Chapter 2

LOADS ON BUILDINGS AND STRUCTURES

2.1 INTRODUCTION

2.1.1 Scope

This Chapter specifies the minimum design forces including dead load, live load, wind and earthquake loads, miscellaneous loads and their various combinations. These loads shall be applicable for the design of buildings and structures in conformance with the general design requirements provided in Chapter 1.

2.1.2 Limitations

Provisions of this Chapter shall generally be applied to majority of buildings and other structures covered in this Code subject to normally expected loading conditions. For those buildings and structures having unusual geometrical shapes, response characteristics or site locations, or for those subject to special loading including tornadoes, special dynamic or hydrodynamic loads etc., site-specific or case-specific data or analysis may be required to determine the design loads on them. In such cases, and all other cases for which loads are not specified in this Chapter, loading information may be obtained from reliable references or specialist advice may be sought. However, such loads shall be applied in compliance with the provisions of other Parts or Sections of this Code.

2.1.3 Terminology

The following definitions apply only to the provisions of this Chapter:

ALLOWABLE STRESS DESIGN METHOD (ASD)	A method for proportioning structural members such that the maximum stresses due to service loads obtained from an elastic analysis does not exceed a specified allowable value. This is also called Working Stress Design Method (WSD).
APPROVED	Acceptable to the authority having jurisdiction.
BASE	The level at which the earthquake motions are considered to be imparted to the structures or the level at which the structure as a dynamic vibrator is supported.
BASE SHEAR	Total design lateral force or shear due to earthquake at the base of a structure.
BASIC WIND SPEED, *V*	Three-second gust speed at 10 m above the ground in Exposure B (Sec 2.4.6.3) having a return period of 50 years.
BEARING WALL SYSTEM	A structural system without a complete vertical load carrying space frame.
BRACED FRAME	An essentially vertical truss system of the concentric or eccentric type provided to resist lateral forces.
BUILDING, ENCLOSED	A building that does not comply with the requirements for open or partially enclosed buildings.
BUILDING ENVELOPE	Cladding, roofing, exterior walls, glazing, door assemblies, window assemblies, skylight assemblies, and other components enclosing the building.
BUILDING, LOW-RISE	Enclosed or partially enclosed buildings that comply with the following conditions 1. Mean roof height *h* less than or equal to 18.3 m. 2. Mean roof height *h* does not exceed least horizontal dimension.

BUILDING, OPEN — A building having each wall at least 80 percent open. This condition is expressed for each wall by the equation $A_o \leq 0.8A_g$ where,

A_o = total area of openings in a wall that receives positive external pressure (m^2).

A_g = the gross area of that wall in which A_o is identified (m^2).

BUILDING, PARTIALLY ENCLOSED — A building that complies with both of the following conditions:

1. The total area of openings in a wall that receives positive external pressure exceeds the sum of the areas of openings in the balance of the building envelope (walls and roof) by more than 10 percent.
2. The total area of openings in a wall that receives positive external pressure exceeds 0.37 m^2 or 1 percent of the area of that wall, whichever is smaller, and the percentage of openings in the balance of the building envelope does not exceed 20 percent.

These conditions are expressed by the following equations:

1. $A_o \Sigma 1.10A_{oi}$
2. $A_o \Sigma 0.37m^2$ or $\Sigma 0.01A_g$, whichever is smaller, and $A_{oi}/A_{gi} \leq 0.20$

Where, A_o, A_g are as defined for open building

A_{oi} = the sum of the areas of openings in the building envelope (walls and roof) not including A_o, in m^2.

A_{gi} = the sum of the gross surface areas of the building envelope (walls and roof) not including A_g, in m^2.

BUILDING, SIMPLE DIAPHRAGM — A building in which both windward and leeward wind loads are transmitted through floor and roof diaphragms to the same vertical MWFRS (e.g., no structural separations).

BUILDING FRAME SYSTEM — An essentially complete space frame which provides support for gravity loads.

BUILDING OR OTHER STRUCTURE, FLEXIBLE — Slender buildings or other structures that have a fundamental natural frequency less than 1 Hz.

BUILDING OR OTHER STRUCTURE, REGULAR SHAPED — A building or other structure having no unusual geometrical irregularity in spatial form.

BUILDING OR OTHER STRUCTURES, RIGID — A building or other structure whose fundamental frequency is greater than or equal to 1 Hz.

CAPACITY CURVE — A plot of the total applied lateral force, V_j, versus the lateral displacement of the control point, δ_j, as determined in a nonlinear static analysis.

COMPONENTS AND CLADDING — Elements of the building envelope that do not qualify as part of the MWFRS.

CONTROL POINT — A point used to index the lateral displacement of the structure in a nonlinear static analysis.

CRITICAL DAMPING — Amount of damping beyond which the free vibration will not be oscillatory.

CYCLONE PRONE REGIONS — Areas vulnerable to cyclones; in Bangladesh these areas include the Sundarbans, southern parts of Barisal and Patuakhali, Hatia, Bhola, eastern parts of Chittagong and Cox's Bazar

DAMPING — The effect of inherent energy dissipation mechanisms in a structure (due to sliding, friction, etc.) that results in reduction of effect of vibration, expressed as a percentage of the critical damping for the structure.

DESIGN ACCELERATION RESPONSE SPECTRUM — Smoothened idealized plot of maximum acceleration of a single degree of freedom structure as a function of structure period for design earthquake ground motion.

DESIGN EARTHQUAKE	The earthquake ground motion considered (for normal design) as two-thirds of the corresponding Maximum Considered Earthquake (MCE).
DESIGN FORCE, F	Equivalent static force to be used in the determination of wind loads for open buildings and other structures.
DESIGN PRESSURE, p	Equivalent static pressure to be used in the determination of wind loads for buildings.
DESIGN STRENGTH	The product of the nominal strength and a resistance factor.
DIAPHRAGM	A horizontal or nearly horizontal system of structures acting to transmit lateral forces to the vertical resisting elements. The term "diaphragm" includes reinforced concrete floor slabs as well as horizontal bracing systems.
DUAL SYSTEM	A combination of a Special or Intermediate Moment Resisting Frame and Shear Walls or Braced Frames designed in accordance with the criteria of Sec 1.3.2.4
DUCTILITY	Capacity of a structure, or its members to undergo large inelastic deformations without significant loss of strength or stiffness.
EAVE HEIGHT, h	The distance from the ground surface adjacent to the building to the roof eave line at a particular wall. If the height of the eave varies along the wall, the average height shall be used.
ECCENTRIC BRACED FRAME (EBF)	A steel braced frame designed in conformance with Sec 10.20.15.
EFFECTIVE WIND AREA, A	The area used to determine GC_p. For component and cladding elements, the effective wind area as mentioned in Sec 2.4.11 is the span length multiplied by an effective width that need not be less than one-third the span length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.
EPICENTRE	The point on the surface of earth vertically above the focus (point of origin) of the earthquake.
ESCARPMENT	Also known as scarp, with respect to topographic effects in Sec 2.4.7, a cliff or steep slope generally separating two levels or gently sloping areas (see Figure 6.2.4).
ESSENTIAL FACILITIES	Buildings and structures which are necessary to remain functional during an emergency or a post disaster period.
FACTORED LOAD	The product of the nominal load and a load factor.
FLEXIBLE DIAPHRAGM	A floor or roof diaphragm shall be considered flexible, for purposes of this provision, when the maximum lateral deformation of the diaphragm is more than two times the average storey drift of the associated storey. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm under lateral load with the storey drift of adjoining vertical resisting elements under equivalent tributary lateral load.
FLEXIBLE ELEMENT OR SYSTEM	An element or system whose deformation under lateral load is significantly larger than adjoining parts of the system.
FREE ROOF	Roof (monoslope, pitched, or troughed) in an open building with no enclosing walls underneath the roof surface.
GLAZING	Glass or transparent or translucent plastic sheet used in windows, doors, skylights, or curtain walls.
GLAZING, IMPACT RESISTANT	Glazing that has been shown by testing in accordance with ASTM E1886 and ASTM E1996 or other approved test methods to withstand the impact of wind-borne missiles likely to be generated in wind-borne debris regions during design winds.
HILL	With respect to topographic effects in Sec 2.4.7, a land surface characterized by strong relief in any horizontal direction (Figure 6.2.4).

