting more than 150 square feet (13.94 m^2), except for floors in places of public assembly and for live loads greater than 100 psf (4.79 kN/m^2), in accordance with the following formula:

$$R = 0.08\ (A - 150)$$

For SI:

$$R = 0.08\ (A - 13.94)$$

The reduction shall not exceed 40 percent or R, as determined by the following formula:

$$R = 23.1\ (1 + D/L)$$

Where:

A = area of floor or roof supported by the member, square feet (m^2).

D = dead load per square foot (m^2) of area supported by the member.

L = unit live load per square foot (m^2) of area supported by the member.
R = reduction in percentage.

3.4 Snow Loads

Buildings and all portions thereof that are subjected to snow loadings shall be designed to resist the snow loads. Potential unbalanced accumulation of snow at valleys, parapets, roof and offsets of uneven configuration shall be considered. Roof shall be designed for minimum snow load of 20 psf but shall be increased if the actual snow load exceeds this value.

3.5 Wind Loads

Every building or structure and every portion thereof shall be designed and constructed to resist the wind effects. Wind shall be assumed to come from any horizontal direction. No reduction in wind pressure shall be taken for the shielding effect of adjacent structures.

3.6 Earthquake Loads

Earthquake loads shall be determined in accordance with Chapter 4.

3.7 Other Minimum Loads

3.7.1 General

In addition to the other design loads specified in this chapter, structures shall be designed to resist the loads specified in this section.

3.7.2 Other Loads

Buildings and other structures and portions thereof shall be designed to resist all loads due to applicable fluid pressures, F, lateral soil pressures, H, ponding loads, P, and self-straining forces, T.

3.7.3 Impact Loads

Impact loads shall be included in the design of any structure where impact loads occur.

3.7.4 Loads for Anchorage of Concrete and Masonry Walls

Anchorage of the roof to walls and columns, and of walls and columns to foundations, shall be provided to resist the uplift and sliding forces that result from the application of the prescribed forces. Such anchorage shall be capable of resisting the load combinations of Section 4.1.1.1 or 4.1.1.2 using the greater of the wind or earthquake loads required by this chapter or a minimum horizontal force of 280 pounds per linear foot (4.09 kN/m) of wall, substituted for E.

3.7.5 Lateral Loads on Interior Wall

Interior walls, permanent partitions and temporary partitions that exceed 6 feet (1,829 mm) in height shall be designed to resist all loads to which they are subjected but not less than a load, L, of 4 psf (0.24 kN/m^2) applied perpendicular to the walls. The 4 psf (0.24 kN/m^2) load need not be applied simultaneously with wind or seismic loads. The deflection of such walls under a load of 5 psf (0.24 kN/m^2) shall not exceed 1/240 of the span for walls with brittle finishes and 1/120 of the span for walls with flexible finishes. See Table 4-H for earthquake design requirements where such requirements are more restrictive.

EXCEPTION: Flexible, folding or portable partitions are not required to meet the load and deflection criteria but must be anchored to the supporting structure to meet the provisions of this code.

3.7.6 Retaining Walls

Retaining walls shall be designed to resist loads due to the lateral pressure of retained material in accordance with accepted engineering practice.

Retaining walls shall be designed to resist sliding by at least 1.5 times the lateral force and overturning by at least 1.5 times the overturning moment, using allowable stress design loads.

3.7.7 Hydrostatic Uplift

All foundations, slabs and other footings subjected to water pressure shall be designed to resist a uniformly distributed uplift load, F. equal to the full hydrostatic pressure.

TABLE 3-A-UNIFORM AND CONCENTRATED LOADS

User or Occupancy		**Uniform Load[1] (psf)**	**Concentrated Load (pounds)**
Category	**Description**	**x 0.0479 for N/m^2**	**x 0.00448 for N**
1. Access floor systems	Office use	50	2,000[2]
	Computer use	100	2,000[2]
2. Armories		150	0
3. Assembly areas[3] and auditoriums and balconies therewith	Fixed seating areas	50	0
	Movable seating and other areas	100	0
	Stage areas and enclosed platforms	125	0
4. Cornices and marquees		60[4]	0
5. Exit facilities[5]		100	0[6]
6. Garages	General storage and/or repair	100	[7]
	Private or pleasure-type motor vehicle storage	50	[7]
7. Hospitals	Wards and rooms	70	1,000[2]
8. Libraries	Reading rooms	100	1,000[2]
	Stack rooms	125	1,500[2]
9. Manufacturing	Light	75	2,000[2]
	Heavy	125	3,000[2]
10. Offices		60	2,000[2]
11. Printing plants	Press rooms	150	2,500[2]
	Composing and linotype rooms	100	2,000[2]
12. Residential[8]	Basic floor area	40	0[6]
	Exterior balconies	60[4]	0
	Decks	40[4]	0
	Storage	40	0
13. Restrooms[9]		40	
14. Schools	Classrooms	70	1,000[2]
15.Sidewalks and driveways	Public access	250	[7]
16. Storage	Light	125	
	Heavy	250	
17. Stores		100	3,000[2]
18. Pedestrian bridges and walkways		100	

[1] See Section 3.3.6 for live load reductions.
[2] See Section 3.3.4, first paragraph, for area of load application.
[3] Assembly areas include such occupancies as dance halls, drill rooms, gymnasiums, playgrounds, plazas, terraces and similar occupancies that are generally accessible to the public.
[4] When snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design as determined by the building official. See Section 3.4.
[5] Exit facilities shall include such uses as corridors serving an occupant load of 10 or more persons, exterior exit balconies, stairways, fire escapes and similar uses.
[6]1ndividual stair treads shall be designed to support a 300-pound (1.33 kN) concentrated load placed in a position that would cause maximum stress. Stair stringers may be designed for the uniform load set forth in the table.
[7] See Section 3.3.4, second paragraph, for concentrated loads.
[8] Residential occupancies include private dwellings, apartments and hotel guest rooms.
[9] Restroom loads shall not be less than the load for the occupancy with which they are associated, but need not exceed 50 pounds per square foot (2.4 kN/m^2).

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Chapter 4 General Requirements for Seismic Analysis and Design of Buildings

4.0 General

4.0.1 Purpose

The purpose of the seismic provision herein is primarily to safeguard against major structural failures and loss of life, not limit damage or maintain function.

4.0.2 Minimum Seismic Design

Structures and portions thereof shall, as a minimum, be designed and constructed to resist the effect of seismic ground motion as provided in this chapter.

4.0.3 Seismic and Wind Design

When the wind design produces greater effects, the wind design shall governs, but detailing requirements and limitations prescribed in this code shall be followed.

4.1 Criteria Selection

4.1.1 Basis for Design

The procedures and the limitations for the design of structures shall be determined considering seismic zoning, site characteristics, occupancy, configuration, structural system and height in accordance with this section. Structures shall be designed with adequate strength to withstand the lateral displacements induced by the Design Basis Ground Motion, considering the inelastic response of the structure and the inherent redundancy, over strength and ductility of the lateral-force resisting system. The minimum design strength shall be based on the Design Seismic Forces determined in accordance with the static lateral force procedure of Section 4.2, except as modified by Section 4.3.5.4. Where strength design is used, the load combinations of Section 4.1.1.1 shall apply. Where Allowable Stress Design is used, the load combinations of Section 4.1.1.2 shall apply. Allowable Stress Design may be used to evaluate sliding or overturning at the soil-structure interface regardless of the design approach used in the design of the structure, provided load combinations of this section are utilized. One- and two-family dwellings in Seismic Zone 1 need not conform to the provisions of this section.

4.1.1.1 Load Combinations for Strength Design

Where Strength Design is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

$1.4D$	(4.1-1)
$1.4D + 1.7L$	(4.1-2)
$0.75(1.4D + 1.7L \pm 1.7W)$	(4.1-3)
$0.9D \pm 1.3W$	(4.1-4)
$0.75(1.4D + 1.7L \pm 1.87E)$	(4.1-4)
$0.9D \pm 1.43E$	(4.1-6)

Other loads. Where F. H, P or T are to be considered in design, each applicable load shall be added to the above combinations factored as follows: 1.3F. 1.6H, 1.2P and 1.2T.

4.1.1.2 Load Combinations Using Allowable Stress Design

Where Allowable Stress Design (Working Stress Design) is used, structures and all portions thereof shall resist the most critical effects resulting from the following combinations of loads:

D	(4.1-7)
$D + L$	(4.1-8)
$D + (W \text{ or } E/1.4)$	(4.1-9)
$0.9D \pm E/1.4$	(4.1-10)
$D + 0.75\ [L + (W \text{ or } E/1.4)]$	(4.1-11)

4.1.1.3 Deflection

The deflection of any structural member shall not exceed the values set forth in Table 4-A. The deflection criteria representing the most restrictive condition shall apply.

4.1.2 Occupancy Categories

For purposes of earthquake resistant design, each structure shall be placed in one of the occupancy categories listed in Table 4-D. The table assigns importance factors, I and I_p and structural observation requirements for each category.

4.1.3 Site Geology and Soil Characteristics

Each site shall be assigned a soil profile type based on properly substantiated geotechnical data using Table 4-C.

EXCEPTION: When the soil properties are not known in sufficient detail to determine the soil profile type, Type S_D shall be used. Soil Profile Type S_E or S_F need not be assumed unless the building official determines that Type S_E or S_F may be present at the site or in the event that Type S_E or S_F is established by geotechnical data.

4.1.3.1 Soil profile type

Soil Profile Types S_A. S_B, S_C, S_D and S_E are defined in Table 4-C and Soil Profile Type S_F is defined as soils requiring site-specific evaluation as follows:

1. Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.

2. Peats and/or highly organic clays, where the thickness of peat or highly organic clay exceeds 10 feet (3,048 mm).

3. Very high plasticity clays with a plasticity index, *PI* > 74, where the depth of clay exceeds 24 feet (7,620 mm).

4. Very thick soft/medium stiff clays, where the depth of clay exceeds 120 feet (36,476 mm).

4.1.4 Site Seismic Hazard Characteristics

Seismic hazard characteristics for the site shall be established based on the seismic zone and proximity of the site to active seismic sources, site soil profile characteristics and the structure's importance factor.

4.1.4.1 Seismic zone

Each site of October 8 earthquake-affected area shall be assigned a seismic zone in accordance with Table 4-O. Currently, there is no realistic seismic zoning map available for Pakistan due to the unavailability of authentic instrumented seismic data (Ref. 1). As argued, the UNGSHAP map (figure. 4.3-2) may be used with sufficient accuracy for various areas of Pakistan. However, seismic hazard categorization (Table 4-O) of various cities and towns of the earthquake-affected area was carried out on the basis of available damage data. Relation like Intensity vs PGA (Ref. 2) and Magnitude vs PGA (Ref. 3) were used for such zoning. Each zone shall be assigned a seismic zone factor Z, in accordance with Table 4-B.

4.1.4.2 Seismic Zone 4 near-source factor

In Seismic Zone 4, each site shall be assigned a near-source factor in accordance with Table 4-L and the Seismic Source
Type set forth in Table 4-N.

4.1.4.3 Seismic response coefficients

Each structure shall be assigned a seismic coefficient, C_a.

in accordance with Table 4-J and a seismic coefficient, C_v, in accordance with Table 4-K.

4.1.5 Configuration Requirements

4.1.5.1 General

Each structure shall be designated as being structurally regular or irregular in accordance with Sections 4.1.5.2 and 4.1.5.3.

4.1.5.2 Regular structures

Regular structures have no significant physical discontinuities in plan or vertical configuration or in their lateral-force-resisting systems such as the irregular features described in Section 4.1.5.3.

4.1.5.3 Irregular structures

1. Irregular structures have significant physical discontinuities in configuration or in their lateral-force-resisting systems. Irregular features include, but are not limited to, those described in Tables 4-E and 4-F. All structures in Seismic Zone 1 and Occupancy Categories 4 and 5 in Seismic Zone 2 need to be evaluated only for vertical irregularities of Type 5 (Table 4-E) and horizontal irregularities of Type 1 (Table 4-F).

2. Structures having any of the features listed in Table 4-E shall be designated as if having a vertical irregularity.

EXCEPTION: Where no story drift ratio under design lateral forces is greater than 1.3 times the story drift ratio of the story above, the structure may be deemed to not have the structural irregularities of Type 1 or 2 in Table 4-E. The story drift ratio for the top two stories need not be considered. The story drifts for this determination may be calculated neglecting torsional effects.

3. Structures having any of the features listed in Table 4-F shall be designated as having a plan irregularity.

4.1.6 Structural Systems

4.1.6.1 General

Structural systems shall be classified as one of the types listed in Table 4-G and defined in this section.

4.1.6.2 Bearing wall system

A structural system without a complete vertical load-carrying space frame. Bearing walls or bracing systems provide support for all or most gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

4.1.6.3 Building frame system

A structural system with an essentially complete space frame providing support for gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

4.1.6.4 Moment-resisting frame system

A structural system with an essentially complete space frame providing support for gravity loads. Moment-resisting frames provide resistance to lateral load primarily by flexural action of members.

4.1.6.5 Dual system

A structural system with the following features:

1. An essentially complete space frame that provides support for gravity loads.

2. Resistance to lateral load is provided by shear walls or braced frames and moment-resisting frames (IMRF, OMRF). The moment-resisting frames shall be designed to independently resist at least 25 percent of the design base shear.

3. The two systems shall be designed to resist the total design base shear in proportion to their relative rigidities considering the interaction of the dual system at all levels.

4.1.6.6 Cantilevered column system

A structural system relying on cantilevered column elements for lateral resistance.

4.1.6.7 Undefined structural system

A structural system not listed in Table 4-G.

4.1.6.8 Non-building structural system

A structural system conforming to Section 4.6.

4.1.7 Height Limits

Height limits for the various structural systems in Seismic Zones 3 and 4 are given in Table 4-G.

4.1.8 Selection of Lateral-force Procedure

4.1.8.1 General

Any structure may be, and certain structures defined below shall be, designed using the dynamic lateral-force procedures of Section 4.3.

4.1.8.2 Simplified Static.

The simplified static lateral-force procedure set forth in Section 4.2.2.3 may be used for buildings not more than two stories in height excluding basements.

4.1.8.3 Static.

The static lateral force procedure of Section 4.2 may be used for the following structures:

1. All structures, regular or irregular, in Seismic Zone 1 and in Occupancy Categories 4 and 5 in Seismic Zone 2.

2. Regular structures under 200 feet (60,960 mm) in height with lateral force resistance provided by systems listed in Table 4-G, except where Section 4.1.8.4, Item 4, applies.

3. Irregular structures not more than three stories or 35 feet (10,668 mm) in height.

4.1.8.4 Dynamic.

The dynamic lateral-force procedure of Section 4.3 shall be used for all other structures, including the following:

1. Structures 200 feet (60,960 mm) or more in height, except as permitted by Section 4.1.8.3, Item 1.

2. Structures having a stiffness, weight or geometric vertical irregularity of Type 1, 2 or 3, as defined in Table 4-E, or structures having irregular features not described in Table 4-E or 4-F, except as permitted by Section 4.2.4.2.

3. Structures over three stories or 35 feet (10,668 mm) in height in Seismic Zones 3 and 4 not having the same structural system throughout their height except as permitted by Section 4.2.4.2.

4. Structures, regular or irregular, located on Soil Profile Type S_F, that have a period greater than 0.7 second. The analysis shall include the effects of the soils at the site and shall conform to Section 4.3.2, Item 4.

4.1.9 System Limitations

4.1.9.1 Discontinuity.

Structures with a discontinuity in capacity, vertical irregularity Type 5 as defined in Table 4-E, shall not be over two stories or 25 feet (7,620 mm) in height where the weak story has a calculated strength of less than 65 percent of the story above.

4.1.9.2 Undefined structural systems

For undefined structural systems not listed in Table 4-G, the coefficient R shall be substantiated by approved cyclic test data and analyses. The following items shall be addressed when establishing R:

1. Dynamic response characteristics,
2. Lateral force resistance,
3. Over strength and strain hardening or softening,
4. Strength and stiffness degradation,
5. Energy dissipation characteristics,
6. System ductility, and
7. Redundancy.

4.1.9.3 Irregular features

All structures having irregular features described in Table 4-E or 4-F shall be designed to meet the additional requirements of those sections referenced in the tables.

4.1.10 Alternative Procedures

4.1.10.1 General

Alternative lateral-force procedures using rational analyses based on well-established principles of mechanics may be used in lieu of those prescribed in these provisions.

4.1.10.2 Seismic isolation

Seismic isolation, energy dissipation and damping systems may be used in the design of structures when approved by the building official and when special detailing is used to provide results equivalent to those obtained by the use of conventional structural systems.

4.2 Design Lateral Forces and Related Effects

4.2.1 Modeling Requirements

4.2.1.1 General requirements

The mathematical model of the physical structure shall include all elements of the lateral force-resisting system. The model shall also include the stiffness and strength of elements, which are significant to the distribution of forces, and shall represent the spatial distribution of the mass and stiffness of the structure. In addition, effects of cracked section in the stiffness properties of reinforced concrete and masonry elements shall be considered in the model.

4.2.1.2 P-Δ effects

The resulting member forces and moments and the story drifts induced by $P\Delta$ effects shall be considered in the evaluation of overall structural frame stability and shall be evaluated using the forces producing the displacements of Δ_S. $P\Delta$ need not be considered when the ratio of secondary moment to primary moment does not exceed 0.10; the ratio may be evaluated for any story as the product of the total dead, floor live and snow load, as required in Chapter 3, above the story times the seismic drift in that story divided by the product of the seismic shear in that story times the height of that story. In Seismic Zones 3 and 4, $P\Delta$ need not be considered when the story drift ratio does not exceed $0.02/R$.