HORIZONTAL BRACING SYSTEM	A horizontal truss system that serves the same function as a floor or roof diaphragm.
IMPACT RESISTANT COVERING	A covering designed to protect glazing, which has been shown by testing in accordance with ASTM E1886 and ASTM E1996 or other approved test methods to withstand the impact of wind-borne debris missiles likely to be generated in wind-borne debris regions during design winds.
IMPORTANCE FACTOR, WIND LOAD	A factor that accounts for the degree of hazard to human life and damage to property.
IMPORTANCE FACTOR, EARTHQUAKE LOAD	It is a factor used to increase the design seismic forces for structures of importance.
INTENSITY OF EARTHQUAKE	It is a measure of the amount of ground shaking at a particular site due to an earthquake
INTERMEDIATE MOMENT FRAME (IMF)	A concrete or steel frame designed in accordance with Sec 8.3.10 or Sec 10.20.10 respectively.
LIMIT STATE	A condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).
LIQUEFACTION	State in saturated cohesionless soil wherein the effective shear strength is reduced to negligible value due to pore water pressure generated by earthquake vibrations, when the pore water pressure approaches the total confining pressure. In this condition, the soil tends to behave like a liquid.
LOAD EFFECTS	Forces, moments, deformations and other effects produced in structural members and components by the applied loads.
LOAD FACTOR	A factor that accounts for unavoidable deviations of the actual load from the nominal value and for uncertainties in the analysis that transforms the load into a load effect.
LOADS	Forces or other actions that arise on structural systems from the weight of all permanent constructions, occupants and their possessions, environmental effects, differential settlement, and restrained dimensional changes. Permanent loads are those loads in which variations in time are rare or of small magnitude. All other loads are variable loads.
MAGNITUDE OF EARTHQUAKE	The magnitude of earthquake is a number, which is a measure of energy released in an earthquake.
MAIN WIND-FORCE RESISTING SYSTEM (MWFRS)	An assemblage of structural elements assigned to provide support and stability for the overall structure. The system generally receives wind loading from more than one surface.
MAXIMUM CONSIDERED EARTHQUAKE (MCE)	The most severe earthquake ground motion considered by this Code.
MEAN ROOF HEIGHT, *h*	The average of the roof eave height and the height to the highest point on the roof surface, except that, for roof angles of less than or equal to 10^{o}, the mean roof height shall be the roof heave height.
MODAL MASS	Part of the total seismic mass of the structure that is effective in mode k of vibration.
MODAL PARTICIPATION FACTOR	Amount by which mode k contributes to the overall vibration of the structure under horizontal and vertical earthquake ground motions.

MODAL SHAPE COEFFICIENT	When a system is vibrating in a normal mode, at any particular instant of time, the vibration amplitude of mass i expressed as a ratio of the vibration amplitude of one of the masses of the system, is known as modal shape coefficient
MOMENT RESISTING FRAME	A frame in which members and joints are capable of resisting lateral forces primarily by flexure. Moment resisting frames are classified as ordinary moment frames (OMF), intermediate moment frames (IMF) and special moment frames (SMF).
NOMINAL LOADS	The magnitudes of the loads such as dead, live, wind, earthquake etc. specified in Sections 2.2 to 2.6 of this Chapter.
NOMINAL STRENGTH	The capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modelling effects and differences between laboratory and field conditions.
NUMBER OF STOREYS (n)	Number of storeys of a building is the number of levels above the base. This excludes the basement storeys, where basement walls are connected with ground floor deck or fitted between the building columns. But, it includes the basement storeys, when they are not so connected.
OPENINGS	Apertures or holes in the building envelope that allow air to flow through the building envelope and that are designed as "open" during design winds as defined by these provisions.
ORDINARY MOMENT FRAME (OMF)	A moment resisting frame not meeting special detailing requirements for ductile behaviour.
PERIOD OF BUILDING	Fundamental period (for 1st mode) of vibration of building for lateral motion in direction considered.
P-DELTA EFFECT	It is the secondary effect on shears and moments of frame members due to action of the vertical loads due to the lateral displacement of building resulting from seismic forces.
RATIONAL ANALYSIS	An analysis based on established methods or theories using mathematical formulae and actual or appropriately assumed data.
RECOGNIZED LITERATURE	Published research findings and technical papers that are approved.
RESISTANCE FACTOR	A factor that accounts for unavoidable deviations of the actual strength from the nominal value and the manner and consequences of failure. This is also known as strength reduction factor.
RESPONSE REDUCTION FACTOR	It is the factor by which the actual base shear force that would develop if the structure behaved truly elastic during earthquake, is reduced to obtain design base shear. This reduction is allowed to account for the beneficial effects of inelastic deformation (resulting in energy dissipation) that can occur in a structure during a major earthquake, still ensuring acceptable response of the structure.
RIDGE	With respect to topographic effects in Sec 2.4.7, an elongated crest of a hill characterized by strong relief in two directions (Figure 6.2.4).
SEISMIC DESIGN CATEGORY	A classification assigned to a structure based on its importance factor and the severity of the design earthquake ground motion at the site.
SEISMIC-FORCE-RESISTING SYSTEM	That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces.
SHEAR WALL	A wall designed to resist lateral forces acting in its plane (sometimes referred to as a vertical diaphragm or a structural wall).
SITE CLASS	Site is classified based on soil properties of upper 30 m.