4.2.2 Static Force Procedure

4.2.2.1 Design base shear

The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{C_v I}{RT} W \qquad (4.2\text{-}4)$$

The total design base shear need not exceed the following:

$$V = \frac{2.5 C_a I}{R} W \qquad (4.2\text{-}5)$$

The total design base shear shall not be less than the following:

$$V = 0.11 C_a I W \qquad (4.2\text{-}6)$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{0.8 Z N_v I}{R} W \qquad (4.2\text{-}7)$$

4.2.2.2 Structure period

The value of T shall be determined from one of the following methods:

1. Method A: For all buildings, the value T may be approximated from the following formula:

$$T = C_t (h_n)^{3/4} \quad (4.2\text{-}8)$$

Where:

C_t = 0.030 (0.0731) for reinforced concrete moment-resisting

C_t = 0.030 (0.0731) for reinforced concrete moment-resisting frames and eccentrically braced frames.

C_t = 0.020 (0.0488) for all other buildings.

Alternatively, the value of C_t for structures with concrete or masonry shear walls may be taken as $0.1/\sqrt{A_c}$ (for SI: $0.0743/\sqrt{A_c}$ for A_c in m^2).

The value of A_c shall be determined from the following formula:

$$A_c = \sum A_e [0.2 + (D_e / h_n)^2] \quad (4.2\text{-}9)$$

The value of D_e/h_n used in Formula (4.2-9) shall not exceed 0.9.

2. 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. The analysis shall be in accordance with the requirements of Section 4.2.1.1. The value of T from Method B shall not exceed a value 30 percent greater than the value of T obtained from Method A in Seismic Zone 4, and 40 percent in Seismic Zones 1, 2 and 3.

The fundamental period T may be computed by using the following formula:

$$T = 2\pi \sqrt{(\sum_{i=1}^{n} w_i \delta_i^2) \div (g \sum_{i=1}^{n} f_i \delta_i)} \quad (4.2\text{-}10)$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of Formulas (4.2-13), (4.2-14) and (4.2-15) or any other rational distribution. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i.

4.2.2.3 Simplified design base shear

4.2.2.3.1 General

Structures conforming to the requirements of section 4.1.8.2 may be designed using this procedure.

4.2.2.3.2 Base shear

The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{3.0C_a}{R}W \qquad (4.2\text{-}11)$$

Where the value of C_a shall be based on Table 4-J for the soil profile type. When the soil properties are not known in sufficient detail to determine the soil profile type, Type S_D shall be used in Seismic Zones 3 and 4, and Type S_E shall be used in Seismic Zones 1, 2A and 2B. In Seismic Zone 4, the Near-Source Factor, N_a, need not be greater than 1.3 if none of the following structural irregularities are present: Type 1, 4 or 5 of Table 4-E, or Type 1 or 4 of Table 4-F.

4.2.2.3.3 Vertical distribution

The forces at each level shall be calculated using the following formula:

$$F_x = \frac{3.0C_a}{R}w_i \qquad (4.2\text{-}12)$$

Where the value of C_a shall be determined in Section 4.2.2.3.2.

4.2.2.3.4 Applicability

Sections 4.2.1.1, 4.2.1.2, 4.2.2.1, 4.2.2.2, 4.2.5, 4.2.9, 4.2.10 and 4.3 shall not apply when using the simplified procedure.

EXCEPTION: For buildings with relatively flexible structural systems, the building official may require consideration of PΔ effects and drift in accordance with

Sections 4.2.1.2, 4.2.9 and 4.2.10. Δ_s shall be prepared using design seismic forces from Section 4.2.2.3.2.

Where used, Δ_M shall be taken equal to 0.01 times the story height of all stories. In Section 4.5.2.9, Formula (4.5-1) shall read

$F_{px} = \frac{3.0C_a}{R} w_{px}$ and need not exceed $1.0C_a w_{px}$, but shall not be less than $0.5C_a w_{px}$. R shall be taken from Table 4-G.

4.2.3 Determination of Seismic Factors

4.2.3.1 Determination of R

The value of R shall be taken from Table 4-G.

4.2.4 Combinations of Structural Systems

4.2.4.1 General

Where combinations of structural systems are incorporated into the same structure, the requirements of this section shall be satisfied.

4.2.4.2 Vertical combinations

The value of R used in the design of any story shall be less than or equal to the value of R used in the given direction for the story above.

EXCEPTION: This requirement need not be applied to a story where the dead weight above that story is less than 10 percent of the total dead weight of the structure.

Structures may be designed using the lowest *R* of the lateral-force-resisting systems used.

4.2.4.3 Combinations along different axes

In Seismic Zones 3 and 4 where a structure has a bearing wall system in only one direction, the value of R used for design in the orthogonal direction shall not be greater than that used for the bearing wall system.

Any combination of bearing wall systems, building frame systems, dual systems or moment-resisting frame systems may be used to resist seismic forces in structures less than 80 feet (24,384 mm) in height. Only combinations of dual systems and special moment-resisting frames shall be used to resist seismic forces in structures exceeding 80 feet (24,384 mm) in height in Seismic Zones 3 and 4.

4.2.4.4 Combinations along the same axis

For other than dual systems and shear wall-frame interactive systems in Seismic Zone 1, where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of R used for design in that direction shall not be greater than the least value for any of the systems utilized in that same direction.

4.2.5 Vertical Distribution of Force

The total force shall be distributed over the height of the structure in conformance with Formulas (4.2-13), (4.2-14) and (4.2-15) in the absence of a more rigorous procedure.

$$V = F_t + \sum_{i=1}^{n} F_i \qquad (4.2\text{-}13)$$

The concentrated force F_t at the top, which is in addition to F_n shall be determined from the formula:

$$F_t = 0.07\ T\ V \qquad (4.2\text{-}14)$$

The value of T used for the purpose of calculating F_t shall be the period that corresponds with the design base shear as computed using Formula (4.2-4). F_t need not exceed 0.25V and may be considered as zero where T is 0.7 second or less. The remaining portion of the base shear shall be distributed over the height of the structure, including Level n, according to the following formula:

$$F_x = \frac{(V - F_t) w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (4.2\text{-}15)$$

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level. Structural displacements and design seismic forces shall be calculated as the effect of forces F_x and F_t applied at the appropriate levels above the base.

4.2.6 Horizontal Distribution of Shear

The design story shear, V_x, in any story is the sum of the forces F_t and F_x above that story. 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 diaphragm. See Section 4.5.2.4 for rigid elements that are not intended to be part of the lateral-force resisting systems.

Where diaphragms are not flexible, the mass at each level shall be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration. The effect of this displacement on the story shear distribution shall be considered.

Diaphragms shall be considered flexible for the purposes of distribution of story shear and torsional moment when the maximum lateral deformation of the diaphragm is more than two times the average story drift of the associated story. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm itself under lateral load with the story drift of adjoining vertical-resisting elements under equivalent tributary lateral load.

4.2.7 Horizontal Torsional Moments

Provisions shall be made for the increased shears resulting from horizontal torsion where diaphragms are not flexible. The most severe load combination for each element shall be considered for design.

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

The accidental torsional moment shall be determined by assuming the mass is displaced as required by Section 4.2.6.

Where torsional irregularity exists, as defined in Table 4-F, the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor, A_x, determined from the following formula:

$$A_x = \left[\frac{\delta_{\max}}{1.2\delta_{avg}} \right]^2 \qquad (4.2\text{-}16)$$

WHERE:

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x

δ_{max} = the average of the displacements at the extreme points of the structure at Level x

The value of A_x need not exceed 3.0.

4.2.8 Overturning

4.2.8.1 General

Every structure shall be designed to resist the overturning effects caused by earthquake forces specified in Section 4.2.5. At any level, the overturning moments to be resisted shall be determined using those seismic forces (F_t and F_x) that act on levels above the level under consideration. At any level, the incremental changes of the design overturning moment shall be distributed to the various resisting elements in the manner prescribed in Section 4.2.6. Overturning effects on every element shall be carried down to the foundation. See Sections 4.1.1 for combining gravity and seismic forces.

4.2.8.2 Elements supporting discontinuous systems

4.2.8.2.1 General

Where any portion of the lateral-load resisting system is discontinuous, such as for vertical irregularity Type 4 in Table 4-E or plan irregularity Type 4 in Table 4-F, concrete and masonry elements supporting such discontinuous systems shall have the design strength to resist the combination loads resulting from the seismic load combinations of Section 4.1.1.

For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and a resistance factor, ϕ of 1.0. This increase shall not be combined with the one third stress increase permitted by Section 4.1.1.2.

4.2.8.2.2 Detailing requirements in Seismic Zones 3 and 4

In Seismic Zones 3 and 4, elements supporting discontinuous systems shall meet the following detailing or member limitations:

1. Reinforced concrete elements designed primarily as axial load members shall comply with Section 5.2.4.5.

2. Reinforced concrete elements designed primarily as flexural members and supporting other than light-frame wood shear wall systems or light-frame steel and wood structural panel shear wall systems shall comply with Sections 5.1.2 and 5.1.3. Strength computations for portions of slabs designed as supporting

elements shall include only those portions of the slab that comply with the requirements of these sections.

4.2.8.3 At foundation

For regular buildings the force F_t as provided in Section 4.2.5 may be omitted when determining the overturning moment to be resisted at the foundation soil interface.

4.2.9 Drift

Drift or horizontal displacements of the structure shall be computed where required by this code. For both Allowable Stress Design and Strength Design, the Maximum Inelastic Response Displacement, Δ_M, of the structure caused by the Design Basis Ground Motion shall be determined in accordance with this section. The drifts corresponding to the design seismic forces of Section 4.2.2.1, Δ_s shall be determined in accordance with Section 4.2.9.1. To determine Δ_M, these drifts shall be amplified in accordance with Section 4.2.9.2.

4.2.9.1 Determination of Δ_S

A static, elastic analysis of the lateral force-resisting system shall be prepared using the design seismic forces from Section 4.2.2.1. Alternatively, dynamic analysis may be performed in accordance with Section 4.3. The mathematical model shall comply with Section 4.2.1.1. The resulting deformations, denoted as Δ_s, shall be determined at all critical locations in the structure. Calculated drift shall include translational and torsional deflections.

4.2.9.2 Determination of Δ_M

The Maximum Inelastic Response Displacement, Δ_M, shall be computed as follows:

$$\boldsymbol{\Delta_M} = 0.7\,R\Delta_s \qquad (4.2\text{-}17)$$

The analysis used to determine the Maximum Inelastic Response Displacement Δ_M shall consider PΔ effects.

4.2.10 Story Drift Limitation

4.2.10.1 General

Story drifts shall be computed using the Maximum Inelastic Response Displacement Δ_M.

4.2.10.2 Calculated

Calculated story drift using Δ_M shall not exceed 0.025 times the story height for structures having a fundamental period of less than 0.7 second. For structures having a fundamental period of 0.7 second or greater, the calculated story drift shall not exceed 0.020 times the story height.

4.2.10.3 Limitations

The design lateral forces used to determine the calculated drift may disregard the limitations of Formula (4.2-6) and may be based on the period determined from Formula (4.2-10) neglecting the 30 or 40 percent limitations of Section 4.2.2.2, Item 2.

4.2.11 Vertical Component

The following requirements apply in Seismic Zones 3 and 4 only. Horizontal cantilever components shall be designed for a net upward force of $0.7C_aIW_p$.

4.3 Dynamic Analysis Procedures

4.3.1 General

Dynamic analyses procedures, when used, shall conform to the criteria established in this section. The analysis shall be based on an appropriate ground motion representation and shall be performed using accepted principles of dynamics. Structures that are designed in accordance with this section shall comply with all other applicable requirements of these provisions.

4.3.2 Ground Motion

The ground motion representation shall, as a minimum, be one having a 10-percent probability of being exceeded in 50 years, shall not be reduced by the quantity R and may be one of the following:

1. An elastic design response spectrum constructed in accordance with Figure 4.3-1, using the values of C_a and C_v consistent with the specific site. The design acceleration ordinates shall be multiplied by the acceleration of gravity, 386.4 in./sec.^{2} (9.815 m/sec.^{2}).

2. A site-specific elastic design response spectrum based on the geologic, tectonic, seismologic and soil characteristics associated with the specific site. The spectrum shall be developed for a damping ratio of 0.05, unless a different value is shown to be consistent with the anticipated structural behavior at the intensity of shaking established for the site.

3. Ground motion time histories developed for the specific site shall be representative of actual earthquake motions. Response spectra from time histories, either individually or in combination, shall approximate the site design spectrum conforming to Section 4.3.2, Item 2.

4. For structures on Soil Profile Type S_F, the following requirements shall apply when required by Section 4.1.8.4, Item 4:

   4.1 The ground motion representation shall be developed in accordance with Items 2 and 3.

4.2 Possible amplification of building response due to the effects of soil-structure interaction and lengthening of building period caused by inelastic behavior shall be considered.

5. The vertical component of ground motion may be defined by scaling corresponding horizontal accelerations by a factor of two - thirds. Alternative factors may be used when substantiated by site-specific data. Where the Near Source Factor, N_a, is greater than 1.0, site-specific vertical response spectra shall be used in lieu of the factor of two-thirds.

4.3.3 Mathematical Model

A mathematical model of the physical structure shall represent the spatial distribution of the mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of its dynamic response. A three-dimensional model shall be used for the dynamic analysis of structures with highly irregular plan configurations such as those having a plan irregularity defined in Table 4-F and having a rigid or semi-rigid diaphragm. The stiffness properties used in the analysis and general mathematical modeling shall be in accordance with Section 4.2.1.1.

4.3.4 Description of Analysis Procedures

4.3.4.1 Response spectrum analysis

An elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

4.3.4.2 Time-history analysis

An analysis of the dynamic response of a structure at each increment of time when the base is subjected to a specific ground motion time history.

4.3.5 Response Spectrum Analysis

4.3.5.1 Response spectrum representation & interpretation of results

The ground motion representation shall be in accordance with Section 4.3.2. The corresponding response parameters, including forces, moments and displacements, shall be denoted as Elastic Response Parameters. Elastic Response Parameters may be reduced in accordance with Section 4.3.5.4.

4.3.5.2 Number of modes

The requirement of Section 4.3.4.1 that all significant modes be included may be satisfied by demonstrating that for the modes considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

4.3.5.3 Combining modes

The peak member forces, displacements, story forces, story shears and base reactions for each mode shall be combined by recognized methods. When three - dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maxima.

4.3.5.4 Reduction of Elastic Response Parameters for design

Elastic Response Parameters may be reduced for the purposes of design but it shall not be less than Equivalent Static Lateral Force as determined in Section 4.2.

4.3.5.5 Torsion

The analysis shall account for torsional effects, including accidental torsional effects as prescribed in Section 4.2.7. 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 equivalent static procedures such as provided in Section 4.2.6.

4.3.5.6 Dual systems

Where the lateral forces are resisted by a dual system as defined in Section 4.1.6.5, the combined system shall be capable of resisting the base shear determined in accordance with this section. The moment-resisting frame shall conform to Section 4.1.6.5, Item 2, and may be analyzed using either the procedures of Section 4.2.5 or those of Section 4.3.5.

4.3.6 Time-history Analysis

4.3.6.1 Time history

Time-history analysis shall be performed with pairs of appropriate horizontal ground-motion time history components that shall be selected and scaled from not less than three recorded events. Appropriate time histories shall have magnitudes, fault distances and source mechanisms that are consistent with those that control the design-basis earthquake (or maximum capable earthquake). Where three appropriate recorded ground-motion time-history pairs are not available, appropriate simulated ground-motion time-history pairs may be used to make up the total number required. For each pair of horizontal ground motion components, the square root of the sum of the squares (SRSS) of the 5 percent-damped site-specific spectrum of the scaled horizontal components shall be constructed. The motions shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5 percent-damped spectrum of the design-basis earthquake for periods from 0.2T second to 1.5T seconds. Each pair of time histories shall be applied simultaneously to the model considering torsional effects.

The parameter of interest shall be calculated for each time history analysis. If three time-history analyses are performed, then the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, then the average value of the response parameter of interest may be used for design.

4.3.6.2 Elastic time-history analysis.

Elastic time history shall conform to Sections 4.3.1, 4.3.2, 4.3.3, 4.3.5.2, 4.3.5.4, 4.3.5.5, 4.3.5.6, and 4.3.6.1. Response parameters from elastic time-history analysis shall be denoted as Elastic Response Parameters. All elements shall be designed using Strength Design. Elastic Response Parameters may be scaled in accordance with Section 4.3.5.4.

4.4 Lateral Force on Nonstructural Components & Equipment Supported by Structures

4.4.1 General

Elements of structures and their attachments, permanent nonstructural components and their attachments, and the attachments for permanent equipment supported by a structure shall be designed to resist the total design seismic forces prescribed in Section 4.4.2. Attachments for floor- or roof-mounted equipment weighing less than 400 pounds (181 kg) and furniture need not be designed.

Attachments shall include anchorages and required bracing. Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces.

When the structural failure of the lateral-force-resisting systems of non-rigid equipment would cause a life hazard, such systems shall be designed to resist the seismic forces prescribed in Section 4.4.2.

4.4.2 Design for Total Lateral Force

The total design lateral seismic force, F_p, shall be determined from the following formula:

$$F_p = 4.0\, C_a\, I_p\, W_p \qquad (4.4\text{-}1)$$

Alternatively, F_p may be calculated using the following formula:

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \qquad (4.4\text{-}2)$$

Except that:

F_p shall not be less than $0.7C_aI_pW_p$ and need not be more than

$$4C_aI_pW_p \qquad (4.4\text{-}3)$$

WHERE:

h_x is the element or component attachment elevation with respect to ground level. h_x shall not be taken less than zero.

h_r is the structure roof elevation with respect to ground level.

a_p is the in-structure Component Amplification Factor that varies from 1.0 to 2.5.

A value for a_p shall be selected from Table 4-H. Alternatively, this factor may be determined based on the dynamic properties or empirical data of the component and the structure that supports it. The value shall not be taken less than 1.0.

R_p is the Component Response Modification Factor that shall be taken from Table 4-H, except that R_p for anchorages shall equal 1.5 for shallow expansion anchor bolts, shallow chemical anchors or shallow cast-in-place anchors. Shallow anchors are those with an embedment length-to-diameter ratio of less than 8. When anchorage is constructed of non ductile materials, or by use of adhesive, R_p shall equal 1.0.

The design lateral forces determined using Formula (4.4-1) or (4.4-2) shall be distributed in proportion to the mass distribution of the element or component.

Forces determined using Formula (4.4-1) or (4.4-2) shall be used to design members and connections that transfer these forces to the seismic-resisting systems. Members and connection design shall use the load combinations and factors specified in Section 4.1.1.

For applicable forces and Component Response Modification Factors in connectors for exterior panels and diaphragms, refer to Sections 4.5.2.4, 4.5.2.8 and 4.5.2.9.

Forces shall be applied in the horizontal directions, which result in the most critical loadings for design.

4.4.3 Specifying Lateral Forces

Design specifications for equipment shall either specify the design lateral forces prescribed herein or reference these provisions.

4.4.4 Relative Motion of Equipment Attachments

For equipment in Categories 1 and 2 buildings as defined in Table 4-D, the lateral-force design shall consider the effects of relative motion of the points of attachment to the structure, using the drift based upon Δ_M.

4.4.5 Alternative Designs

Where an approved national standard or approved physical test data provide a basis for the earthquake-resistant design of a particular type of equipment or other nonstructural component, such a standard or data may be accepted as a basis for design of the items with the following limitations:

1. These provisions shall provide minimum values for the design of the anchorage and the members and connections that transfer the forces to the seismic-resisting system.

2. The force, F_p, and the overturning moment used in the design of the nonstructural component shall not be less than 80 percent of the values that would be obtained using these provisions.

4.5 Detailed Systems Design Requirements

4.5.1 General

All structural framing systems shall comply with the requirements of Section 4.1. Only the elements of the designated seismic-force-resisting system shall be used to resist design forces. The individual components shall be designed to resist the prescribed design seismic forces acting on them. The components shall also comply with the specific requirements of Chapters 5 and 6. In addition, such framing systems and components shall comply with the detailed system design requirements of Section 4.5.

All building components in Seismic Zones 2, 3 and 4 shall be designed to resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead, floor live and snow loads.

Consideration shall be given to design for uplift effects caused by seismic loads.

In Seismic Zones 2, 3 and 4, provision shall be made for the effects of earthquake forces acting in a direction other than the principal axes in each of the following circumstances:

1. The structure has plan irregularity Type 5 as given in Table 4-F.

2. The structure has plan irregularity Type 1 as given in Table 4-F for both major axes.

3. A column of a structure forms part of two or more intersecting lateral-force-resisting systems.

EXCEPTION: If the axial load in the column due to seismic forces acting in either direction is less than 20 percent of the column axial load capacity.

The requirement that orthogonal effects be considered may be satisfied by designing such elements for 100 percent of the prescribed design seismic forces in one direction plus 30 percent of the prescribed design seismic forces in the perpendicular direction. The combination requiring the greater component strength shall be used for design. Alternatively, the effects of the two orthogonal directions may be combined on a square root of the sum of the squares (SRSS) basis. When the SRSS method of combining directional effects is used, each term computed shall be assigned the sign that will result in the most conservative result.

4.5.2 Structural Framing Systems

4.5.2.1 General

Four types of general building framing systems defined in Section 4.1.6 are recognized in these provisions and shown in Table 4-G. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces.

4.5.2.2 Detailing for combinations of systems

For components common to different structural systems, the more restrictive detailing requirements shall be used.

4.5.2.3 Connections

Connections that resist design seismic forces shall be designed and detailed on the drawings.

4.5.2.4 Deformation compatibility

All structural framing elements and their connections, not required by design to be part of the lateral-force-resisting system, shall be designed and/or detailed to be adequate to maintain support of design dead plus live loads when subjected to the expected deformations caused by seismic forces. $P\Delta$ effects on such elements shall be considered. Expected deformations shall be determined as the greater of the Maximum Inelastic Response Displacement, Δ_M, considering $P\Delta$ effects determined in accordance with Section 4.2.9.2 or the deformation induced by a story drift of 0.0025 times the story height. When computing expected deformations, the stiffening effect of those elements not part of the lateral-force-resisting system shall be neglected.

For concrete and masonry elements that are part of the lateral force-resisting system, the assumed flexural and shear stiffness properties shall not exceed one half of the gross section properties unless a rational cracked-section analysis is performed. Additional deformations that may result from foundation flexibility and diaphragm deflections shall be considered.

4.5.2.4.1 Adjoining rigid elements

Moment-resisting frames and shear walls may be enclosed by or adjoined by more rigid elements; provided it can be shown that the participation or failure of the more rigid elements will not impair the vertical and lateral load-resisting ability of the gravity load and lateral-force-resisting systems. The effects of adjoining rigid elements shall be considered when assessing whether a structure shall be designated regular or irregular in Section 4.1.5.1.

4.5.2.4.2 Exterior elements

Exterior nonbearing, nonshear wall panels or elements that are attached to or enclose the exterior shall be designed to resist the forces per Formula (4.4-1) or (4.4-2) and shall accommodate movements of the structure based on Δ_M and temperature changes. Such elements shall be supported by means of cast-in-place concrete or by mechanical connections and fasteners in accordance with the following provisions:

1. Connections and panel joints shall allow for a relative movement between stories of not less than two times story drift caused by wind, the calculated story drift based on Δ_M or ½ in. (12.7 mm), whichever is greater.

2. Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, or other connections providing equivalent sliding and ductility capacity.

3. Bodies of connections shall have sufficient ductility and rotation capacity to preclude fracture of the concrete or brittle failures at or near welds.

4. The body of the connection shall be designed for the force determined by Formula (4.4-2), where R_p = 3.0 and a_p = 1.0.

5. All fasteners in the connecting system, such as bolts, inserts, welds and dowels, shall be designed for the forces determined by Formula (4.4-2), where R_p = 1.0 and a_p = 1.0.

6. Fasteners embedded in concrete shall be attached to, or hooked around, reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel.

4.5.2.5 Ties and continuity

All parts of a structure shall be interconnected and the connections shall be capable of transmitting the seismic force induced by the parts being connected. As a minimum, any smaller portion of the building shall be tied to the remainder of the building with elements having at least strength to resist 0.5 C_aI times the weight of the smaller portion.

A positive connection for resisting horizontal force acting parallel to the member shall be provided for each beam, girder or truss. This force shall not be less than 0.3 C_aI times the dead plus live load.

4.5.2.6 Collector elements

Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces.

Collector elements, splices and their connections to resisting elements shall resist the forces determined in accordance with Formula (4.5-1).

4.5.2.7 Concrete frames

Concrete frames required by design to be part of the lateral-force-resisting system shall conform to the following:

1. In Seismic Zones 3 and 4 they shall be special moment resisting frames.

2. In Seismic Zones 1 and 2 they shall, as a minimum, be intermediate moment-resisting frames.

4.5.2.8 Anchorage of concrete walls

Concrete walls shall be anchored to all floors and roofs that provide out-of-plane lateral support of the wall. The anchorage shall provide a positive direct connection between the wall and floor or roof capable of resisting the larger of the horizontal forces specified in this section and Sections 3.7.4, & 4.4. In addition, in Seismic Zones 3 and 4, diaphragm to wall anchorage using embedded straps shall have the straps attached to or hooked around the reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel. Requirements for developing anchorage forces in diaphragms are given in Section 4.5.2.9. Diaphragm deformation shall be considered in the design of the supported walls.

4.5.2.8.1 Out-of-plane wall anchorage to flexible diaphragms

This section shall apply in Seismic Zones 3 and 4 where flexible diaphragms, as defined in Section

4.2.6, provide lateral support for walls.

1. Elements of the wall anchorage system shall be designed for the forces specified in Section 4.4 where $R_p = 3.0$ and $a_p = 1.5$.

In Seismic Zone 4, the value of F_p used for the design of the elements of the wall anchorage system shall not be less than 420 pounds per lineal foot (6.1 kN per lineal meter) of wall substituted for E.

See Section 3.7.4 for minimum design forces in other seismic zones.

2. When elements of the wall anchorage system are not loaded concentrically or are not perpendicular to the wall, the system shall be designed to resist all components of the forces induced by the eccentricity.

3. When pilasters are present in the wall, the anchorage force at the pilasters shall be calculated considering the additional load transferred from the wall panels to the pilasters. However, the minimum anchorage force at a floor or roof shall be that specified in Section 4.5.2.8.1, Item 1.

4. The strength design forces for steel elements of the wall anchorage system shall be 1.4 times the forces otherwise required by this section.

5. The strength design forces for wood elements of the wall anchorage system shall be 0.85 times the force otherwise required by this section and these wood elements shall have a minimum actual net thickness of 2½ in. (63.5 mm).

4.5.2.9 Diaphragms

1. The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

2. Floor and roof diaphragms shall be designed to resist the forces determined in accordance with the following formula:

$$F_{px} = \frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (4.5\text{-}1)$$

The force F_{px} determined from Formula (4.5-1) need not exceed $1.0C_aIw_{px}$, but shall not be less than $0.5C_aIw_{px}$.

When the diaphragm is required to transfer design seismic 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 Formula (4.5-1).

3. Design seismic forces for flexible diaphragms providing lateral supports for walls or frames of masonry or concrete shall be determined using Formula (4.5-1) based on the load determined in accordance with Section 4.2.2 using the value of R not exceeding 4.

4. Diaphragms supporting concrete or masonry walls shall have continuous ties or struts between diaphragm chords to distribute the anchorage forces specified in Section 4.5.2.8.

5. Where wood diaphragms are used to laterally support concrete or masonry walls, the anchorage shall conform to Section 4.5.2.8. In Seismic Zones 2, 3 and 4, anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, wood ledgers or framing shall not be used in cross-grain bending or cross-grain tension, and the continuous ties required by Item 4 shall be in addition to the diaphragm sheathing.

6. Connections of diaphragms to the vertical elements in structures in Seismic Zones 3 and 4, having a plan irregularity of Type 1, 2, 3 or 4 in Table 4-F, shall be designed without considering either the one-third increase or the duration of load increase considered in

allowable stresses for elements resisting earthquake forces.

7. In structures in Seismic Zones 3 and 4 having a plan irregularity of Type 2 in Table 4-F, diaphragm chords and drag members shall be designed considering independent movement of the projecting wings of the structure. Each of these diaphragm elements shall be designed for the more severe of the following two assumptions:

1. Motion of the projecting wings in the same direction.

2. Motion of the projecting wings in opposing directions.

EXCEPTION: This requirement may be deemed satisfied if the procedures of Section 4.3 in conjunction with a three-dimensional model have been used to determine the lateral seismic forces for design.

4.5.2.10 Framing below the base

The strength and stiffness of the framing between the base and the foundation shall not be less than that of the superstructure. The special detailing requirements of Chapter 5, as appropriate, shall apply to columns supporting discontinuous lateral-force-resisting elements and to SMRF and IMRF system elements below the base, which are required to transmit the forces resulting from lateral loads to the foundation.

4.5.2.11 Building separations

All structures shall be separated from adjoining structures. Separations shall allow for the displacement Δ_M. Adjacent buildings on the same property shall be separated by at least Δ_{MT} where

$$\Delta_{MT} = \sqrt{(\Delta_{M1})^2 + (\Delta_{M2})^2} \qquad (4.5\text{-}2)$$

and Δ_{M1} and Δ_{M2} are the displacements of the adjacent buildings.

When a structure adjoins a property line not common to a public way, that structure shall also be set back from the property line by at least the displacement Δ_M of that structure.

EXCEPTION: Smaller separations or property line setbacks may be permitted when justified by rational analyses based on maximum expected ground motions.

4.6 Non-Building Structures

4.6.1 General

4.6.1.1 Scope

Non-building structures include all self supporting structures other than buildings that carry gravity loads and resist the effects of earthquakes. Non-building structures shall be designed to provide the strength required to resist the displacements induced by the minimum lateral forces specified in Section 4.6.

4.6.1.2 Criteria

The minimum design seismic forces prescribed in this section are at a level that produces displacements in a fixed base, elastic model of the structure, comparable to those expected of the real structure when responding to the Design Basis Ground Motion. Reduction in these forces using the coefficient R is permitted where the design of non building structures provides sufficient strength and ductility, consistent with the provisions specified herein for buildings, to resist the effects of seismic ground motions as represented by these design forces. The design of non-building structures shall use the load combinations or factors specified in Section 4.1.1.1 and 4.1.1.2.

4.6.1.3 Weight

The weight, W, for nonbuilding structures shall include all dead loads as defined for buildings in Section 3.2. For purposes of calculating design seismic forces in nonbuilding structures, W shall also include all normal operating contents for items such as tanks, vessels, bins and piping.

4.6.1.4 Period

The fundamental period of the structure shall be determined by rational methods such as by using Method B in Section 4.2.2.2.

4.6.1.5 Drift

The drift limitations of Section 4.2.10 need not apply to nonbuilding structures. Drift limitations shall be established for structural or non structural elements whose failure would cause life hazards. $P\Delta$ effects shall be considered for structures whose calculated drifts exceed the values in Section 4.2.1.2.

4.6.1.6 Interaction effects

In Seismic Zones 3 and 4, structures that support flexible nonstructural elements whose combined weight exceeds 25 percent of the weight of the structure shall be designed considering interaction effects between the structure and the supported elements.

4.6.2 Lateral Force

Lateral-force procedures for nonbuilding structures with structural systems similar to buildings (those with structural systems which are listed in Table 4-G) shall be selected in accordance with the provisions of Section 4.1.

EXCEPTION: Intermediate moment-resisting frames (lMRF) may be used in Seismic Zones 3 and 4 for non building structures in Occupancy Categories 3 and 4 if (1) the structure is less than 50 feet (I5,240 mm) in height and (2) the value R used in reducing calculated member forces and moments does not exceed 2.8.

4.6.3 Rigid Structures

Rigid structures (those with period T less than 0.06 second) and their anchorages shall be designed for the lateral force obtained from Formula (4.6-1).

$$V = 0.7C_a IW \qquad (4.6\text{-}1)$$

The force V shall be distributed according to the distribution of mass and shall be assumed to act in any horizontal direction.

4.6.4 Tanks with Supported Bottoms

Flat bottom tanks or other tanks with supported bottoms, founded at or below grade, shall be designed to resist the seismic forces calculated using the procedures in Section 4.6 for rigid structures considering the entire weight of the tank and its contents. Alternatively, such tanks may be designed using one of the two procedures described below:

1. A response spectrum analysis that includes consideration of the actual ground motion anticipated at the site and the inertial effects of the contained fluid.

2. A design basis prescribed for the particular type of tank by an approved national standard, provided that the seismic zones and

occupancy categories shall be in conformance with the provisions of Sections 4.1.4 and 4.1.2, respectively.

4.6.5 Other Nonbuilding Structures.

Nonbuilding structures that are not covered by Sections 4.6.3 and 4.6.4 shall be designed to resist design seismic forces not less than those determined in accordance with the provisions in Section 4.2 with the following additions and exceptions:

1. The factor R shall be as set forth in Table 4-G. The total design base shear determined in accordance with Section 4.2.2 shall not be less than the following:

$$V = 0.56 C_a I W \qquad (4.6\text{-}2)$$

Additionally, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{1.6 Z N_v I}{R} W \qquad (4.6\text{-}3)$$

2. The vertical distribution of the design seismic forces in structures covered by this section may be determined by using the provisions of Section 4.2.5 or by using the procedures of Section 4.3.

EXCEPTION: For irregular structures assigned to Occupancy Categories 1 and 2 that cannot be modeled as a single mass, the procedures of Section 4.3 shall be used.

The seismic zones and occupancy categories shall be in conformance with the provisions of Sections 4.1.4 and 4.1.2, respectively.

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

TABLE 4-A-MAXIMUM ALLOWABLE DEFLECTION FOR REINFORCED CONCRETE

TYPE OF MEMBER	MEMBER LOADED WITH LIVE LOAD ONLY *(L)*	MEMBER LOADED WITH LIVE LOAD PLUS DEAD LOAD *(L + 2.0 D)*
Roof member supporting plaster or floor member	l/360	l/240

L.-live load.
D.- dead load.
l - length of member in same units as deflection.