SITE-SPECIFIC DATA	Data obtained either from measurements taken at a site or from substantiated field information required specifically for the structure concerned.
SOFT STOREY	Storey in which the lateral stiffness is less than 70 percent of the stiffness of the storey above or less than 80 percent of the average lateral stiffness of the three storeys above.
SPACE FRAME	A three-dimensional structural system without bearing walls composed of members interconnected so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.
SPECIAL MOMENT FRAME (SMF)	A moment resisting frame specially detailed to provide ductile behaviour complying with the seismic requirements provided in Chapters 8 and 10 for concrete and steel frames respectively.
STOREY	The space between consecutive floor levels. Storey-x is the storey below level-x.
STOREY DRIFT	The horizontal deflection at the top of the story relative to bottom of the storey.
STOREY SHEAR	The total horizontal shear force at a particular storey (level).
STRENGTH	The usable capacity of an element or a member to resist the load as prescribed in these provisions.
STRENGTH DESIGN METHOD	A method of proportioning structural members using load factors and resistance factors satisfying both the applicable limit state conditions. This is also known as Load Factor Design Method (LFD) or Ultimate Strength Design Method (USD).
TARGET DISPLACEMENT	An estimate of the maximum expected displacement of the control point calculated for the design earthquake ground motion in nonlinear static analysis.
VERTICAL LOAD-CARRYING FRAME	A space frame designed to carry all vertical gravity loads.
WEAK STOREY	Storey in which the lateral strength is less than 80 percent of that of the storey above.
WIND-BORNE DEBRIS REGIONS	Areas within cyclone prone regions located: 1. Within 1.6 km of the coastal mean high water line where the basic wind speed is equal to or greater than 180 km/h or 2. In areas where the basic wind speed is equal to or greater than 200 km/h.
WORKING STRESS DESIGN METHOD (WSD)	See ALLOWABLE STRESS DESIGN METHOD.

2.1.4 Symbols and Notation

The following symbols and notation apply only to the provisions of this Chapter:

A = Effective wind area, in m^2

A_f = Area of open buildings and other structures either normal to the wind direction or projected on a plane normal to the wind direction, in m^2.

A_g = Gross area of that wall in which A_o is identified, in m^2.

A_{gi} = Sum of gross surface areas of the building envelope (walls and roof) not including A_g, in m^2

A_o = Total area of openings in a wall that receives positive external pressure, in m^2.

A_{oi} = Sum of the areas of openings in the building envelope (walls and roof) not including A_o, in m^2

A_{og} = Total area of openings in the building envelope in m^2

A_c = Gross area of the solid freestanding wall or solid sign, in m^2

A_s = Torsion amplification factor at level-x.

B	=	Horizontal dimension of building measured normal to wind direction, in m.
C_d	=	Deflection amplification factor.
C_f	=	Force coefficient to be used in determination of wind loads for other structures
C_N	=	Net pressure coefficient to be used in determination of wind loads for open buildings
C_p	=	External pressure coefficient to be used in determination of wind loads for buildings
C_c	=	Normalized acceleration response spectrum.
C_t	=	Numerical coefficient to determine building period
D	=	Diameter of a circular structure or member in m (as used in Sec 2.4).
D	=	Dead loads, or related internal moments and forces, Dead load consists of: a) weight of the member itself, b) weight of all materials of construction incorporated into the building to be permanently supported by the member, including built-in partitions, c) weight of permanent equipment (as used in Sec 2.7).
D^u	=	Depth of protruding elements such as ribs and spoilers in m.
E	=	Total load effects of earthquake that include both horizontal and vertical, or related internal moments and forces. The horizontal seismic load effect shall include system overstrength factor, Ω_o, if applicable. For specific definition of the earthquake load effect, E, see sec 2.5.
E_h	=	Horizontal seismic load effect when the effect of system overstrength factor, Ω_o, is not included.
E_{mh}	=	Horizontal seismic load effect when the effect of system overstrength factor, Ω_o, is included.
E_v	=	Vertical effect of seismic load.
F	=	Design wind force for other structures, in N (as used in Sec 2.4).
F	=	Loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights or related internal moments and forces (as used in Sec 2.7).
F_a	=	Loads due to flood or tidal surge or related internal moments and forces.
F_i, F_n, F_s	=	Design lateral force applied to level-i, -n, or -x respectively.
F_c	=	Lateral forces on an element or component or on equipment supports.
G	=	Gust effect factor
G_f	=	Gust effect factor for MWFRSs of flexible buildings and other structures
GC_p	=	Product of external pressure coefficient and gust effect factor to be used in determination of wind loads for buildings
GC_{pf}	=	Product of the equivalent external pressure coefficient and gust-effect factor to be used in determination of wind loads for MWFRS of low-rise buildings
GC_{pi}	=	Product of internal pressure coefficient and gust effect factor to be used in determination of wind loads for buildings
GC_{pn}	=	Combined net pressure coefficient for a parapet
H	=	Height of hill or escarpment in Figure 6.2.4 in m.
H	=	Loads due to weight and pressure of soil, water in soil, or other materials, or related internal moments and forces (as used in Sec 2.7)
I	=	Importance factor
I_z	=	Intensity of turbulence from Eq. 6.2.7
K_1, K_2, K_3	=	Multipliers in Figure 6.2.4 to obtain K_{zt}

K_d = Wind directionality factor in Table 6.2.12

K_h = Velocity pressure exposure coefficient evaluated at height $z = h$

K_z = Velocity pressure exposure coefficient evaluated at height z

K_{zt} = Topographic factor as defined in Sec 2.4.7

L = Horizontal dimension of a building measured parallel to the wind direction, in m (as used in Sec 2.4)

L = Live loads due to intended use and occupancy, including loads due to movable objects and movable partitions and loads temporarily supported by the structure during maintenance, or related internal moments and forces, L includes any permissible reduction. If resistance to impact loads is taken into account in design, such effects shall be included with the live load L. (as used in Sec 2.7)

L_h = Distance upwind of crest of hill or escarpment in Figure 6.2.4 to where the difference in ground elevation is half the height of hill or escarpment, in m.

L_r = Roof live loads, or related internal moments and forces. (as used in Sec 2.7)

L_r = Horizontal dimension of return corner for a solid freestanding wall or solid sign from Figure 6.2.20, in m. (as used in Sec 2.4)

$L_{\check{z}}$ = Integral length scale of turbulence, in m.

Level-i = Floor level of the structure referred to by the subscript i, e.g., $i = 1$ designates the first level above the base.

Level- n = Uppermost level in the main portion of the structure.

M_s = Overturning moment at level-x

N_1 = Reduced frequency from Eq. 6.2.14

N_i = Standard Penetration Number of soil layer i

P_{net} = Net design wind pressure from Eq. 6.2.4, in N/m^2

P_{net30} = Net design wind pressure for Exposure A at h = 9.1 m and I = 1.0 from Figure 6.2.3, in N/m^2.

P_p = Combined net pressure on a parapet from Eq. 6.2.22, in N/m^2.

P_c = Net design wind pressure from Eq. 6.2.3, in N/m^2.

P_{c30} = Simplified design wind pressure for Exposure A at h = 9.1 m and I = 1.0 from Figure 6.2.2, in N/m^2.

P_s = Total vertical design load at level-x

P_w = Wind pressure acting on windward face in Figure 6.2.9, in N/m^2.

Q = Background response factor from Eq. 6.2.8

R = Resonant response factor from Eq. 6.2.12

R = Response reduction factor for structural systems. (as used in Sec 2.5)

R = Rain load, or related internal moments and forces. (as used in Sec 2.7)

R_B, R_h, R_L = Values from Eq. 6.2.15

R_i = Reduction factor from Eq. 6.2.18

R_n = Value from Eq. 6.2.13

S = Soil factor.