TABLE 4-B-SEISMIC ZONE FACTOR Z

Zone	1	2A	2B	3	4
Z	0.10	0.15	0.20	0.30	0.40

Note: The zone shall be determined from the seismic zone Table 4-O

TABLE 4-C-SOIL PROFILE TYPES

SOIL PROFILE TYPE	SOIL PROFILE NAME/GENERIC DESCRIPTION	AVERAGE SOIL PROPERTIES FOR TOP 100 FEET (30 480 mm) OF SOIL PROFILE		
		Shear Wave Velocity, V_S feet/second (m/s)	Standard Penetration Test, $\overline{N}$ [$\overline{N}_{CH}$ for cohesionless soil layers] (blows/foot)	Undrained Shear Strength, $\overline{s}_u$ psf (kpa)
S_A	Hard Rock	> 5,000 (1,500)		
S_B	Rock	2,500 to 5,000 (760 to 1,500)	-	-
S_c	Very Dense Soil and Soft Rock	1,200 to 2,500 (360 to 760)	>50	> 2,000 (100)
S_D	Stiff Soil Profile	600 to 1,200 (180 to 360)	15 to 50	1 ,000 to 2,000 (50 to 100)
SE^1	Soft Soil Profile	<600 (180)	<15	< 1,000 (50)
S_F	Soil Requiring Site-specific Evaluation. See Section 4.1.3.1.			

[1] Soil Profile Type S_E also includes any soil profile with more than 10 feet (3,048 mm) of soft clay defined as a soil with a plasticity index, *PI* > 20, $w_{mc} \geq 40$ percent and $s_u < 500$ psf (24 kPa).

TABLE 4-D-OCCUPANCY CATEGORY

Occupancy Category	Buildings	Importance factor, *I*	Seismic Importance factor , I_p
1	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, etc.	1.25	1.50
2	Hazardous facilities supporting toxic or explosive chemicals or substances, power plants, etc	1.25	1.50
3	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.	1.00	1.25
4	Ordinary buildings, not belonging to above mentioned categories	1.00	1.00
5	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.	1.00	0.90

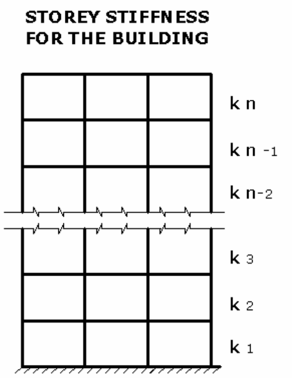
TABLE 4-E-VERTICAL STRUCTURAL IRREGULARITIES

TYPES of IRREGULARITIES
1. Stiffness Irregularity- Soft story
2. Weight (Mass) Irregularity
3. Vertical geometric irregularity
4. In-plane Discontinuity in vertical lateral force resisting element
5. Discontinuity in capacity- weak story

Explanation of Irregularities mentioned in Table 4-E

4-E.1. Stiffness Irregularity (Soft Storey)

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



Soft Storey when

i. $k_i < 0.7\, k_{i+1}$

OR

ii $k_i < 0.8\, (k_{i+1}+k_{i+2}+k_{i+3})/3$

Fig. 4-E.1.1 Stiffness Irregularity (Soft Storey)

4-E.2. Weight (Mass) Irregularity

Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story.

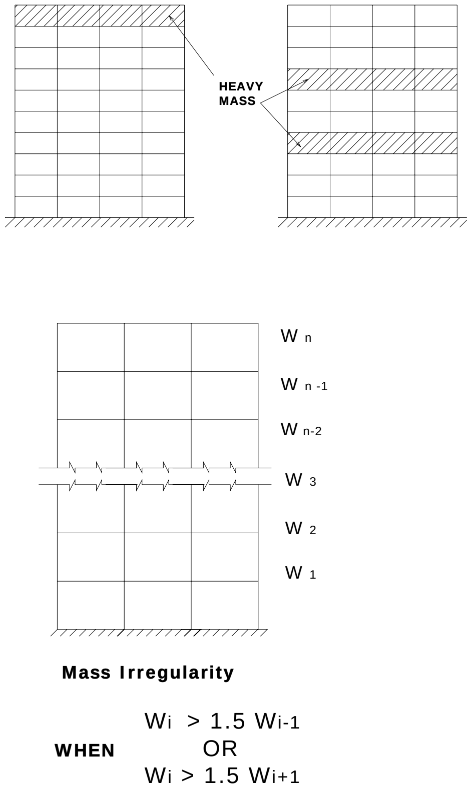


Fig. 4-E.2-1 Mass Irregularity

4-E.3. Vertical Geometric Irregularity

This type of irregularity exits in a building if:

a) For gradual setbacks preserving axial symmetry, the setback at any floor shall be greater than 20 % of the previous plan dimension in the direction of the setback (see Figure **4-E.3-1**.**a** and Figure **4-E.3-1**.**b**);

b) For a single setback within the lower 15 % of the total height of the main structural system, the setback shall be greater than 50 % of the previous plan dimension (see Figure **4-E.3-1**.**c**).

c) If the setbacks do not preserve symmetry, in each face the sum of the setbacks at all story's shall be greater than 30 % of the plan dimension at the ground floor above the foundation or above the top of a rigid basement, and the individual setbacks shall be greater than 10 % of the previous plan dimension (see Figure **4-E.3-1**.**d**).

d) When building is such that a larger dimension is above the smaller dimension, it acts as an inverted pyramid and is particularly undesirable. For gradual setbacks preserving axial symmetry, the total setback shall be greater than 20 % of the smaller dimension (see Figure **4-E.3-1.e**).

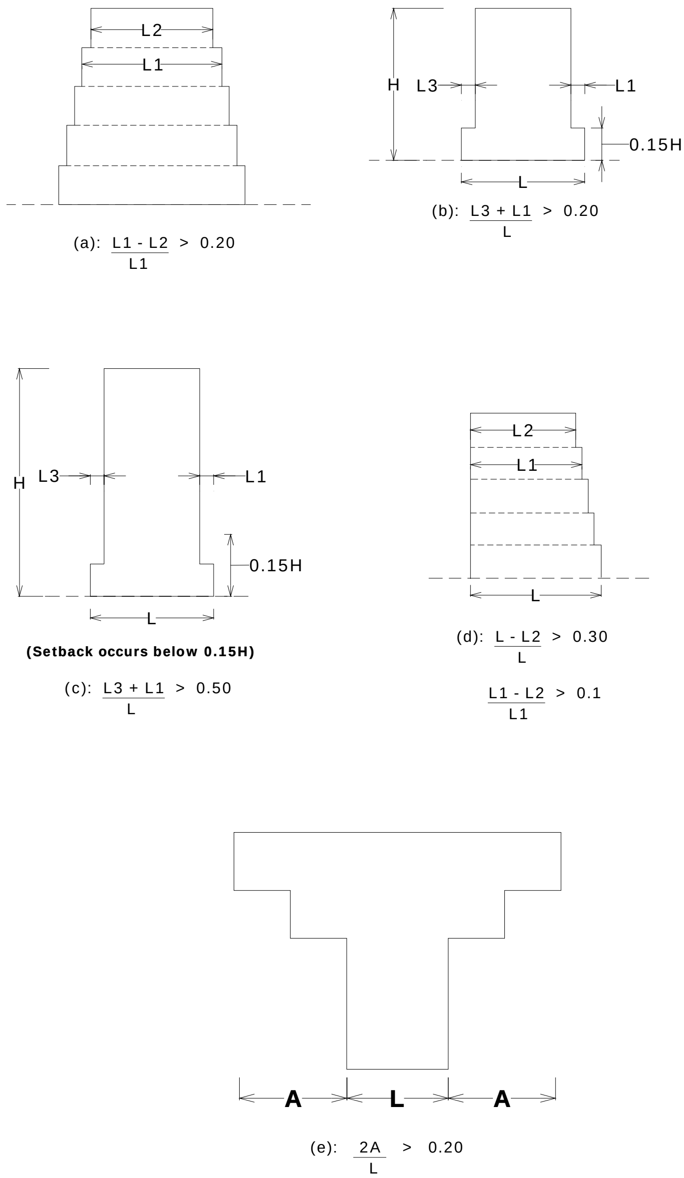


Fig. 4-E.3.1 Criteria for irregularity of buildings with setbacks

4-E.4. In-Plane Discontinuity in Vertical Elements Resisting Lateral Force

An in-plane offset of the lateral force resisting elements greater than the length of those elements

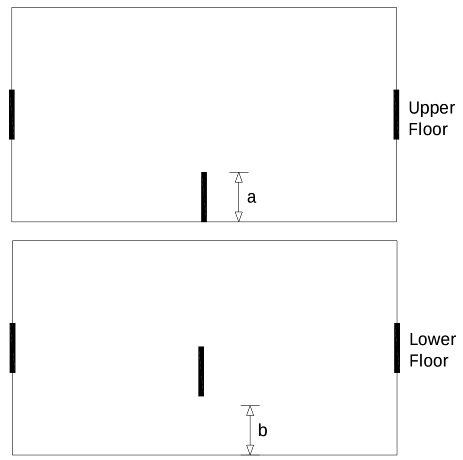


Fig. 4-E.4-1 Plane Discontinuity in vertical elements resisting lateral force when b > a

4-E.5. Discontinuity in capacity – Weak story

A weak story is one in which the story strength is less than 80 percent of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.

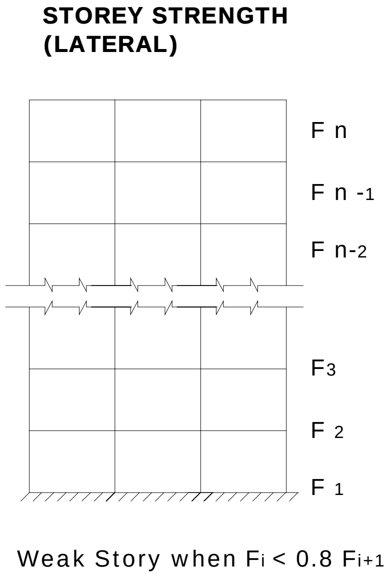


Figure 4.E-5.1 Discontinuity in capacity - Weak story

TABLE 4-F-PLAN STRUCTURAL IRREGULARITIES

TYPE of IRREGULARITIES
1. Torsional irregularity-to be considered when diaphragms are not flexible
2. Re-entrant corners
3. Diaphragm discontinuity
4. Out-of-plane offsets
5. Nonparallel systems

Explanation of Irregularities mentioned in Table 4-F

4-F.1. Torsional Irregularity— to be considered when floor diaphragms are not flexible

Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure(see fig. 4-F.1-1).

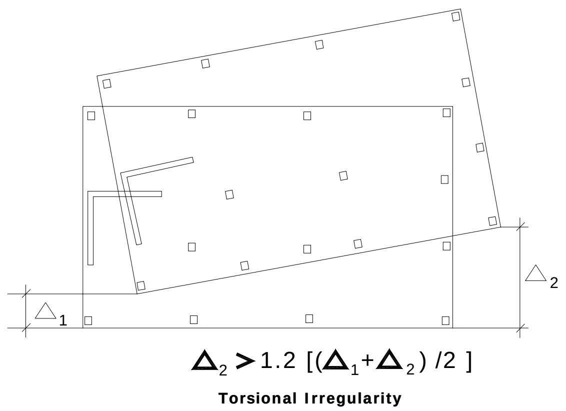
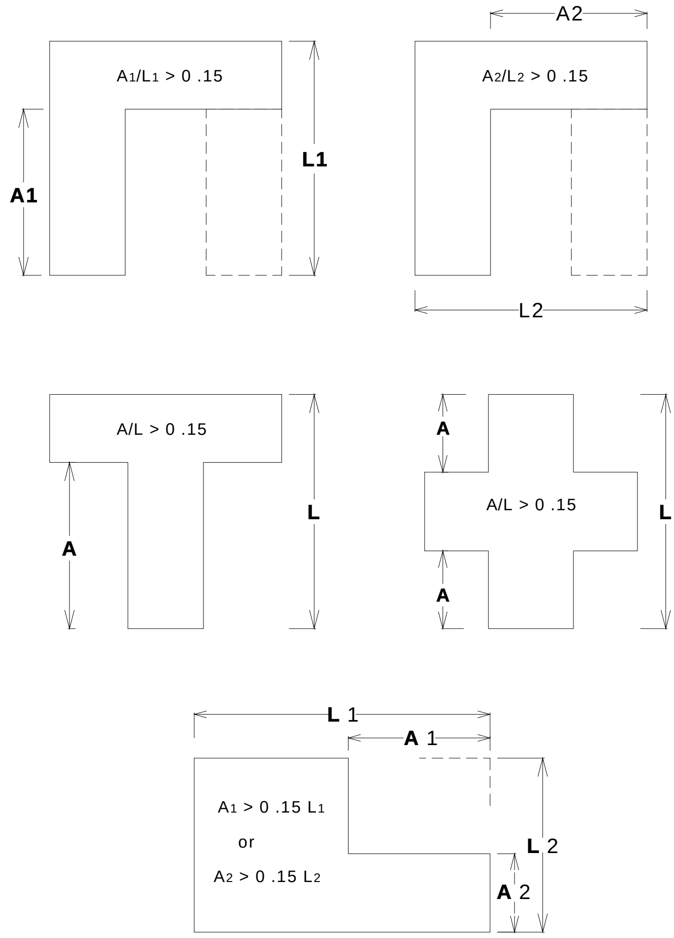


Fig. 4-F.1-1Torsional Irregularity

4-F.2. Re-entrant Corners

A building having a regular configuration can be square, rectangular, or circular. A square or rectangular building with large re-entrant corners would be classified as an irregular configuration. The response of the wings of this type of building is generally different from the response of the building as a whole, and this produces higher local forces than would be determined by application of Section 4.2.2 without modification.
Other plan configurations such as H-shapes that have a geometrical symmetry also would be classified as irregular because of the response of the wings.
Specifications for a building to be classified as irregular with regard to reentrant corners are covered in fig 4-F.2-1



4-F.2-1. Plan Irregularity Due to Reentrant Corners

4-F.3. Diaphragm discontinuity

Diaphragm discontinuity changes the lateral load distribution to different elements as compared to what it would be with continuous floor diaphragm. Diaphragm discontinuity shall be considered in the following cases:

1. The total area of the openings including those of stairs and elevator shafts exceeds 50 of the gross floor area,
2. The local floor openings make it difficult the safe transfer of seismic loads to vertical structural elements,
3. The changes in effective diaphragm stiffness of more than 50 percent from one storey to the next.

Significant differences in stiffness between portions of a diaphragm at a level are also classified as irregularities since they may cause a change in the distribution of seismic forces to the vertical components and create torsional forces not accounted for in the normal distribution considered for a regular building.

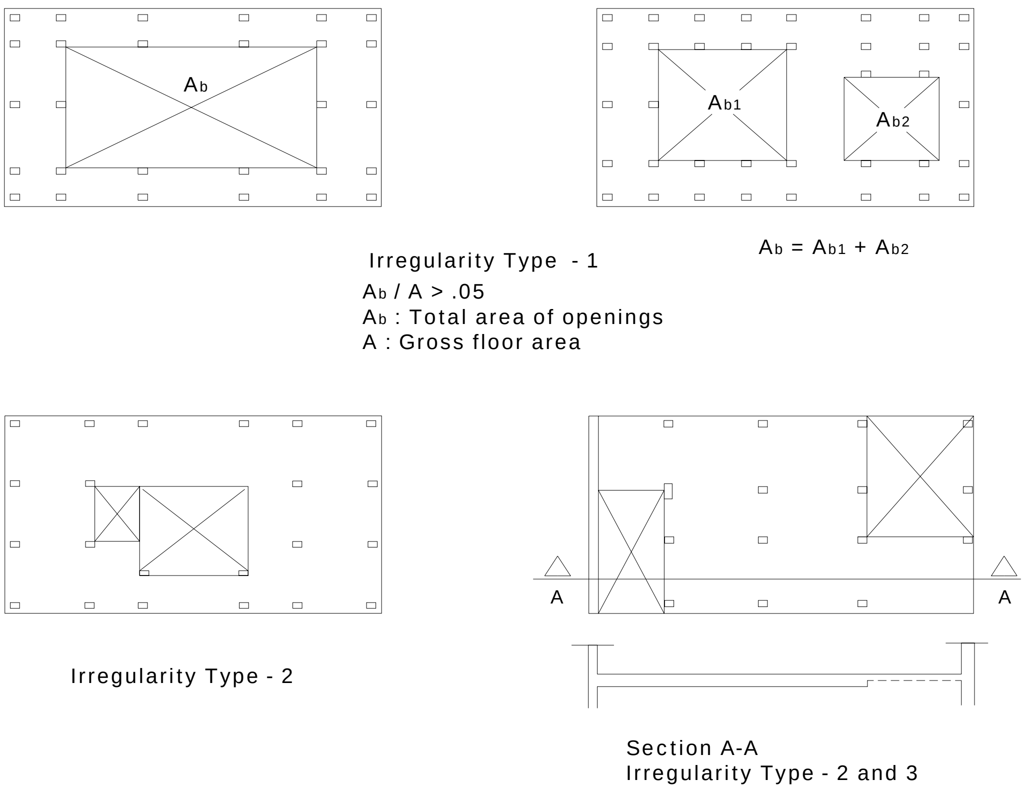


Fig. 4-F.3-1. Diaphragm discontinuity

4-F.4. Out-of-Plane Offsets

The most critical of the discontinuities to be considered is the out-of-plane offset of vertical elements of the seismic-force-resisting elements. Such offsets impose additional vertical and lateral load effects on horizontal elements

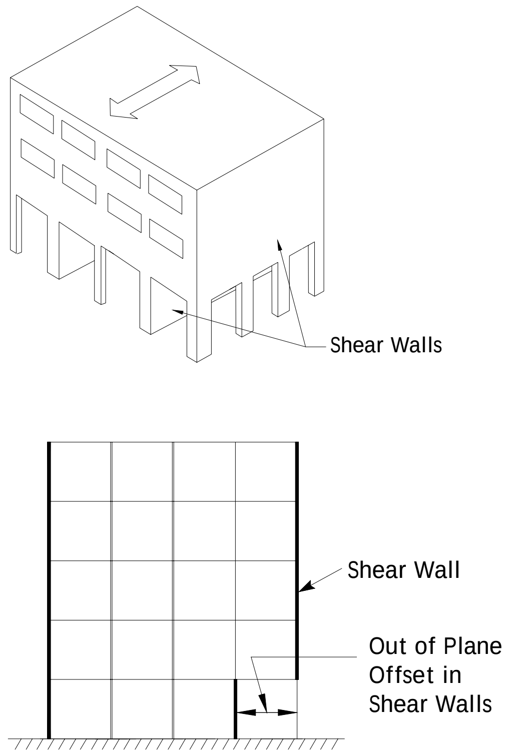


Fig. 4-F.4-1. Out of plan offset

4-F.5. Non-Parallel System

Where vertical elements of the lateral-force-resisting system are not parallel to or symmetric about major orthogonal axes, the static lateral force procedures of Section 4.2.2 cannot be applied as given and, thus, the structure must be considered to be "irregular."