S_a = Design Spectral Acceleration (in units of g)

S_{ui} = Undrained shear strength of cohesive layer i

T = Fundamental period of vibration of structure, in seconds, of the structure in the direction under consideration. (as used in Sec 2.5)

T = Self-straining forces and cumulative effect of temperature, creep, shrinkage, differential settlement, and shrinkage-compensating concrete, or combinations thereof, or related internal moments and forces. (as used in Sec 2.7)

T_e = Effective fundamental period of the structure in the direction under consideration, as determined for nonlinear static analysis

V = Basic wind speed obtained from Figure 6.2.1 or Table 6.2.8, in m/s. The basic wind speed corresponds to a 3-s gust speed at 10 m above ground in Exposure Category B having an annual probability of occurrence of 0.02.

V = Total design base shear calculated by equivalent static analysis. (as used in Sec 2.5)

V_i = Unpartitioned internal volume m^3

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

V_1 = Total applied lateral force at the first increment of lateral load in nonlinear static analysis.

V_y = Effective yield strength determined from a bilinear curve fitted to the capacity curve

V_{rc} = Total design base shear calculated by response spectrum analysis

V_{th} = Total design base shear calculated by time history analysis

V_{ci} = Shear wave velocity of soil layer i

V_s = Design storey shear in storey x

W = Width of building in Figures 6.2.12, 6.2.14(a) and 6.2.14(b), and width of span in Figures 6.2.13 and 6.2.15 in m.

W = Total seismic weight of building. (as used in Sec 2.5)

W = Wind load, or related internal moments and forces. (as used in Sec 2.7)

X = Distance to center of pressure from windward edge in Figure 6.2.18, in m.

Z = Seismic zone coefficient.

a = Width of pressure coefficient zone, in m.

b = Mean hourly wind speed factor in Eq. 6.2.16 from Table 6.2.10

$\hat{b}$ = 3-s gust speed factor from Table 6.2.10

c = Turbulence intensity factor in Eq. 6.2.7 from Table 6.2.10

e_{ai} = Accidental eccentricity of floor mass at level-i

g = Acceleration due to gravity.

g_Q = Peak factor for background response in Equations 6.2.6 and 6.2.10

g_R = Peak factor for resonant response in Eq. 6.2.10

g_V = Peak factor for wind response in Equations 6.2.6 and 6.2.10

h = Mean roof height of a building or height of other structure, except that eave height shall be used for roof angle 8 of less than or equal to 10^o, in m.

h_e = Roof eave height at a particular wall, or the average height if the eave varies along the wall

h_i, h_n, h_s = Height in metres above the base to level i, -n or -x respectively

h_{cs} = Storey Height of storey x (below level- x)

l = Integral length scale factor from Table 6.2.10 in m.

n_1 = Building natural frequency, Hz

p = Design pressure to be used in determination of wind loads for buildings, in N/m^2

p_L = Wind pressure acting on leeward face in Figure 6.2.9, in N/m^2

q = Velocity pressure, in N/m^2.

q_h = Velocity pressure evaluated at height $z = h$, in N/m^2

q_i = Velocity pressure for internal pressure determination, in N/m^2.

q_p = Velocity pressure at top of parapet, in N/m^2.

q_z = Velocity pressure evaluated at height z above ground, in N/m^2.

r = Rise-to-span ratio for arched roofs.

s = Vertical dimension of the solid freestanding wall or solid sign from Figure 6.2.20, in m.

w_i, w_s = Portion of W which is assigned to level i and x respectively

x = Distance upwind or downwind of crest in Figure 6.2.4, in m.

z = Height above ground level, in m.

$\bar{z}$ = Equivalent height of structure, in m.

z_g = Nominal height of the atmospheric boundary layer used in this standard. Values appear in Table 6.2.10

z_{min} = Exposure constant from Table 6.2.10

Δ_a = Maximum allowable storey drift

Δ_s = Design storey drift of storey x

$\in$ = Ratio of solid area to gross area for solid freestanding wall, solid sign, open sign, face of a trussed tower, or lattice structure

$\bar{\in}$ = Integral length scale power law exponent in Eq. 6.2.9 from Table 6.2.10

Ω_o = Horizontal seismic overstrength factor from Table 6.2.19

α = 3-s gust-speed power law exponent from Table 6.2.10

$\hat{\alpha}$ = Reciprocal of α from Table 6.2.10

$\bar{\alpha}$ = Mean hourly wind-speed power law exponent in Eq. 6.2.16 from Table 6.2.10

þ = Damping ratio, percent critical for buildings or other structures

ð$_i$ = Horizontal displacement at level-i relative to the base due to applied lateral forces.

ð$_j$ = The displacement of the control point at load increment j.

ð$_T$ = The target displacement of the control point.

ð$_1$ = The displacement of the control point at the first increment of lateral load.

ð$_y$ = The effective yield displacement of the control point determined from a bilinear curve fitted to the capacity curve

5 = Value used in Eq. 6.2.15 (see Sec 2.4.8.2)

5 = Damping correction factor

8 = Angle of plane of roof from horizontal, in degrees. (as used in Sec 2.4)

8 = Stability coefficient to assess P-delta effects. (as used in Sec 2.5)

h = Adjustment factor for building height and exposure from Figures 6.2.2 and 6.2.3

u = Height-to-width ratio for solid sign

£ = Viscous damping ratio of the structure

\$$_{ik}$ = Modal shape coefficient at level i for mode k

2.2 DEAD LOADS

2.2.1 General

The minimum design dead load for buildings and portions thereof shall be determined in accordance with the provisions of this Section. In addition, design of the overall structure and its primary load-resisting systems shall conform to the general design provisions given in Chapter 1.

2.2.2 Definition

Dead Load is the vertical load due to the weight of permanent structural and non-structural components and attachments of a building such as walls, floors, ceilings, permanent partitions and fixed service equipment etc.

2.2.3 Assessment of Dead Load

Dead load for a structural member shall be assessed based on the forces due to:

- weight of the member itself,
- weight of all materials of construction incorporated into the building to be supported permanently by the member,
- weight of permanent partitions,
- weight of fixed service equipment, and
- net effect of prestressing.

2.2.4 Weight of Materials and Constructions

In estimating dead loads, the actual weights of materials and constructions shall be used, provided that in the absence of definite information, the weights given in Tables 6.2.1 and 6.2.2 shall be assumed for the purposes of design.

Table 6.2.1: Unit Weight of Basic Materials

Material	Unit Weight (kN/m^3)	Material	Unit Weight (kN/m^3)
Aluminium	27.0	Granite, Basalt	26.4
Asphalt	21.2	Iron - cast	70.7
Brass	83.6	- wrought	75.4
Bronze	87.7	Lead	111.0
Brick	18.9	Limestone	24.5
Cement	14.7	Marble	26.4
Coal, loose	8.8	Sand, dry	15.7
Concrete - stone aggregate (unreinforced)	22.8*	Sandstone	22.6
- brick aggregate (unreinforced)	20.4*	Slate	28.3
Copper	86.4	Steel	77.0
Cork, normal	1.7	Stainless Steel	78.75
Cork, compressed	3.7	Timber	5.9-11.0
Glass, window (soda-lime)	25.5	Zinc	70.0

* for reinforced concrete, add 0.63 kN/m^3 for each 1% by volume of main reinforcement

Table 6.2.2: Weight of Construction Materials.

Material/Component/Member	Weight per Unit Area (kN/m^2)
Floor	
Asphalt, 25 mm thick	0.526
Clay tiling, 13 mm thick	0.268
Concrete slab (stone aggregate)*:	
solid, 100 mm thick	2.360
solid, 150 mm thick	3.540
Galvanized steel floor deck (excl. topping)	0.147-0.383
Magnesium oxychloride:	
normal (sawdust filler), 25 mm thick	0.345
heavy duty (mineral filler), 25 mm thick	0.527
Terrazzo paving 16 mm thick	0.431
Roof	
Acrylic resin sheet, corrugated:	
3 mm thick, standard corrugations	0.043
3 mm thick, deep corrugations	0.062
Aluminium, corrugated sheeting:	
(incl. lap and fastenings)	
1.2 mm thick	0.048
0.8 mm thick	0.028
0.6 mm thick	0.024
Aluminium sheet(plain):	
1.2 mm thick	0.033
1.0 mm thick	0.024
0.8 mm thick	0.019
Bituminous felt (5 ply) and gravel	0.431
Slates:	
4.7 mm thick	0.335
9.5 mm thick	0.671
Steel sheet, flat galvanized:	
1.00 mm thick	0.082
0.80 mm thick	0.067
0.60 mm thick	0.053
Steel, galvanized std. corrugated sheeting:	
(incl. lap and fastenings)	
1.0 mm thick	0.120
0.8 mm thick	0.096
0.6 mm thick	0.077
Tiles :	
terra-cotta tiles (French pattern)	0.575
concrete , 25 mm thick	0.527
clay tiles	0.6-0.9
Walls and Partitions	
Acrylic resin sheet, flat, per mm thickness	0.012
Asbestos cement sheeting:	
4.5 mm thick	0.072
6.0 mm thick	0.106
Brick masonry work, excl. plaster:	
burnt clay, per 100 mm thickness	1.910
sand-lime, per 100 mm thickness	1.980
Concrete (stone aggregate)*:	
100 mm thick	2.360
150 mm thick	3.540
250 mm thick	5.900
Fibre insulation board, per 10 mm thickness	0.034
Fibrous plaster board, per 10 mm thickness	0.092
Glass, per 10 mm thickness	0.269
Hardboard, per 10 mm thickness	0.961
Particle or flake board, per 10 mm thickness	0.075
Plaster board, per 10 mm thickness	0.092
Plywood, per 10 mm thickness	0.061
Ceiling	
Fibrous plaster, 10 mm thick	0.081
Cement plaster, 13 mm thick	0.287
Suspended metal lath and plaster (two faced incl. studding)	0.480
Miscellaneous	
Felt (insulating), per 10 mm thickness	0.019
Plaster:	
Cement plaster, per 10 mm thickness	0.230
Lime plaster, per 10 mm thickness	0.191
PVC sheet, per 10 mm thickness	0.153
Rubber paving, per 10 mm thickness	0.151
Terra-cotta Hollow Block Masonry:	
75 mm thick	0.671
100 mm thick	0.995
150 mm thick	1.388

* For brick aggregate, 90% of the listed values may be used.

2.2.5 Weight of Permanent Partitions

When partition walls are indicated on the plans, their weight shall be considered as dead load acting as concentrated line loads in their actual positions on the floor. The loads due to anticipated partition walls, which are not indicated on the plans, shall be treated as live loads and determined in accordance with Sec 2.3.6.

2.2.6 Weight of Fixed Service Equipment

Weights of fixed service equipment and other permanent machinery, such as electrical feeders and other machinery, heating, ventilating and air-conditioning systems, lifts and escalators, plumbing stacks and risers etc. shall be included as dead load whenever such equipment are supported by structural members.

2.2.7 Additional Loads

In evaluating the final dead loads on a structural member for design purposes, allowances shall be made for additional loads resulting from the (i) difference between the prescribed and the actual weights of the members and construction materials; (ii) inclusion of future installations; (iii) changes in occupancy or use of buildings; and (iv) inclusion of structural and non-structural members not covered in Sections 2.2.2 and 2.2.3.

2.3 LIVE LOADS

2.3.1 General

The live loads used for the structural design of floors, roof and the supporting members shall be the greatest applied loads arising from the intended use or occupancy of the building, or from the stacking of materials and the use of equipment and propping during construction, but shall not be less than the minimum design live loads set out by the provisions of this Section. For the design of structural members for forces including live loads, requirements of the relevant Sections of Chapter 1 shall also be fulfilled.

2.3.2 Definition

Live load is the load superimposed by the use or occupancy of the building not including the environmental loads such as wind load, rain load, earthquake load or dead load.

2.3.3 Minimum Floor Live Loads

The minimum floor live loads shall be the greatest actual imposed loads resulting from the intended use or occupancy of the floor, and shall not be less than the uniformly distributed load patterns specified in Sec 2.3.4 or the concentrated loads specified in Sec 2.3.5 whichever produces the most critical effect. The live loads shall be assumed to act vertically upon the area projected on a horizontal plane.

2.3.4 Uniformly Distributed Loads

The uniformly distributed live load shall not be less than the values listed in Table 6.2.3, reduced as may be specified in Sec 2.3.13, applied uniformly over the entire area of the floor, or any portion thereof to produce the most adverse effects in the member concerned.

2.3.5 Concentrated Loads

The concentrated load to be applied non-concurrently with the uniformly distributed load given in Sec 2.3.4, shall not be less than that listed in Table 6.2.3. Unless otherwise specified in Table 6.2.3 or in the following paragraph, the concentrated load shall be applied over an area of 300 mm x 300 mm and shall be located so as to produce the maximum stress conditions in the structural members.

In areas where vehicles are used or stored, such as car parking garages, ramps, repair shops etc., provision shall be made for concentrated loads consisting of two or more loads spaced nominally 1.5 m on centres in absence of the uniform live loads. Each load shall be 40 percent of the gross weight of the maximum size vehicle to be accommodated and applied over an area of 750 mm x 750 mm. For the storage of private or pleasure-type vehicles without repair or fuelling, floors shall be investigated in the absence of the uniform live load, for a minimum concentrated wheel load of 9 kN spaced 1.5 m on centres, applied over an area of 750 mm x 750 mm. The uniform live loads for these cases are provided in Table 6.2.3. The condition of concentrated or uniform live load producing the greater stresses shall govern.