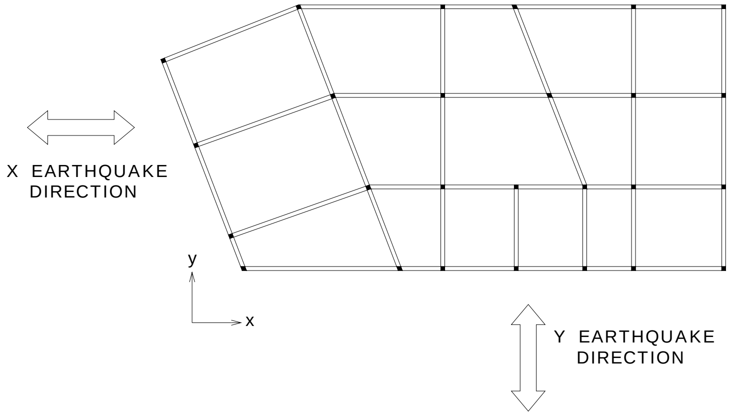


Fig. 4-F.5-1Non-parallel system

Table 4-G-Structural Systems

Basic Structural system	Lateral Force resisting System Description	R	Height Limit for seismic Zone 3 and 4 (feet)
1. Bearing wall system	1. Concrete shear walls 2. Braced concrete frames where bracing carries gravity load 3. Unreinforced masonry 4. Confined masonry 5. Reinforced masonry	3.5 2.25 1.7 2.5 3.5	80 -- Please refer to Chapter 6
2. Building frame system	1. Concrete shear walls 2. Ordinary Concrete braced frames	4 4	120 --
3. Moment Resisting Frames	1. Special concrete Moment resisting frames (SMRF) 2. Intermediate concrete Moment resisting frames (IMRF)	5.5 4.5	150 100
4. Dual systems	1. Shear walls a) Concrete with concrete SMRF b) Concrete with concrete IMRF c) Masonry with concrete IMRF	 5.5 4.5 3.5	 150 100 80
5. Cantilevered column building systems	1. Cantilevered column building	1.7	22

Table 4-H-Horizontal Force Factors a_p and R_p

Element of Structures and Nonstructural Components and Equipment	a_p	R_p
1. Elements of structures A. Walls including the following: (1) Unbraced (cantileverd) parapets.	2.4	1.25
(2) Exterior walls at or above the ground floor (3) Parapets braced above their centres of gravity.	1.0 1.0	2.0 2.0
(4) All interior walls	1.0	2.0
B. Penthouse (expect when framed by an extension of the structural frame)	2.4	2.0
2. Non-structural components A. Exterior and interior ornamentations and appendages	2.4	1.0
B. Chimney, stacks and trussed towers supported on or projecting above the roof; (1) Laterally braced or anchored to the structural frame at a point below their centers of mass.	2.4	1.25
(2) laterally braced or anchored to the structural frame at or above their centers of mass.	1.0	2.0
C. Signs and billboards	2.4	1.25
D. Storage racks (include contents) over 6 feet (1,829 mm) tall.	2.4	2.0
E. Anchorage and lateral bracing for suspended ceilings and light fixtures.	1.0	2.0
F. Masonry or concrete fences over 6 feet (1,829 mm) high	1.0	1.0
G. Partitions	1.0	2.0
3. Equipment A. Tanks and vessels (include contents), including support systems.	1.0	1.0
B. Any flexible equipment laterally braced or anchored to the structural frame at a point below their center of mass.	2.4	1.0

Table 4-I-R Factor for Non-building Structures

Structure Type	R
1. Vessels, including tanks and pressurized spheres, on braced or unbraced legs	1.50
2. Cast-in-place concrete silos and chimneys having wall continuous to the foundations	2.50
3. Distributed mass cantilever structures such as stacks, chimneys, silos and skirt-supported vertical vessels	2.00
4. Trussed towers (freestanding or guyed), guyed stacks and chimney	2.00
5. Cantilevered structures with mass lumped at the top	1.50
6. Cooling towers	2.50
7. Storage racks	2.50
8. All other self-supporting structures not otherwise covered	1.50

Table 4-J-Seismic Coefficient C_a

Soil Profile Type	Seismic Zone Factor, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	0.32 N_v
S_B	0.08	0.15	0.20	0.30	0.40 N_v
S_C	0.09	0.18	0.24	0.33	0.40 N_v
S_D	0.12	0.22	0.28	0.36	0.44 N_v
S_E	0.19	0.30	0.34	0.36	0.36 N_v
S_F	See Footnote[1]				

[1] Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficient for Soil Profile Type S_F

Table 4-K-Seismic Coefficient C_v

Soil Profile Type	Seismic Zone Factor, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	0.32 N_v
S_B	0.08	0.15	0.20	0.30	0.40 N_v
S_C	0.13	0.25	0.32	0.45	0.56 N_v
S_D	0.18	0.32	0.40	0.54	0.64 N_v
S_E	0.26	0.50	0.64	0.84	0.96 N_v
S_F	See Footnote[1]				

[1] Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficient for Soil Profile Type S_F

Table 4-L-NEAR- SOURCE FACTOR N_a[1]

Seismic Source Type	Closest Distance to Known Seismic Source[2]			
	≤ 2 km	5 km	≥ 10 km	≥ 15 km
A	1.5	1.2	1.0	1.0
B	1.3	1.0	1.0	1.0
C	1.0	1.0	1.0	1.0

[1] The Near-Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.
[2] The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e, surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

Table 16-M-NEAR- SOURCE FACTOR N_V[1]

Seismic Source Type	Closest Distance to Known Seismic Source[2]			
	≤ 2 km	5 km	10 km	≥ 15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0

[1] The Near-Source Factor may be based on the linear interpolation of values for distances other than those shown in the table
[2] the closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e, surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value ofr the Near-Source Factor considering all sources shall be used for design.

Table 4-N-SEISMIC SOURCE TYPE[1]

Seismic source Type	Seismic source Description	Seismic source Definition	
		Maximum Moment Magnitude, M	Slip Rate, SR (mm/year)
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity	$M \geq 7.0$	$SR \geq 5$
B	All faults other than type A and C	$M \geq 7.0$ $M < 7.0$ $M \geq 6.5$	$SR < 5$ $SR > 2$ $SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity	$M < 6.5$	$SR \leq 2$

[1] All Seismic sources shall be considered as type A till detailed seismic hazard analysis is carried out for Pakistan

Table 4-O- Recommended Seismic Zone for Earthquake-affected Areas

S. No	District	Tehsil/Town	Recommended Seismic Zone
1	Mansehra	Mansehra	3
		Balakot	4
		Oghi	3
		Tribal Areas Adj.Mansehra Distt.	3
		Battal	3
		Baffa	3
		Shinkiari	3
2	Batagram	Batagram	3
		Allai	3
3	Abbottabad	Abbottabad	3
		Garhi Habibullah	4
		Havelian	3
4	Haripur	Haripur	2b
		Ghazi	2b
	Kohistan	Dassu	2b

Note: Currently there is no authentic seismic zoning map for Pakistan area. The UNGSHAP seismic map (fig 4.3-2) for Pakistan may be used till more detailed research based seismic map is available. However, based on October 8 earthquake damage data, using the intensity vs PGA relations (Ref. 2) and M_w vs PGA relations (Ref. 3) seismic zoning (Table 4-O) for the affected areas (districts) are carried out.

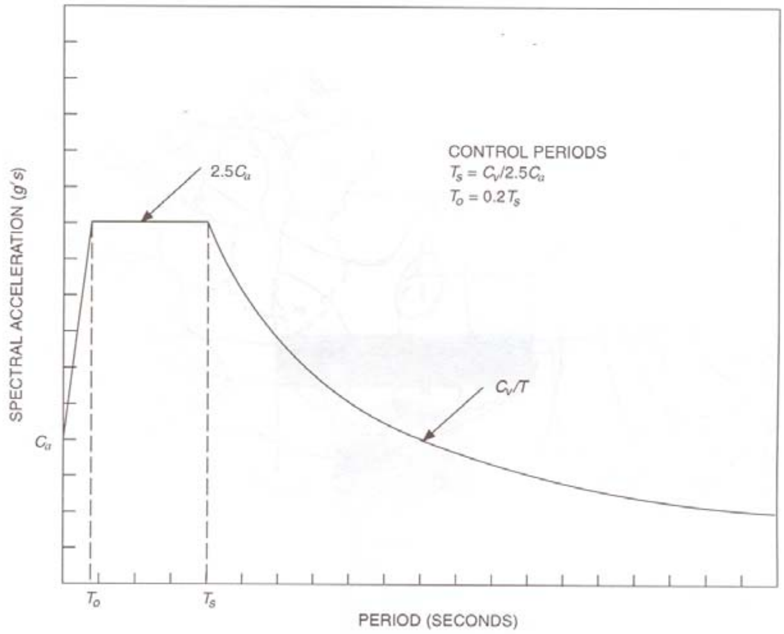


Figure 4.3-1: Design Response Spectra

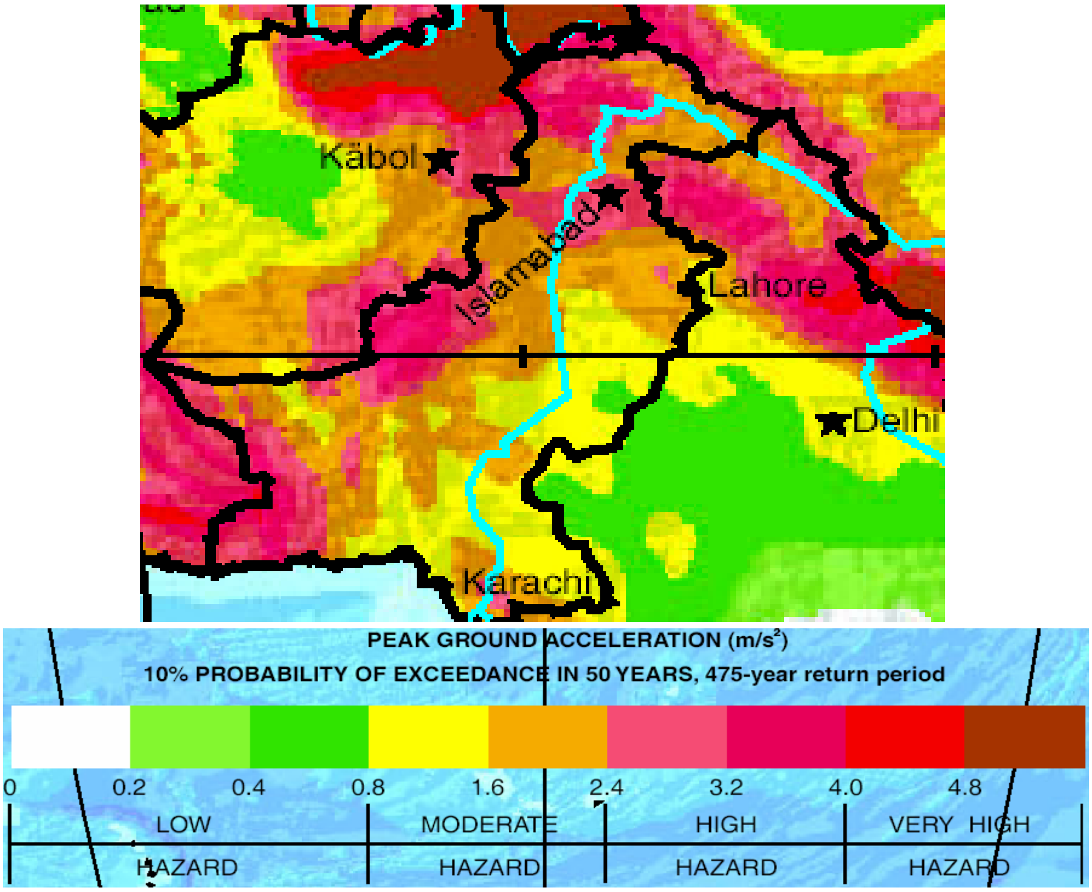


Figure 4.3-2 Seismic Zoning Map as Recommended by United Nation Global Seismic Hazard Assessment Program.

References:

1. Ali, Qaisar, and Khan, Akhtar Naeem, '*A Critical Review of the Seismic Hazard Zoning of Peshawar and Adjoining Areas'*, Journal of Earthquake Engineering, Vol. 9, No. 5, September 2005, pp. 587-607.

2. Murphy, J.R. and O'Brien, J.L., 'The Correlation of Peak Ground Acceleration Amplitude with Seismic Intensity and other Physical Parameter', Bulletin of the Seismological Society of America, 67 (3) (1977), pp. 877-915.

3. Saragoni et al, '*The Chile Earthquake of March 3, 1985*', Earthquake Spectra, Vol. 2, No.2, February 1986, pp. 249-513.

Chapter 5 Special Requirements for Concrete Structures

5.0 General Requirements

5.0.1 Scope

5.0.1.1 This chapter 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.

5.0.1.2 In Seismic Zones 1 and 2, reinforced concrete frames resisting forces induced by earthquake ground motions shall be intermediate moment-resisting frames proportioned to satisfy only Section 5.5.

5.0.1.3 In Seismic Zones 3 and 4, all reinforced concrete structural members that are part of the lateral-force-resisting system shall satisfy the requirements of Sections 5.0 through 5.4.

5.0.1.4 A reinforced concrete structural system not satisfying the requirements of this section may be used if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this section.

5.0.2 Analysis and proportioning of structural members

5.0.2.1 The interaction of all structural and nonstructural members which materially affect the linear and nonlinear response of the structure to earthquake ground motions shall be considered in the analysis.

5.0.2.2 Structural members below base of structure required to transmit to the foundation forces resulting from earthquake effects shall also comply with the requirements of this chapter.

5.0.3 Strength-reduction factors

Strength-reduction factors shall be as given in Annexure A1.

5.0.4 Concrete in members resisting earthquake-induced forces

5.0.4.1 Compressive strength f'_c shall not be less than 3,000 psi (20.69 MPa).

5.0.4.2 Compressive strength of lightweight-aggregate concrete used in design shall not exceed 4,000 psi (27.58 MPa). Lightweight aggregate concrete with higher design compressive strength shall be permitted if demonstrated by experimental evidence that structural members made with that lightweight aggregate concrete provide strength and toughness equal to or exceeding those of comparable members made with normal-weight aggregate concrete of the same strength. In no case shall the compressive strength of lightweight concrete used in design exceed 6,000 psi (41.37 MPa).

5.0.5 Reinforcement in members resisting earthquake-induced forces

5.0.5.1 Alloy A 706 reinforcement
Reinforcement resisting earthquake-induced flexural and axial forces in frame members shall comply with low alloy A 706 except as allowed in Section 5.0.5.2.

5.0.5.2 Billet steel A 615 reinforcement
Billet steel A 615 Grades 40 and 60 reinforcement shall be permitted to be used in frame members if (1) the actual yield strength based on mill tests does not exceed the specified yield strength by more than 18,000 psi (124.1 MPa) [retests shall not exceed this value by more than an additional 3,000 psi (20.69 MPa)], and (2) the ratio of the actual ultimate tensile stress to the actual yield strength is not less than 1.25.

5.0.6 Welded splices and mechanically connected reinforcement

5.0.6.1 Reinforcement resisting earthquake-induced flexural or axial forces in frame members shall be permitted to be spliced using welded splices or mechanical connectors conforming to Annexures A2.1 and A2.2 respectively.

5.0.6.2 Splice locations in frame members shall conform to the following:

1. Welded splices. In Seismic Zones 2, 3 and 4, welded splices on billet steel A 615 or low alloy A 706 reinforcement shall neither be used within an anticipated plastic hinge region nor within a distance of one beam depth on either side of the plastic hinge region or within a joint.

2. Mechanical connection splices. Splices with mechanical connections shall be classified according to strength capacity as follows:

Type 1 splice. Mechanical connections meeting the requirements of Annexure A2.2 and A2.3.

Type 2 splice. Mechanical connections that develop in tension the lesser of 95 percent of the ultimate tensile strength or 160 percent of specified yield strength f_y, of the bar.