Table 6.2.3: Minimum Uniformly Distributed and Concentrated Live Loads[a]

Occupancy or Use	Uniform kN/m²	Concentrated kN
Apartments (see Residential)		
Access floor systems		
Office use	2.40	9.0
Computer use	4.80	9.0
Armories and drill rooms	7.20	--
Assembly areas and theaters		
Fixed seats (fastened to floor)	2.90	--
Lobbies	4.80	--
Movable seats	4.80	--
Platforms (assembly)	4.80	--
Stage floors	7.20	--
Balconies (exterior)	4.80	--
On one- and two-family residences only, and not exceeding 19.3 m^2	2.90	--
Bowling alleys, poolrooms, and similar recreational areas	3.60	--
Catwalks for maintenance access	2.00	1.33
Corridors		
First floor	4.80	--
Other floors, same as occupancy served except as indicated		
Dance halls and ballrooms	4.80	--
Decks (patio and roof)	Same as area served, or for the type of occupancy accommodated	
Dining rooms and restaurants	4.80	--
Dwellings (see *Residential*)	--	
Elevator machine room grating (on area of 2,580 mm^2)	--	1.33
Finish light floor plate construction (on area of 645 mm^2)	--	0.90
Fire escapes	4.80	--
On single-family dwellings only	2.00	--
Fixed ladders	See Sec 2.3.11	
Garages (passenger vehicles only), Trucks and buses	2.0[b,c]	
Grandstands	See *Stadiums and arenas, Bleachers*	
Gymnasiums—main floors and balconies	4.80	--
Handrails, guardrails, and grab bars	See Sec 2.3.11	
Hospitals		
Operating rooms, laboratories	2.90	4.50
Patient rooms	2.00	4.50
Corridors above first floor	3.80	4.50
Hotels	See *Residential*	
Libraries		
Reading rooms	2.90	4.50
Stack rooms	7.20 [d]	4.50
Corridors above first floor	3.80	4.50

Table 6.2.3: Minimum Uniformly Distributed and Concentrated Live Loads[a]

Occupancy or Use	Uniform kN/m²	Concentrated kN
Manufacturing*		
Light	4.00	6.00
Medium	6.00	9.00
Heavy	12.00	13.40
Garments manufacturing floor except stacking or storage area	4.00[e]	--
Stacking or storage area of garments manufacturing industry	6.00 [f]	10.00 [f]
Marquees	3.60	--
Office Buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first-floor corridors	4.80	9.00
Offices	2.40	9.00
Corridors above first floor	3.80	9.00
Penal Institutions		
Cell blocks	2.00	--
Corridors	4.80	--
Residential		
Dwellings (one- and two-family)		
Uninhabitable attics without storage	0.50	--
Uninhabitable attics with storage	1.00	--
Habitable attics and sleeping areas	1.50	--
All other areas except stairs and balconies	2.00	--
Hotels and multifamily houses		
Private rooms and corridors serving them	2.00	--
Public rooms and corridors serving them	4.80	--
Reviewing stands, grandstands, and bleachers	4.80 [g]	--
Roofs		
Ordinary flat roof	1.00 [h]	--
Pitched and curved roofs	See Table 6.2.4	
Roofs used for promenade purposes	2.90	--
Roofs used for roof gardens or assembly purposes	4.80	--
Roofs used for other special purposes	See Note [i] below	
Awnings and canopies		
Fabric construction supported by a lightweight rigid skeleton structure	0.24 (nonreduceable)	--
All other construction	1.00	--
Primary roof members, exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages	--	9.00
All other occupancies	--	1.33
All roof surfaces subject to maintenance workers	--	1.33
Schools		
Classrooms	2.00	4.50
Corridors above first floor	3.80	4.50
First-floor corridors	4.80	4.50
Scuttles, skylight ribs, and accessible ceilings		0.90
Sidewalks, vehicular driveways, and yards subject to trucking	12.00 [j]	35.60 [k]

Table 6.2.3: Minimum Uniformly Distributed and Concentrated Live Loads[a]

Occupancy or Use	Uniform kN/m²	Concentrated kN
Stadiums and arenas		
Bleachers	4.80 [g]	--
Fixed seats (fastened to floor)	2.90 [g]	--
Stairs and exit ways	4.80	See Note [l] below
One- and two-family residences only	2.00	--
Storage areas above ceilings	1.00	--
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	6.00	--
Heavy	12.00	--
Stores		
Retail		
First floor	4.80	4.50
Upper floors	3.60	4.50
Wholesale, all floors	6.00	4.50
Vehicle barriers	See Sec 2.3.11	
Walkways and elevated platforms (other than exit ways)	2.90	--
Yards and terraces, pedestrian	4.80	--

Notes:

a It must be ensured that the average weight of equipment, machinery, raw materials and products that may occupy the floor is less than the specified value in the Table. In case the weight exceeds the specified values in the Table, actual maximum probable weight acting in the actual manner shall be used in the analysis and design.

b Floors in garages or portions of a building used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 6.2.3 or the following concentrated load: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 13.35 kN acting on an area of 114 mm by 114 mm footprint of a jack; and (2) for mechanical parking structures without slab or deck that are used for storing passenger car only, 10 kN per wheel.

c Garages accommodating trucks and buses shall be designed in accordance with an approved method, which contains provisions for truck and bus loadings.

d The loading applies to stack room floors that support non-mobile, double-faced library book stacks subject to the following limitations: (1) The nominal book stack unit height shall not exceed 2290 mm; (2) the nominal shelf depth shall not exceed 300 mm for each face; (3) parallel rows of double-faced book stacks shall be separated by aisles not less than 900 mm wide.

e Subject to the provisions of reduction of live load as per Sec 2.3.13

f Uniformly distributed and concentrated load provisions are applicable for a maximum floor height of 3.5 m. In case of higher floor height, the load(s) must be proportionally increased.

g In addition to the vertical live loads, the design shall include horizontal swaying forces applied to each row of the seats as follows: 0.350 kN per linear meter of seat applied in a direction parallel to each row of seats and 0.15 kN per linear meter of seat applied in a direction perpendicular to each row of seats. The parallel and perpendicular horizontal swaying forces need not be applied simultaneously.

h Where uniform roof live loads are reduced to less than 1.0 kN/m² in accordance with Sec 2.3.14.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the greatest unfavorable effect.

i Roofs used for other special purposes shall be designed for appropriate loads as approved by the authority having jurisdiction.

j Other uniform loads in accordance with an approved method, which contains provisions for truck loadings, shall also be considered where appropriate.

k The concentrated wheel load shall be applied on an area of 114 mm by 114 mm footprint of a jack.

l Minimum concentrated load on stair treads (on area of 2,580 mm²) is 1.33 kN.

* The loading in industrial buildings varies considerably and so the loadings under the terms 'light,' 'medium' and 'heavy' are introduced in order to allow for which the relevant floor is designed. It is however important to assess the actual loads to ensure that they are not in excess of the stipulated load, in case where they are in excess, the design shall be based on the actual loadings.

2.3.6 Provision for Partition Walls

When partitions, not indicated on the plans, are anticipated to be placed on the floors, their weight shall be included as an additional live load acting as concentrated line loads in an arrangement producing the most severe effect on the floor, unless it can be shown that a more favourable arrangement of the partitions shall prevail during the future use of the floor.

In the case of light partitions, wherein the total weight per metre run is not greater than 5.5 kN, a uniformly distributed live load may be applied on the floor in lieu of the concentrated line loads specified above. Such uniform live load per square metre shall be at least 33% of the weight per metre run of the partitions, subject to a minimum of 1.2 kN/m^2.

2.3.7 More than One Occupancy

Where an area of a floor is intended for two or more occupancies at different times, the value to be used from Table 6.2.3 shall be the greatest value for any of the occupancies concerned.

2.3.8 Minimum Roof Live Loads

Roof live loads shall be assumed to act vertically over the area projected by the roof or any portion of it upon a horizontal plane, and shall be determined as specified in Table 6.2.4.

Table 6.2.4: Minimum Roof Live Loads(1)

	Type and Slope of Roof	Distributed Load, kN/m^2	Concentrated Load, kN
I	Flat roof (slope = 0)	See Table 6.2.3	
II	(A) Pitched or sloped roof (0 < slope < 1/3) (B) Arched roof or dome (rise < 1/8 span)	1.0	0.9
III	(A) Pitched or sloped roof (1/3 ≤ slope < 1.0) (B) Arched roof or dome (1/8 ≤ rise < 3/8 span)	0.8	0.9
IV	(A) Pitched or sloped roof (slope ≥ 1.0) (B) Arched roof or dome (rise ≥ 3/8 span)	0.6	0.9
V	Greenhouse, and agriculture buildings	0.5	0.9
VI	Canopies and awnings, except those with cloth covers	Same as given in I to IV above based on the type and slope.	

Note: (1) Greater of this load and rain load as specified in Sec 2.6.2 shall be taken as the design live load for roof. The distributed load shall be applied over the area of the roof projected upon a horizontal plane and shall not be applied simultaneously with the concentrated load. The concentrated load shall be assumed to act upon a 300 mm x 300 mm area and need not be considered for roofs capable of laterally distributing the load, e.g. reinforced concrete slabs.

2.3.9 Loads not Specified

Live loads, not specified for uses or occupancies in Sections 2.3.3, 2.3.4 and 2.3.5, shall be determined from loads resulting from:

(a) weight of the probable assembly of persons;

(b) weight of the probable accumulation of equipment and furniture, and

(c) weight of the probable storage of materials.

2.3.10 Partial Loading and Other Loading Arrangements

The full intensity of the appropriately reduced live load applied only to a portion of the length or area of a structure or member shall be considered, if it produces a more unfavourable effect than the same intensity applied over the full length or area of the structure or member.

Where uniformly distributed live loads are used in the design of continuous members and their supports, consideration shall be given to full dead load on all spans in combination with full live loads on adjacent spans and on alternate spans whichever produces a more unfavourable effect.

2.3.11 Other Live Loads

Live loads on miscellaneous structures and components, such as handrails and supporting members, parapets and balustrades, ceilings, skylights and supports, and the like, shall be determined from the analysis of the actual loads on them, but shall not be less than those given in Table 6.2.5.

2.3.12 Impact and Dynamic Loads

The live loads specified in Sec 2.3.3 shall be assumed to include allowances for impacts arising from normal uses only. However, forces imposed by unusual vibrations and impacts resulting from the operation of installed machinery and equipment shall be determined separately and treated as additional live loads. Live loads due to vibration or impact shall be determined by dynamic analysis of the supporting member or structure including foundations, or from the recommended values supplied by the manufacture of the particular equipment or machinery. In absence of definite information, values listed in Table 6.2.6 for some common equipment, shall be used for design purposes.

Table 6.2.5: Miscellaneous Live Loads

Structural Member or Component	Live Load(1) (kN/m)
A. Handrails, parapets and supports:	
(a) Light access stairs, gangways etc.	
(i) width ≤ 0.6 m	0.25
(ii) width > 0.6 m	0.35
(b) Staircases other than in (a) above, ramps, balconies:	
(i) Single dwelling and private	0.35
(ii) Staircases in residential buildings	0.35
(iii) Balconies or portion thereof, stands etc. having fixed seats within 0.55 m of the barrier	1.5
(iv) Public assembly buildings including theatres, cinemas, assembly halls, stadiums, mosques, churches, schools etc.	3.0
(v) Buildings and occupancies other than (i) to (iv) above	0.75
B. Vehicle barriers for car parks and ramps:	
(a) For vehicles having gross mass ≤ 2500 kg	100(2)
(b) For vehicles having gross mass > 2500 kg	165(2)
(c) For ramps of car parks etc.	see note(3)

Notes: (1) These loads shall be applied non-concurrently along horizontal and vertical directions, except as specified in note (2) below.
(2) These loads shall be applied only in the horizontal direction, uniformly distributed over any length of 1.5 m of a barrier and shall be considered to act at bumper height. For case 2(a) bumper height may be taken as 375 mm above floor level.
(3) Barriers to access ramps of car parks shall be designed for horizontal forces equal to 50% of those given in 2(a) and 2(b) applied at a level of 610 mm above the ramp. Barriers to straight exit ramps exceeding 20 m in length shall be designed for horizontal forces equal to twice the values given in 2(a) and 2(b).

Table 6.2.6: Minimum Live Loads on Supports and Connections of Equipment due to Impact(1)

Equipment or Machinery	Additional load due to impact as percentage of static load including self-weight: Vertical	Horizontal
1. Lifts, hoists and related operating machinery	100%	Not applicable
2. Light machinery (shaft or motor driven)	20%	Not applicable
3. Reciprocating machinery, or power driven units.	50%	Not applicable
4. Hangers supporting floors and balconies	33%	Not applicable
5. Cranes :		
(a) Electric overhead cranes	25% of maximum wheel load	(i) Transverse to the rail : 20% of the weight of trolley and lifted load only, applied one-half at the top of each rail (ii) Along the rail : 10% of maximum wheel load applied at the top of each rail
(b) Manually operated cranes	50% of the values in (a) above	50% of the values in (a) above
(c) Cab-operated travelling cranes	25%	Not applicable

(1) All these loads shall be increased if so recommended by the manufacturer. For machinery and equipment not listed, impact loads shall be those recommended by the manufacturers, or determined by dynamic analysis.

2.3.13 Reduction of Live Loads

Except for roof uniform live loads, all other minimum uniformly distributed live loads, L_o in Table 6.2.3, may be reduced according to the following provisions.

2.3.13.1 General

Subject to the limitations of Sections 2.3.13.2 to 2.3.13.5, members for which a value of $K_{LL}A_T$ is 37.16 m^2 or more are permitted to be designed for a reduced live load in accordance with the following formula:

$$L = L_0\left(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}\right) \qquad (6.2.1)$$

Where, L = reduced design live load per m^2 of area supported by the member; L_0= unreduced design live load per m^2 of area supported by the member (Table 6.2.3); K_{LL}= live load element factor (Table 6.2.7); A_T= tributary area in m^2. L shall not be less than $0.50L_0$ for members supporting one floor and L shall not be less than $0.40L_0$ for members supporting two or more floors.

Table 6.2.7: Live Load Element Factor, K_{LL}

Element	K_{LL}*
Interior columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2
All other members not identified including: • Edge beams with cantilever slabs • Cantilever beams • One-way slabs • Two-way slabs • Members without provisions for continuous shear transfer normal to their span	1

* In lieu of the preceding values, K_{LL} is permitted to be calculated.

2.3.13.2 Heavy live loads

Live loads that exceed 4.80 kN/m^2 shall not be reduced.

Exception: Live loads for members supporting two or more floors may be reduced by 20 percent.

2.3.13.3 Passenger car garages

The live loads shall not be reduced in passenger car garages.

Exception: Live loads for members supporting two or more floors may be reduced by 20 percent.