Mechanical connection splices shall be permitted to be located as follows:

Type 1 splice. In Seismic Zone 1, a Type 1 splice shall be permitted in any location within a member. In Seismic Zones 2, 3 and 4, a Type 1 splice shall not be used within an anticipated plastic hinge region or within a distance of one beam depth on either side of the plastic hinge region or within a joint.

Type 2 splice. A Type 2 splice shall be permitted in any location within a member.

5.0.6.3 Welding of stirrups, ties, inserts or other similar elements to longitudinal reinforcement required by design shall not be permitted.

5.1 Flexural Members of Frames

5.1.1 Scope

Requirements of this section apply to frame members (1) resisting earthquake-induced forces and (2) proportioned primarily to resist flexure. These frame members shall also satisfy the following conditions:

5.1.1.1 Factored axial compressive force on the member shall not exceed $A_g f'_c/10$.

5.1.1.2 Clear span for the members shall not be less than four times its effective depth.

5.1.1.3 The width-to-depth ratio shall not be less than 0.3.

5.1.1.4 The width shall not be (1) less than 10 in. (254 mm) and (2) 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.

5.1.2 Longitudinal reinforcement

5.1.2.1 At any section of a flexural member, for top as well as for bottom reinforcement, the amount of reinforcement shall not be less than that given by the following formula:

$$A_{s,\min} = \frac{3\sqrt{f_c'}}{f_y} b_w d$$

but not less than 200 b_w d /f_y, (For SI: 1.38 $b_w d/f_y$) and the reinforcement ratio, ρ, shall not exceed 0.025. At least two bars shall be provided continuously, both top and bottom.

5.1.2.2 Positive-moment strength at joint face shall not be less than one half of the negative-moment strength provided at that face of the joint. Neither the negative nor the positive-moment strength at any section along member length shall be less than one fourth the maximum moment strength provided at face of either joint.

5.1.2.3 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 or 4 in. (102 mm). Lap splices shall not be used (1) within the joints, (2) within a distance of twice the member depth from the face of joint, and (3) at locations where analysis indicates flexural yielding caused by inelastic lateral displacements of the frame.

5.1.2.4 Welded splices and mechanical connections shall conform to Section 5.0.6.1.

5.1.3 Transverse reinforcement

5.1.3.1 Hoops shall be provided in the following regions of frame members:

1. Over a length equal to twice the member depth measured from the face of the supporting member toward midspan, at both ends of the flexural members.

2. Over lengths equal to twice the member depth on both sides of a section where flexural yielding may occur in connection with inelastic lateral displacements of the frame.

5.1.3.2 The first hoop shall be located not more than 2 in. (51 mm) from the face of a supporting member. Maximum spacing of the hoops shall not exceed (1) d/4, (2) eight times the diameter of the smallest longitudinal bars, (3) 24 times the diameter of the hoop bars, and (4) 12 in. (305 mm).

5.1.3.3 Where hoops are required, longitudinal bars on the perimeter shall have lateral support conforming to the following:

Ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees and a bar shall be not farther than 6 in. (152 mm) clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie shall be permitted.

5.1.3.4 Where hoops are not required, stirrups with seismic hooks at both ends shall be spaced at a distance not more than d/2 throughout the length of the member.

5.1.3.5 Stirrups or ties required to resist shear shall be hoops over lengths of members as specified in Sections 5.1.3, 5.2.4 and 5.3.2.

5.1.3.6 Hoops in flexural members shall be permitted to be made up of two pieces of reinforcement: a stirrup having seismic hooks at both ends and closed by a crosstie. Consecutive crossties engaging the same longitudinal bar shall have their 90-degree hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the cross ties are confined by a slab on only one side of the flexural frame member, the 90-degree hooks of the crossties shall all be placed on that side.

5.1.4 Shear strength

5.1.4.1 Design forces

The design shear forces V_e shall be determined from consideration of the static forces on the portion of the member between faces of the joint. 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 tributary gravity load along its span.

5.1.4.2 Transverse reinforcement

Transverse reinforcement over the lengths identified in Section 5.1.3.1 shall be proportioned to resist shear assuming $V_c = 0$ when both of the following conditions occur:

1. The earthquake-induced shear force calculated in accordance with Section 5.1.4.1 represents one-half or more of the maximum required shear strength within those lengths.

2. The factored axial compressive force including earthquake effects is less than $A_g f'_c / 20$.

5.2 Frame Members Subjected to Bending and Axial Load

5.2.1 Scope

The requirements of Section 5.2 apply to frame members (1) resisting earthquake-induced forces and (2) having a factored axial force exceeding $A_g f'_c/10$. These frame members shall also satisfy the following conditions:

5.2.1.1

The shortest cross-sectional dimension, measured on a straight line passing through the geometric centroid, shall not be less than 12 in. (305 mm).

5.2.1.2

The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

5.2.2 Minimum flexural strength of columns

5.2.2.1

Flexural strength of any column proportioned to resist a factored axial compressive force exceeding $A_g f'_c /10$ shall satisfy Section 5.2.2.2 or 5.2.2.3.

Lateral strength and stiffness of columns not satisfying Section 5.2.2.2 shall be ignored in determining the calculated strength and stiffness of the structure.

5.2.2.2

The flexural strengths of the columns shall satisfy Formula (5-1).

$$\sum M_e \geq \left(6/5\right) \sum M_g \quad (5\text{-}1)$$

WHERE:

ΣM_e = sum of moments, at the center of the joint, corresponding to the design flexural strength of the columns framing into that joint. Column flexural strength shall be calculated for the factored axial force, consistent with the direction of the lateral forces considered, resulting in the lowest flexural strength.

ΣM_g = sum of moments, at the center 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. Formula (5-1) shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.

5.2.2.3

If Section 5.2.2.2 is not satisfied at a joint, columns supporting reactions from that joint shall be provided with transverse reinforcement as specified in Section 5.2.4 over their full height.

5.2.3 Longitudinal reinforcement

5.2.3.1

The reinforcement ratio ρ_g shall not be less than 0.01 and shall not exceed 0.06.

5.2.3.2

Welded splices and mechanical connections shall conform to Section 5.0.6.1. Lap splices shall be permitted only within the center half of the member length and shall be proportioned as tension splices.

5.2.4 Transverse reinforcement

5.2.4.1

Transverse reinforcement as specified below shall be provided unless a larger amount is required by Section 5.2.5.

1. The volumetric ratio of spiral or circular hoop reinforcement, ρ_s shall not be less than that indicated by Formula (5-2).

$$\rho_s = 0.12 f'_c / f_{yh} \qquad (5\text{-}2)$$

and shall not be less than that required Annexure A8.1

2. The total cross-sectional area of rectangular hoop reinforcement shall not be less than that given by Formulas (5-3) and (5-4).

$$A_{sh} = 0.3\left(sh_c f'_c / f_{yh}\right)\left[\left(A_g / A_{ch}\right) - 1\right] \qquad (5\text{-}3)$$

$$A_{sh} = 0.09\left(sh_c f'_c / f_{yh}\right) \qquad (5\text{-}4)$$

3. Transverse reinforcement shall be provided by either single or overlapping hoops. Crossties of the same bar size and spacing as the hoops shall be permitted to be used. Each end of the crosstie shall engage a peripheral longitudinal reinforcing bar. Consecutive crossties shall be alternated end for end along the longitudinal reinforcement.

4. If the design strength of member core satisfies the requirement of the specified loading combinations including earthquake effect, Annexure A8.1 and Formula (5-3) need not be satisfied.

5. Where the calculated point of contra flexure is not within the middle half of the member clear height, provide transverse reinforcement as specified in Sections 5.2.4.1, Items 1 through 3, over the full height of the member.

5.2.4.2

Transverse reinforcement shall be spaced at distances not exceeding (1) one-quarter minimum member dimension and (2) 4 in. (102 mm). Anchor bolts set in the top of a column shall be enclosed with ties as specified in Section 5.2.4.8.

5.2.4.3

Cross ties or legs of overlapping hoops shall not be spaced more than 14 in. (356 mm) on center in the direction perpendicular to the longitudinal axis of the structural member.

5.2.4.4

Transverse reinforcement in amount specified in Sections 5.2.4.1 through 5.2.4.3 shall be provided over a length l_o from each joint face and on both sides of any section where flexural yielding may occur in connection with inelastic lateral displacements of the frame. The length l_o shall not be less than (1) the depth of the member at the joint face or at the section where flexural yielding may occur, (2) one sixth of the clear span of the member, and (3) 18 in. (457 mm).

5.2.4.5

Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with transverse reinforcement as specified in Sections 5.2.4.1 through 5.2.4.3 over their full height beneath the level at which the discontinuity occurs if the factored axial compressive force in these members, including earthquake effect, exceeds $A_g f'_c/10$. Transverse reinforcement as specified in Sections 5.2.4.1 through 5.2.4.3 shall extend into the discontinued member for at least the development length of the largest longitudinal reinforcement in the column in accordance with Section 5.3.4. If the lower end of the column terminates on a wall, transverse reinforcement as specified in Sections 5.2.4.1 through 5.2.4.3 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 Sections 5.2.4.1 through 5.2.4.3 shall extend at least 12 in. (305 mm) into the footing or mat.

5.2.4.6

Where transverse reinforcement as specified in Sections 5.2.4.1 through 5.2.4.3 is not provided throughout the full length of the column, the remainder of the column length shall contain spiral or hoop reinforcement with center-to-center spacing not exceeding the smaller of six times the diameter of the longitudinal column bars or 6 in. (152 mm).

5.2.4.7

At any section where the design strength, ϕP_n, of the column is less than the sum of the shears V_e computed in accordance with Sections 5.1.4.1 and 5.2.5.1 for all the beams framing into the column above the level under consideration, transverse reinforcement as specified in Sections 5.2.4.1 through 5.2.4.3 shall be provided. For beams framing into opposite sides of the column, the moment components may be assumed to be of opposite sign. For the determination of the design strength, ϕP_n, of the column, these moments may be assumed to result from the deformation of the frame in anyone principal axis.

5.2.4.8 Ties at anchor bolts.

Anchor bolts which are set in the top of a column shall be provided with ties which enclose at least four vertical column bars. Such ties shall be in accordance with Section 5.5.5.5, Item 3, shall be within 5 in. (127 mm) of the top of the column, and shall consist of at least two No.4 or three No.3 bars.

5.2.5 Shear strength requirements

5.2.5.1 Design forces

The design shear force V_e shall be determined from the 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 shear need not exceed those determined from joint strengths based on the probable moment strength, M_{pr} of the transverse members framing in the joint. In no case shall V_e be less than the factored shear determined by analysis of the structure.

5.2.5.2

Transverse reinforcement over the lengths l_o identified in Section 5.2.4.4, shall be proportioned to resist shear assuming $V_c = 0$ when both of the following conditions occur:

1. The earthquake-induced shear force calculated in accordance with Section 5.2.5.1 represents one-half or more of the maximum required shear strength within those lengths.

2. The factored axial compressive force including earthquake effects is less than $A_g f'_c/20$.

5.3 Joints of Frames

5.3.1 General requirements

5.3.1.1

Forces in longitudinal beam reinforcement at the joint face shall be determined by assuming that the stress in the flexural tensile reinforcement is $1.25f_y$.

5.3.1.2

Strength of joint shall be governed by the appropriate strength-reduction factors specified in Annexure A1.

5.3.1.3

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 Section 5.3.4, and in compression according to 5.3.5.

5.3.1.4

Where longitudinal beam reinforcement extends through a beam-column joint, the column dimension parallel to the beam reinforcement shall not be less than 20 times the diameter of the largest longitudinal bar for normal-weight concrete. For lightweight concrete, the dimension shall not be less than 26 times the bar diameter.

5.3.2 Transverse reinforcement

5.3.2.1

Transverse hoop reinforcement as specified in Section 5.2.4 shall be provided within the joint, unless the joint is confined by structural members as specified in Section 5.3.2.2.

5.3.2.2

Within the depth of the shallowest framing member, transverse reinforcement equal to at least one half the amount required by Section 5.2.4.1 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 Section 5.2.4.2 shall be permitted to be increased to 6 in. (152 mm).

5.3.2.3

Transverse reinforcement as required by Section 5.2.4 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.

5.3.3 Shear strength

5.3.3.1

The nominal shear strength of the joint shall not be taken greater than the forces specified below for normal-weight aggregate concrete.

For joints confined on all four faces ------------- $20\sqrt{f'_c}A_j$

(For SI: $1.66\sqrt{f'_c}A_j$)

For joints confined on three faces or on two opposite faces -- $15\sqrt{f'_c}A_j$

(For SI: $1.25\sqrt{f'_c}A_j$)

For others -- $12\sqrt{f'_c}A_j$

(For SI: $1.00\sqrt{f'_c}A_j$)

A member that frames into a face is considered to provide confinement to the joint if at least three fourths 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.

5.3.3.2

For lightweight aggregate concrete, the nominal shear strength of the joint shall not exceed three fourths of the limits for normal-weight aggregate concrete.

5.3.4 Development Length for Reinforcement in Tension

5.3.4.1

The development length, l_{dh}, for a bar with a standard 90-degree hook in normal-weight aggregate concrete shall not be less than $8d_b$, 6 in. (152 mm), and the length required by Formula (5-5).

$$l_{dh} = f_y d_b \Big/ 65\sqrt{f'_c} \qquad (5\text{-}5)$$

For SI: $$l_{dh} = f_y d_b \Big/ 5.4\sqrt{f'_c}$$

for bar sizes No.3 through No.11.

For lightweight aggregate concrete, the development length for a bar with a standard 90-degree hook shall not be less than $10d_b$, 7.5 in. (191 mm), and 1.25 times that required by Formula (5-5).

The 90-degree hook shall be located within the confined core of a column or of a boundary member.

5.3.4.2

For bar sizes No.3 through No. 11, the development length, l_d, for a straight bar shall not be less than (1) 2.5 times the length required by Section 5.3.4.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 12 in. (305 mm), and (2) 3.5 times the length required by Section 5.3.4.1 if the depth of the concrete cast in one lift beneath the bar exceeds 12 in. (305 mm).

5.3.4.3

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.

5.3.5 Development Length for Reinforcement in Compression

5.3.5.1

Development length l_d, in in., for deformed bars in compression shall be computed as the product of the basic development length l_{db} and applicable modification factors as defined in this section, but l_d shall not be less than 8 in. (203 mm).

5.3.5.2

Basic development length l_{db} shall be.

$$0.02 d_b f_y \Big/ \sqrt{f_c^{'}}$$

(For SI: $0.24 d_b f_y \Big/ \sqrt{f_c^{'}}$)

but not less than …………………………….. $0.0003 d_b f_y$

(For SI: $0.044 d_b f_y$)

5.3.5.3

Basic development length l_{db} shall be permitted to be multiplied by applicable factors for:

5.3.5.3.1 Excess reinforcement

Reinforcement in excess of that required by analysis. . . . . ($A_{s\ required}$)/($A_{s\ provided}$)

5.3.5.3.2 Spirals and ties

Reinforcement enclosed within spiral reinforcement not less than ¼ in. (6.4 mm) diameter and not more than 4 in. (102 mm) pitch or within No.4 ties in conformance with Annexure A7 and spaced not more than 4 in. (102 mm) on center. 0.75

5.4 Shear Walls

5.4.1 Scope

The requirements of this section apply to shear walls serving as parts of the earthquake-force-resisting systems.

5.4.2 Reinforcement

5.4.2.1

The reinforcement ratio, ρ_v, for shear walls shall not be less than 0.0025 along the longitudinal and transverse axes. If the design shear force does not exceed $A_{cv}\sqrt{f'_c}$ (For SI: $0.08A_{cv}\sqrt{f'_c}$), the minimum reinforcement for shear walls shall be in conformance with Annexure A3. Reinforcement spacing each way in shear walls shall not exceed 18 in. (457 mm). Reinforcement provided for shear strength shall be continuous and shall be distributed across the shear plane.

5.4.2.2

At least two curtains of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds $2A_{cv}\sqrt{f'_c}$; (For SI: $0.166A_{cv}\sqrt{f'_c}$).

When V_u in the plane of the wall exceeds $A_{cv}\sqrt{f'_c}$ (For SI: $0.08A_{cv}\sqrt{f'_c}$), horizontal reinforcement terminating at the edges of shear walls shall have a standard hook engaging the edge reinforcement, or the edge reinforcement shall be enclosed in "U" stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

5.4.2.3

All continuous reinforcement in shear walls shall be anchored or spliced in accordance with the provisions for reinforcement in tension as specified in Section 5.3.4.

5.4.3 Design forces

The design shear force V_u shall be obtained from the lateral load analysis in accordance with the factored loads and combinations specified in Section 4.1.1.

5.4.4 Shear Strength

5.4.4.1

Nominal shear strength of shear walls shall be determined using either Section 5.4.4.2 or 5.4.4.3.

5.4.4.2

Nominal shear strength, V_n, of shear walls shall be assumed not to exceed the shear force calculated from

$$V_n = A_{cv}\left(2\sqrt{f'_c} + \rho_n f_y\right) \qquad (5\text{-}6)$$

For SI: $V_n = A_{cv}\left(0.166\sqrt{f'_c} + \rho_n f_y\right)$

5.4.4.3

For walls and wall segments having a ratio of (h_w/l_w) less than 2.0, nominal shear strength of wall shall be determined from Formula (5-7)

$$V_n = A_{cv}\left(\alpha_c\sqrt{f'_c} + \rho_n f_y\right) \qquad (5\text{-}7)$$

For SI: $V_n = A_{cv}\left(0.08\alpha_c\sqrt{f'_c} + \rho_n f_y\right)$

Where the coefficient α_c varies linearly from 3.0 for h_w/l_w = 1.5 to 2.0 for h_w/l_w = 2.0.

5.4.4.4

In Section 5.4.4.3 above, the value of ratio (h_w/l_w) used for determining V_n for segments of a wall shall be the largest of the ratios for the entire wall and the segment of wall considered.

5.4.4.5

Walls 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.

5.4.5 Design of shear walls for flexural and axial loads

5.4.5.1

Shear walls and portions of shear walls subject to combined flexural and axial loads shall be designed in accordance with Annexures A4 and A5. The strength-reduction factor ϕ shall be in accordance with Annexure A1.

5.4.5.2

The effective flange widths to be used in the design of I-, L-, C- or T-shaped sections shall not be assumed to extend further from the face of the web than (1) one half the distance to an adjacent shear wall web, or (2) 15 percent of the total wall height for the flange in compression or 30 percent of the total wall height for the flange in tension, not to exceed the total projection of the flange.

5.4.5.3

Walls and portions of walls with P_u>0.35P_o shall not be considered to contribute to the calculated strength of the structure for resisting earthquake-induced forces. Such walls shall conform to the requirements of Section 4.5.2.4.

5.4.5.4

Shear wall boundary zone detail requirements as defined in Section 5.4.5.6 need not be provided in shear walls or portions of shear walls meeting the following conditions:

1. $P_u \leq 0.1 A_g f'_c$ for geometrically symmetrical wall sections $P_u \leq 0.05\ A_g\ f'_c$ for geometrically unsymmetrical wall sections and either

2. $\frac{M_u}{V_u l_w} \leq 1.0$

3. $V_u \leq 3 A_{cv} \sqrt{f'_c}$ and $\frac{M_u}{V_u l_w} \leq 3.0$ (For SI: $V_u \leq 0.25 A_{cv} \sqrt{f'_c}$)

Shear walls and portions of shear walls not meeting the conditions of Section 5.4.5.4 and having $P_u<0.35P_o$ shall have boundary zones at each end a distance varying linearly from 0.25 l_w to 0.15 l_w for P_u varying from 0.35 P_o to 0.15 P_o. The boundary zone shall have minimum length of 0.15 l_w and shall be detailed in accordance with Section 5.4.5.6.

5.4.5.5

Alternatively, the requirements for boundary zones in shear walls or portions of shear walls not meeting the conditions of Section 5.4.5.4 may be based on determination of the compressive strain levels at edges when the wall or portion of wall is subjected to displacement levels resulting from the ground motions specified in Section 4.3.2 using cracked section properties and considering the response modification effects of possible nonlinear behavior of the building.

Boundary zone detail requirements as defined in Section 5.4.5.6 shall be provided over those portions of the wall where compressive strains exceed 0.003. In no instance shall designs be permitted in which compressive strains exceed ε_{max}.

WHERE:

$\varepsilon_{max} = 0.015$ (5-8)

1. Using the displacement of Section 5.4.5.5, determine the curvature of the wall cross section at each location

of potential flexural yielding assuming the possible nonlinear response of the wall and its elements. Using a strain compatibility analysis of the wall cross section, determine the compressive strains resulting from these curvatures.

2. For shear walls in which the flexural limit state response is governed by yielding at the base of the wall, compressive strains at wall edges may be approximated as follows:

Determine the total curvature demand (ϕ_t) as given in Formula (5-9):

$$\phi_t = \frac{\Delta_i}{\left(h_w - l_p/2\right)l_p} + \phi_y \qquad (5\text{-}9)$$

WHERE:
c'_u = neutral axis depth at P'_u and M'_n.

l_p = height of the plastic hinge above critical section and which shall be established on the basis of substantiated test data or may be alternatively taken at $0.5l_w$.

$P'_u = 1.2D + 0.5L + E$

Δ_E = elastic design displacement at the top of the wall using gross section properties and code-specified seismic forces.

Δ_i = inelastic deflection at top of wall.
$= \Delta_t - \Delta_y$

Δ_t = total deflection at the top of the wall equal to t_w, using cracked section properties, or may be taken as $2t_w$, using gross section properties.

Δ_y = displacement at top of wall corresponding to yielding of the tension reinforcement at critical section, or may be taken as $(M'_n/M_E)\Delta_E$, where M_E equals unfactored moment at critical section when top of wall is displaced Δ_E. M'_n is nominal flexural strength of critical section at P'_u.

ϕ_y = yield curvature which may be estimated as $0.003/l_w$

If ϕ_t is less than or equal to $0.003/c'_u$, boundary zone details as defined in Section 5.4.5.6 are not required. If ϕ_t exceeds $0.003/c'_u$, the compressive strains may be assumed to vary linearly over the depth c'_u and have maximum value equal to the product of c'_u and ϕ_t.

5.4.5.6 Shear wall boundary zone detail requirements

When required by Section 5.4.5.1 through 5.4.5.5, boundary zones shall meet the following:

1. Dimensional requirements.

1.1 All portions of the boundary zones shall have a thickness of $l_u/16$ or greater.

1.2 Boundary zones shall extend vertically a distance equal to the development length of the largest vertical bar within the boundary zone above the elevation where the requirements of Section 5.4.5.4 or 5.4.5.5 are met.

Extensions below the base of the boundary zone shall conform to Section 5.2.4.6.

EXCEPTION: The boundary zone reinforcement need not extend above the base of the boundary zone a distance greater than the larger of l_w or $M_u/4V_u$.

1.3 Boundary zones as determined by the requirements of Section 5.4.5.5 shall have a minimum length of 18 in. (457 mm) at each end of the wall or portion of wall.

1.4 In I-, L-, C- or T-shaped sections, the boundary zone at each end shall include the effective flange width and shall extend at least 12 in. (305 mm) into the web.

2. Confinement reinforcement.

2.1 All vertical reinforcement within the boundary zone shall be confined by hoops or cross ties producing an area of steel not less than:

$$A_{sh} = 0.09 s h_c f'_c \big/ f_{yh} \qquad (5\text{-}10)$$

2.2 Hoops and cross ties shall have a vertical spacing not greater than the smaller of 6 in. (152 mm) or 6 diameters of the largest vertical bar within the boundary zone.

2.3 The ratio of the length to the width of the hoops shall not exceed 3. All adjacent hoops shall be overlapping.

2.4 Cross ties or legs of overlapping hoops shall not be spaced further apart than 12 in. (305 mm) along the wall.

2.5 Alternate vertical bars shall be confined by the corner of a hoop or cross tie.

3. Horizontal reinforcement.

3.1 All horizontal reinforcement terminating within a boundary zone shall be anchored in accordance with Section 5.4.2.

3.2 Horizontal reinforcement shall not be lap spliced within the boundary zone.

4. Vertical reinforcement.

4.1 Vertical reinforcement shall be provided to satisfy all tension and compression requirements.

4.2 Area of reinforcement shall not be less than 0.005 times the area of boundary zone or less than two No.5 bars at each edge of boundary zone.

4.3 Lap splices of vertical reinforcement within the boundary zone shall be confined by hoops or cross ties. Spacing of hoops and cross ties confining lap-spliced reinforcement shall not exceed 4 in. (102 mm).

5.4.5.7

Welded splices and mechanical connections of longitudinal reinforcement in the boundary zone shall conform to Section 5.0.6.1.

5.4.6 Construction joints

5.4.6.1

All construction joints in walls and diaphragms shall conform to Annexure A6, and contact surfaces shall be roughened to full amplitude of approximately ¼ in. (6.4 mm).

5.5 Requirements for Frames in Seismic Zones 1 and 2

5.5.1 General

In Seismic Zones 1 and 2, structural frames proportioned to resist forces induced by earthquake ground motions shall satisfy the requirements of this Section.

5.5.2 Steel Reinforcement

Reinforcement details in a frame member shall satisfy Section 5.5.4 if the factored compressive axial load for the member does not exceed ($A_g f'_c$/l0). If the factored compressive axial load is larger, frame reinforcement details shall satisfy Section 5.5.5 unless the member has spiral reinforcement according to Annexure A8.1.

5.5.3 Shear Design

Design shear strength of beams, columns and two-way slabs resisting earthquake effect shall not be less than either (1) 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 gravity loads, or (2) the maximum shear obtained from design load combinations which include earthquake effect E.

5.5.4 Beams

5.5.4.1

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 of the joint. Neither the negative- nor the positive-moment strength at any section along the length of the member shall be less than one fifth the maximum moment strength provided at the face of either joint.

5.5.4.2

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 at not more than 2 in. (51 mm) from the face of the supporting member. Maximum stirrup spacing shall not exceed (1) d/4, (2) eight times the diameter of the smallest longitudinal bar enclosed, (3) 24 times the diameter of the stirrup bar, and (4) 12 in. (305 mm).

5.5.4.3

Stirrups shall be placed at not more than d/2 throughout the length of the member.

5.5.5 Columns

5.5.5.1

Maximum tie spacing shall not exceed s_o over a length l_o measured from the joint face. Spacing s_o shall not exceed (1) eight times the diameter of the smallest longitudinal bar enclosed, (2) 24 times the diameter of the tie bar, (3) one half of the smallest cross-sectional dimension of the frame member, and (4) 12 in. (305 mm). Length l_o shall not be less than (1) one sixth of the clear span of the member, (2) maximum cross-sectional dimension of the member, and (3) 18 in. (457 mm).

5.5.5.2

The first tie shall be located at not more than $s_o/2$ from the joint face.

5.5.5.3

Joint reinforcement not less than required by $A_v = 50\dfrac{b_w s}{f_y}$ within the column for a depth not less than that of the deepest connection of framing elements to the columns.

5.5.5.4

Tie spacing shall not exceed twice the spacing s_o.

5.5.5.5

Columns lateral ties shall be as specified below:

1. No. 5 bar and smaller, 90-degree bend , plus 6 d_b extension at free end of bar or

2. No. 6, No. 7 and No. 8 bar, 90-degree bend, plus 12 d_b extension at free end of bar, or

3. No. 8, smaller, 135-degree bend, plus 6 d_b extension at free end of bar.

5.5.5.6

Anchor bolts set in the top of a column shall be enclosed with ties as specified in Section 5.2.4.8.

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Chapter 6 Analysis and Design of Masonry Buildings

6.0 General

6.0.1 Scope

The provisions of this chapter apply to the design of buildings of unreinforced, confined and reinforced masonry in seismic regions.

6.1 Properties of Materials

6.1.1 Minimum strength of masonry units

The compressive strength of masonry units shall be not less than the minimum values as follows:

Type of masonry units	**$f_{b,min}$, psi (MPa)**	**$f_{bh,min}$, psi (MPa)**
Solid fired clay brick	1200 (8.27)	450 (3.10)
Solid/Hollow* concrete block	800 (5.52)	300 (2.07)

$f_{b,min}$ = Compressive strength normal to the bed face

$f_{bh,min}$ = Compressive strength parallel to the bed face in the plane of wall

*** The compressive strength of hollow concrete block shall be determined using gross area of unit.**

6.1.2 Maximum amount of voids in masonry unit

The volumetric percentage of voids shall not be greater than 40% of total volume of the masonry unit

6.1.3 Minimum strength of masonry mortar

The compressive strength of masonry mortar shall not be less than 600 psi (4.14 MPa) and not greater than 75% of the compressive strength of the masonry unit in the direction normal to the bed face.

6.2 Types of construction and response modification factors

6.2.1 Type of Masonry Construction

Depending on the masonry type used for the seismic resistant elements, masonry buildings shall be assigned to one of the following types of construction:

a) Unreinforced masonry construction;

b) Confined masonry construction;

c) Reinforced masonry construction;

NOTE Frames with infill masonry are not covered in this chapter.

6.2.2 Limitation on unreinforced masonry

Unreinforced brick masonry, properly designed and detailed for gravity load, shall only be allowed in areas with design peak ground acceleration up to 0.20g.

6.2.3 Response Modification Factor

For buildings constructed with masonry systems which provide an enhanced ductility of the structure, specific values of the response modification factor R (other than as given in table 4-G) may be used, provided that the system and the related values for *R* are verified experimentally.

6.3 Structural analysis

6.3.1

The structural model for the analysis of the building shall represent the stiffness properties of the entire system.

6.3.2

The stiffness of the structural elements shall be evaluated taking into account both their flexural and shear flexibility and, if relevant, their axial flexibility.

6.3.3

In the absence of an accurate evaluation of the stiffness properties, substantiated by rational analysis, the cracked bending and shear stiffness may be taken as one half of the gross section uncracked elastic stiffness.

6.3.4

In the structural model masonry spandrels may be taken into account as coupling beams between two wall elements if they are regularly bonded to the adjoining walls and connected both to the floor tie beam and to the lintel below.

6.3.5

If the structural model takes into account the coupling beams, a frame analysis may be used for the determination of the action effects in the vertical and horizontal structural elements.

6.3.6

For non-linear static analysis the drift of a wall at the failure, Δ_{uw}, of masonry buildings shall be taken as follows:

Failure mode	Drift at failure, Δ_{uw}
Diagonal tension	0.004 α h
Sliding	0.006 α h
Rocking	0.008 α h

Where:

α = 1.00 for unreinforced masonry,
1.25 for confined masonry, and
1.50 for reinforced masonry

h = height of the masonry wall

6.4 Design Criteria and Construction Rules

6.4.1 General

6.4.1.1

Masonry buildings shall be composed of floors and walls, which are connected in two orthogonal horizontal directions and in the vertical direction.

6.4.1.2

Any type of floors may be used, provided that the general requirements of continuity and effective diaphragm action are satisfied.

6.4.1.3

The connection between the floors (categorized as flexible diaphragms) and walls shall be provided with reinforced concrete ring beams.

6.4.1.4

Shear walls shall be provided in at least two orthogonal directions.

6.4.1.5

The effective thickness of shear walls, t_{ef}, shall not be less than a minimum value, $t_{ef,min}$, given in Table 6-A.

6.4.1.6

The ratio of the effective wall height, h_{ef}, to its effective thickness, t_{ef}, shall not exceed a maximum value, $(h_{ef}/t_{ef})_{max}$, given in Table 6-A.

6.4.1.7

Shear walls not conforming to the minimum geometric requirements of Sections 6.4.1.5 and 6.4.1.6 may be considered as secondary seismic elements.

6.4.1.8

For buildings having flexible diaphragm, horizontal concrete beams shall be placed in the plane of the wall at every floor level. These beams shall form continuous bonding elements physically connected to each other. Beams continuous over the entire periphery are essential.

6.4.1.9

The beam mentioned in section 6.4.1.8 shall have a minimum depth of 9 in. (229 mm) and shall be reinforced with a minimum longitudinal steel area of 1.0% of the gross area of the cross-section, or 0.8 in^2 (516 mm^2), whichever is greater. The stirrups shall be at least ¼ in. (6.4 mm) in diameter and spaced at not more than 4½ in. (114 mm)

6.4.1.10

All masonry walls of the building shall be connected at the lintel level by a continuous reinforced concrete beam. The beam shall have a minimum depth of 6 in. (152 mm) and shall be reinforced with a minimum longitudinal steel area of 1.0% of the gross area of the cross-section, or 0.40 in^2 (258 mm^2), whichever is greater. The stirrups shall be at least ¼ in. (6.4 mm) in diameter and spaced at not more than 6 in. (152 mm). Such beams, however, need not to be provided in buildings located in Zone 1.

6.4.1.11

The minimum bearing length of lintel beam on each end of the lintel beam shall be equal to 9 in. (229 mm).

6.4.1.12

Concrete beam shall also be provided in case of gable walls with a minimum depth of 9 in. (229 mm). The band shall be reinforced with a minimum longitudinal steel area of 1.0% of the gross area of the cross-section, or 0.8 in^2 (512 mm^2), whichever is greater. The stirrups shall be at least ¼ in. (6.4 mm) in diameter and spaced at not more than 6 in. (152 mm).

6.4.1.13

If the height of the gable exceeds 4 ft (1,219 mm), vertical concrete element/elements shall be provided with a minimum width of 6 in. (152 mm). The element shall be reinforced with a minimum longitudinal steel area of 1.0%

of the gross area of the cross-section, or 0.40 in^2 (258 mm^2), whichever is greater. The stirrups shall be at least ¼ in. (6.4 mm) in diameter and spaced at not more than 6 in. (152 mm).

6.4.2 Openings in Walls

6.4.2.1

The ratio of the length of the wall, *l*, to the greater clear height, *h*, of the openings adjacent to the wall, shall not be less than a minimum value, $(l/h)_{min}$, a given in Table 6-A.

6.4.2.2

The total length of openings in a wall shall not exceed the limits as mentioned in table 6-B

6.4.2.3

The horizontal distance between two openings (pier width) shall be not less than:

i. 50% the height of the shorter opening, but not less than 24 in. (610 mm) for Zone 2, 3 and 4
ii. 40% the height of the shorter opening, but not less than 18 in. (457 mm) for Zone 1

6.4.3 Additional requirements for confined masonry

6.4.3.1

The horizontal and vertical confining elements shall be bonded together and anchored to the elements of the main structural system.

6.4.3.2

Masonry walls shall be tied to the vertical confining elements by providing horizontal reinforcement. 2 - ¼ in. (6.4 mm) diameter bars spaced at a maximum spacing of 24 in. (610 mm). These bars should be properly anchored with ties of vertical confining elements and shall also be extended in to the wall up to a distance of 1/3 of the wall length or 4 ft. (1,219 mm), whichever is smaller.

6.4.3.3

In order to obtain an effective bond between the confining elements and the masonry, the concrete of the confining elements shall be cast after the masonry has been built.

6.4.3.4

The cross-sectional dimensions of both horizontal and vertical confining elements shall not be less than 6 in. (152 mm). In double-leaf walls the thickness of confining elements shall assure the connection of the two leaves and their effective confinement.

6.4.3.5

Vertical confining elements shall be placed

i. At the free edges of each structural wall element;
ii. At both sides of any wall opening with an area of more than:
    a) 15 ft^2 (1.40 m^2) for Zone 3 and 4,
    b) 20 ft^2 (1.86 m^2) for Zone 2 and
    c) 25ft^2 (2.32 m^2) for Zone 1
iii. Within the wall if necessary in order not to exceed a spacing of 15 ft (4,572 mm) between the confining elements;
iv. At the intersections of structural walls, wherever the confining elements imposed by the above provisions are at a distance larger than 5 ft (1,524 mm) for Zones 2, 3, and 4 and 7 ft (2,134 mm) for Zone 1.

6.4.3.6

Horizontal confining elements shall be placed in the plane of the wall at every floor level and in any case with a vertical spacing of not more than 12 ft. (3,658 mm).

6.4.3.7

The longitudinal reinforcement of confining elements shall not have a cross-sectional area less than 0.4 in^2 (258 mm^2), nor than 1% of the cross-sectional area of the confining element.

6.4.3.8

Stirrups not less than ¼ in. (6.4 mm) in diameter and spaced not more than 6 in. (152 mm) shall be provided around the longitudinal reinforcement.

6.4.3.9

Lap splices shall not be less than 60 bar diameters in length.

6.4.4 Additional requirements for reinforced masonry

6.4.4.1

Horizontal reinforcement shall be placed in the bed joints or in suitable grooves in the units, with a vertical spacing not exceeding 27 in. (686 mm).

6.4.4.2

Masonry units with recesses shall accommodate the reinforcement needed in lintels and parapets.

6.4.4.3

Reinforcing steel bars of not less than ¼ in. (6.4 mm) diameter, bent around the vertical bars at the edges of the wall, shall be used.

6.4.4.4

The minimum percentage of horizontal reinforcement in the wall, normalized with respect to the gross area of the section, shall neither be less than 0.09% nor greater than 0.5 %.

6.4.4.5

The vertical reinforcement spread in the wall, as a percentage of the gross area of the horizontal section of the wall, shall neither be less than 0.06% nor greater than 1 %.

6.4.4.6

In any case high percentages of vertical and horizontal reinforcement leading to compressive failure of the units prior to the yielding of the steel, shall be avoided.

6.4.4.7

Vertical reinforcement shall be located in pockets, cavities or holes in the units.

6.4.4.8

Vertical reinforcements with a cross-sectional area of not less than 0.2 in^2 (129 mm^2) shall be arranged:

i. Within 12 in (305 mm) from the free edge of the wall element;
ii. At every wall intersection;
iii. Within the wall, in order not to exceed a spacing of 10 ft (3,048 mm) between such reinforcements;
iv. Within 12 in (305 mm) from free edge on all sides of the openings having smaller dimension greater than 18 in. (457 mm).

6.4.4.9

Spacing between vertical reinforcement shall neither exceed two times the thickness of wall nor 36 in (914 mm).

6.4.4.10

Lap splices shall not be less than 60 bar diameters in length.

6.4.4.11

The parapets and lintels shall be regularly bonded to the masonry of the adjoining walls and linked to them by horizontal reinforcement.

6.5 Safety verification

6.5.1

The verification of the building's safety against collapse shall be explicitly provided, except for buildings satisfying the rules for "simple masonry buildings" given in **6.6.2**.

6.6 Rules for "simple masonry buildings"

6.6.1

The simple masonry buildings are recommended only for occupancy category 4 of Table 4-D.

6.6.2

Simple masonry buildings shall fulfill the following conditions, besides the conditions of plan and elevation regularity defined in Tables 4-E, and 4-F.

6.6.2.1

The walls of the buildings shall be continuous from the foundations to the top of the building.

6.6.2.2

A minimum of two parallel systems of walls should be placed in two orthogonal directions. The gross length of wall, openings free, shall not be less than 50 % of the dimension of the building in the same direction. Computing the gross length shall be included the walls that comply with the geometric requirements in Table 6-A.

6.6.2.3

The distance between these two systems of walls in the orthogonal direction of their longitudinal layout should be greater than 75 % of the length in the same direction (orthogonal to the walls).

6.6.2.4

At least 75 % of the vertical loads should be supported by the shear walls.

6.6.2.5

In each of the two directions, walls resistant to horizontal forces should be placed with a maximum spacing of:

1. 22 ft (6,706 mm) for Unreinforced masonry buildings
2. 25 ft (7,620 mm) for Confined Masonry buildings
3. 30 ft (9,144 mm) for Reinforced masonry buildings

6.6.2.6

The inter-storey height should not be greater than 12 ft. (3,658 mm).

6.6.2.7

For each storey, the ratio between the total resistant cross-sectional area of the walls and the total floor area, should not be less than the values indicated in the following Table 6-C for each of the two orthogonal directions:

6.6.2.8

Additionally, for each storey should be verified that:

$$\sigma = N / A \leq 0.15 f_k$$

where:

N is the total vertical load at the base of the considered wall.

A is the total area of the load-bearing walls (for vertical loads) of the same floor.

f_k is the compressive strength in vertical direction of the masonry.

Table 6-A: Geometric requirements for shear walls

Type of Masonry	$t_{ef,min}$, in (mm)	$(h_{ef} / t_{ef})_{max}$	$(l/h)_{min}$ *
Unreinforced, with dressed natural stone units	15 (381)	6	0.5
Unreinforced, with any other type of units	9 (229)	12	0.4
Confined masonry	9 (229)	15	0.3**
Reinforced masonry	9 (229)	15	No restriction

* $(l/h)_{min}$ shall not be less than 24" in any case. $(l/h)_{min}$ may be reduced by 20% for zone 1 and 2

** Opening can be provided at the end of wall provided vertical confining element is provided on both sides of opening.

Table 6-B: Maximum openings in term ratio of total length of openings to the length of wall between consecutive cross walls

No. of stories (story height =10 ft)	Seismic Zone				
	1	2A	2B	3	4
1-Story building	0.60	0.50	0.50	0.45	0.45
2-Story building	0.50	0.45	0.45	0.40	0.40
≥3-Story building	0.45	0.40	0.40	0.35	0.35

Table 6-C Area of the resistant walls in each orthogonal direction for simple buildings

Seismic Coefficient C_a		≤0.1 g	≤0.15 g	≤0.20 g	≤0.30 g	≤0.40 g	≤0.50 g
Type of structure	Number of storeys						
Unreinforced brick/block masonry	1	3.50 %	4.00%	4.00 %	5.50%	6.00%	
	2	4.00 %	4.50%	5.00%	6.00%	6.50%	
	3	4.50 %	5.00%	5.50 %	6.50%		
Confined masonry	1	3.00%	3.50%	3.50%	4.00%	4.50%	5.00%
	2	4.00%	4.00%	4.00%	4.50%	5.50%	6.00%
	3	4.50%	4.50%	5.00%	5.50%	6.00%	
	4	5.00%	5.50%	5.50%	6.00%		
Reinforced masonry	1	3.00 %	3.00%	3.00 %	3.50 %	4.00 %	4.50 %
	2	3.50 %	3.50%	3.50 %	4.00 %	5.00 %	5.00 %
	3	4.00 %	4.00%	4.00 %	5.00 %	5.50 %	6.00 %
	4	4.50 %	5.00%	5.00 %	5.50 %	6.00 %	6.50 %

Annexures

Annexure A1
Strength Reduction Factors

Strength-reduction factor ϕ shall be as follows:

Flexure, without axial load.	0.90
Axial load and axial load with flexure:	
Axial tension and axial tension with flexure	0.90
Axial compression and axial compression with flexure:	
Members with spiral reinforcement conforming to A8.1	0.75
Other reinforced members	0.70

except that for low values of axial compression, ϕ shall be permitted to be increased in accordance with the following:

For members in which f_y does not exceed 60,000 psi (413.7 MPa), with symmetric reinforcement, and with $(h - d' - d_s)/h$ not less than 0.70, ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from 0.10 f'_c Ag to zero.

For other reinforced members. ϕ shall be permitted to be increased linearly to 0.90 as ϕP_n decreases from 0.10 f'_c Ag or ϕP_b, whichever is smaller, to zero.

Shear and torsion	0.85
Bearing on concrete	0.70

Annexure A2
Welded and Mechanical Splices

A2.1 A full-welded splice shall develop at least 125 percent of specified yield strength, f_y, of the bar.

A2.2 A full mechanical connection shall develop in tension or compression, as required, at least 125 percent of specified yield strength f_y of the bar.

A2.3 Welded splices or mechanical connections not meeting the requirements of Section A2.1 or A2.2 are allowed for No.5 bars and smaller when the area of reinforcement provided is at least twice that required by analysis, and the following requirements are met:

A2.3.1 Splices shall be staggered at least 24 in. (610 mm) and in such manner as to develop at every section at least twice the calculated tensile force at that section but not less than 20,000 psi (137.9 MPa) for total area of reinforcement provided.

A2.3.2 In computing tensile forces developed at each section, rate the spliced reinforcement at the specified splice strength. Unspliced reinforcement shall be rated at that fraction of f_y defined by the ratio of the shorter actual development length to l_d required to develop the specified yield strength f_y.

Annexure A3
Minimum Reinforcement for Walls

A3.1 Minimum vertical and horizontal reinforcement shall be in accordance with Annexures A3.2 and A3.3.

A3.2 Minimum ratio of vertical reinforcement area to gross concrete area shall be:

1. 0.0012 for deformed bars not larger than No.5 with a specified yield strength not less than 60,000 psi (413.7 MPa), or
2. 0.0015 for other deformed bars, or
3. 0.0012 for welded wire fabric (plain or deformed) not larger than W31 or D31.

A3.3 Minimum ratio of horizontal reinforcement area to gross concrete area shall be:

1. 0.0020 for deformed not larger than No.5 with a specified yield strength not less than 60,000 psi (413.7 MPa), or
2. 0.0025 for other deformed bars, or
3. 0.0020 for welded wire fabric (plain or deformed) not larger than W31 or D31.

A3.4 Walls more than 10 in. (254 mm) thick, except basement walls, shall have reinforcement for each direction placed in two layers parallel with faces of wall in accordance with the following:

1. One layer consisting of not less than one half and not more than two thirds of total reinforcement required for each direction shall be placed not less than 2 in. (51 mm) or more than one third the thickness of wall from exterior surface.
2. The other layer, consisting of the balance of required reinforcement in that direction, shall be placed not less than 3/4 in. (19 mm) or more than one third the thickness of wall from interior surface.

A3.5 Vertical and horizontal reinforcement shall not be spaced farther apart than three times the wall thickness, nor 18 in. (457 mm). Unless otherwise required by the engineer, the upper- and lowermost horizontal reinforcement shall be placed within one half of the specified spacing at the top and bottom of the wall.

A3.6 Vertical reinforcement need not be enclosed by lateral ties if vertical reinforcement area is not greater than 0.0 I times gross concrete area, or where vertical reinforcement is not required as compression reinforcement.

A3.7 In addition to the minimum reinforcement required by Annexures A3.1, not less than two No.5 bars shall be provided around all window and door openings. Such bars shall be extended to develop the bar beyond the comers of the openings but not less than 24 in. (610 mm).

Annexure A4
Design Assumptions

A4.1 Strength design of members for flexure and axial loads shall be based on assumptions given in the following items and on satisfaction of applicable conditions of equilibrium and compatibility of strains.

A4.2 Maximum usable strain at extreme concrete compression fiber shall be assumed equal to 0.003.

A4.3 Stress in reinforcement below specified yield strength f_y for grade of reinforcement used shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y, stress in reinforcement shall be considered independent of strain and equal to f_y.

A4.4 Tensile strength of concrete shall be neglected in axial and flexural calculations of reinforced concrete.

A4.5 Relationship between concrete compressive stress distribution and concrete strain shall be assumed to be rectangular, trapezoidal, parabolic or any other shape that results in prediction of strength in substantial agreement with results of comprehensive tests.

A4.6 Requirements of Annexure A4.5 may be considered satisfied by an equivalent rectangular concrete stress distribution defined by the following:

1. Concrete stress of $0.85f'c$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $\alpha = \beta_1 c$ from the fiber of maximum compressive strain.

2. Distance c from fiber of maximum strain to the neutral axis shall be measured in a direction perpendicular to the axis.

3. Factor β_1 shall be taken as 0.85 for concrete strengths $f'c$ up to and including 4,000 psi (27.58 MPa). For strengths above 4,000 psi (27.58 MPa), β_1 shall be reduced continuously at a rate of 0.05 for each 1,000 psi (6.89 MPa) of strength in excess of 4,000 psi (27.58 MPa), but β_1 shall not be taken less than 0.65.

Annexure A5
General Principles and Requirements

A5.1 Design of cross section subject to flexure or axial loads or to combined flexure and axial loads shall be based on stress and strain compatibility using assumptions in Annexure A4.

A5.2 Balanced strain conditions exist at a cross section when tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as concrete in compression reaches its assumed ultimate strain of 0.003.

A5.3 For flexural members, and for members subject to combined flexure and compressive axial load when the design axial load strength (ϕP_n is less than the smaller of 0.1 0 f'_c Ag or ϕP_b , the ratio of reinforcement ρ provided shall not exceed 0.75 of the ratio ρ_b that would produce balanced strain conditions for the section under flexure without axial load. For members with compression reinforcement, the portion of ρ_b equalized by compression reinforcement need not be reduced by the 0.75 factor.

A5.4 Use of compression reinforcement shall be permitted in conjunction with additional tension reinforcement to increase the strength of flexural members.

A5.5 Design axial load strength ϕP_n of compression members shall not be taken greater than the following:

1. For members with tie reinforcement conforming to Annexure A7:

   $$\phi P_{n(\max.)} = 0.8\phi\left[0.85 f_c^{'}\left(A_g - A_{st}\right) + f_y A_{st}\right]$$

2. For members with spiral reinforcement conforming to Annexure A8:

   $$\phi P_{n(\max.)} = 0.85\phi\left[0.85 f_c^{'}\left(A_g - A_{st}\right) + f_y A_{st}\right]$$

Annexure A6
Construction Joints

A6.1 Surface of concrete construction joints shall be cleaned and laitance removed.

A6.2 Immediately before new concrete is placed, all construction joints shall be wetted and standing water removed.

A6.3 Construction joints shall be so made and located as not to impair the strength of the structure. Provision shall be made for transfer of shear and other forces through construction joints.

A6.4 Construction joints in floors shall be located within the middle third of spans of slabs, beams and girders. Joints in girders shall be offset a minimum distance of two times the width of intersecting beams.

A6.5 Beams, girders or slabs supported by columns or walls shall not be cast or erected until concrete in the vertical support members is no longer plastic.

A6.6 Beams, girders, haunches, drop panels and capitals shall be placed monolithic ally as part of a slab system, unless otherwise shown in design drawings or specifications.

Annexure A7
Ties Reinforcement

Tie reinforcement for compression members shall conform to the following:

A7.1 All bars shall be enclosed by lateral ties, at least No.3 in size for longitudinal bars No. 10 or smaller, and at least No.4 in size for Nos. 11, 14 and 18 and bundled longitudinal bars. Deformed wire or welded wire fabric of equivalent area shall be permitted.

A7.2 Vertical spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie bar or wire diameters, or least dimension of the compression member.

A7.3 Ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 degrees and a bar shall be not farther than 6 in. (152 mm) clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie shall be permitted.

A7.4 Ties shall be located vertically not more than one half a tie spacing above the top of footing or slab in any story and shall be spaced as provided herein to not more than one half a tie spacing below the lowest horizontal reinforcement in members supported above.

A7.5 Where beams or brackets frame from four directions into a column, termination of ties not more than 3 in. (76 mm) below reinforcement in shallowest of such beams or brackets shall be permitted.

A7.6 Column ties shall have hooks as specified in Section 5.5.5.5.

Annexure A8
Spiral Reinforcement

Spiral reinforcement for compression members shall conform to the following:

A8.1 Ratio of spiral reinforcement ρ_s shall not be less than the value given by

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c^{'}}{f_y}$$

Where f_y is the specified yield strength of spiral reinforcement but not more than 60,000 psi (413.7 MPa).

A8.2 Spirals shall consist of evenly spaced continuous bar or wire of such size and so assembled as to permit handling and placing without distortion from designed dimensions.

A8.3 For cast-in-place construction, size of spirals shall not be less than 3/8 in. (9.5 mm) diameter.

A8.4 Clear spacing between spirals shall not exceed 3 in. (76 mm) or be less than 1 in. (25 mm).

A8.5 Anchorage of spiral reinforcement shall be provided by one and one-half extra turns of spiral bar or wire at each end of a spiral unit.

A8.6 Splices in spiral reinforcement shall be lap splices of $48d_b$, but not less than 12 in. (305 mm) or welded.

A8.7 Spirals shall extend from top of footing or slab in any story to level of lowest horizontal reinforcement in members supported above.

A8.8 Where beams or brackets do not frame into all sides of a column, ties shall extend above termination of spiral to bottom of slab or drop panel.

A8.9 In columns with capitals, spirals shall extend to a level at which the diameter or width of capital is two times that of the column.

A8.10 Spirals shall be held firmly in place and true to line.