2.3.13.4 Special occupancies

(a) Live loads of 4.80 kN/m^2 or less shall not be reduced in public assembly occupancies.

(b) There shall be no reduction of live loads for cyclone shelters.

2.3.13.5 Limitations on one-way slabs

The tributary area, A_T, for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span.

2.3.14 Reduction in Roof Live Loads

The minimum uniformly distributed roof live loads, L_o in Table 6.2.3, are permitted to be reduced according to the following provisions.

2.3.14.1 Flat, pitched, and curved roofs.

Ordinary flat, pitched, and curved roofs are permitted to be designed for a reduced roof live load, as specified in Eq. 6.2.2 or other controlling combinations of loads, as discussed later in this Chapter, whichever produces the greater load. In structures such as greenhouses, where special scaffolding is used as a work surface for workmen and materials during maintenance and repair operations, a lower roof load than specified in Eq. 6.2.2 shall not be used unless approved by the authority having jurisdiction. On such structures, the minimum roof live load shall be 0.60kN/m^2.

$$L_r = L_o R_1 R_2 \qquad (0.60 \le L_r \le 1.00) \qquad (6.2.2)$$

Where,

L_r = reduced roof live load per m^2 of horizontal projection in kN/m^2

The reduction factors R_1 and R_2 shall be determined as follows:

R_1 = 1 for $A_t \le 18.58$ m^2

= 1.2 − 0.011A_t for 18.58 m^2 < A_t < 55.74 m^2

= 0.6 for $A_t \ge 55.74$ m^2

A_t = tributary area in m^2 supported by any structural member and

R_2 = 1 for $F \le 4$

= 1.2 − 0.05F for 4 < F < 12

= 0.6 for $F \ge 12$

For a pitched roof, F = 0.12 × slope, with slope expressed in percentage points and, for an arch or dome, F = rise-to-span ratio multiplied by 32.

2.3.14.2 Special purpose roofs.

Roofs that have an occupancy function, such as roof gardens, assembly purposes, or other special purposes are permitted to have their uniformly distributed live load reduced in accordance with the requirements of Sec 2.3.13.

2.5 EARTHQUAKE LOADS

2.5.1 General

Minimum design earthquake forces for buildings, structures or components thereof shall be determined in accordance with the provisions of Sec 2.5. Some definitions and symbols relevant for earthquake resistant design for buildings are provided in Sections 2.1.3 and 2.1.4. Section 2.5.2 presents basic earthquake resistant design concepts. Section 2.5.3 describes procedures for soil investigations, while Sec 2.5.4 describes procedures for determining earthquake ground motion for design. Section 2.5.5 describes different types of buildings and structural systems which possess different earthquake resistant characteristics. Static analysis procedures for design are described in Sections 2.5.6, 2.5.7 and 2.5.12. Dynamic analysis procedures are dealt with in Sections 2.5.8 to 2.5.11. Section 2.5.13 presents how seismic effects are accounted in the design and combination of earthquake loading effects in different directions and with other loading effects. Section 2.5.14 deals with allowable drift and deformation limits. Section 2.5.15 addresses design of non-structural components in buildings. Section 2.5.16 presents design considerations for buildings with seismic isolation systems. Design for soft storey condition in buildings is addressed in Sec 2.5.17.

2.5.2 Earthquake Resistant Design – Basic Concepts

2.5.2.1 General principles

The purpose of earthquake resistant design provisions in this Code is to provide guidelines for the design and construction of new structures subject to earthquake ground motions in order to minimize the risk to life for all structures, to increase the expected performance of higher occupancy structures as compared to ordinary structures, and to improve the capability of essential structures to function after an earthquake. It is not economically feasible to design and construct buildings without any damage for a major earthquake event. The intent is therefore to allow inelastic deformation and structural damage at preferred locations in the structure without endangering structural integrity and to prevent structural collapse during a major earthquake.

The seismic zoning map (Fig. 6.2.24) divides the country into four seismic zones with different expected levels of intensity of ground motion. Each seismic zone has a zone coefficient which provides expected peak ground acceleration values on rock/firm soil corresponding to the maximum considered earthquake (MCE). The design basis earthquake is taken as 2/3 of the maximum considered earthquake.

The effects of the earthquake ground motion on the structure is expressed in terms of an idealized elastic design acceleration response spectrum, which depends on (a) seismic zone coefficient and local soil conditions defining ground motion and (b) importance factor and response reduction factor representing building considerations. The earthquake forces acting on the structure is reduced using the response modification/reduction factor R in order to take advantage of the inelastic energy dissipation due to inherent ductility and redundancy in the structure as well as material over-strength. The importance factor I increases design forces for important structures. The provisions of this Code for ductility and detailing need to be satisfied even for structures and members for which load combinations that do not contain the earthquake effect indicate larger demands than combinations including earthquake. The elastic deformations calculated under these reduced design forces are multiplied by the deflection amplification factor, C_d to estimate the deformations likely to result from the design earthquake.

The seismic design guidelines presented in this Section are based on the assumption that the soil supporting the structure will not liquefy, settle or slide due to loss of strength during the earthquake. Reinforced and prestressed concrete members shall be suitably designed to ensure that premature failure due to shear or bond does not occur. Ductile detailing of reinforced concrete members is of prime importance. In steel structures, members and their connections should be so proportioned that high ductility is obtained, avoiding premature failure due to elastic or inelastic buckling of any type.

The building structure shall include complete lateral and vertical force-resisting systems capable of providing adequate strength, stiffness, and energy dissipation capacity to withstand the design ground motions within the prescribed limits of deformation and strength demand. The design ground motions shall be assumed to occur along any horizontal direction of a building structure. The adequacy of the structural systems shall be demonstrated through the construction of a mathematical model and evaluation of this model for the effects of design ground motions.

2.5.2.2 Characteristics of Earthquake Resistant Buildings

The desirable characteristics of earthquake resistant buildings are described below:

Structural Simplicity, Uniformity and Symmetry:

Structural simplicity, uniformity and plan symmetry is characterized by an even distribution of mass and structural elements which allows short and direct transmission of the inertia forces created in the distributed masses of the building to its foundation. A building configuration with symmetrical layout of structural elements of the lateral force resisting system, and well-distributed in-plan, is desirable. Uniformity along the height of the building is also important, since it tends to eliminate the occurrence of sensitive zones where concentrations of stress or large ductility demands might cause premature collapse.

Some basic guidelines are given below:

(i) With respect to the lateral stiffness and mass distribution, the building structure shall be approximately symmetrical in plan with respect to two orthogonal axes.
(ii) Both the lateral stiffness and the mass of the individual storeys shall remain constant or reduce gradually, without abrupt changes, from the base to the top of a particular building.
(iii) All structural elements of the lateral load resisting systems, such as cores, structural walls, or frames shall run without interruption from the foundations to the top of the building.
(iv) An irregular building may be subdivided into dynamically independent regular units well separated against pounding of the individual units to achieve uniformity.
(v) The length to breadth ratio ($\mathrm{h} = \mathrm{L}_{\mathrm{mas}}/\mathrm{L}_{\mathrm{min}}$) of the building in plan shall not be higher than 4, where $\mathrm{L}_{\mathrm{mas}}$ and $\mathrm{L}_{\mathrm{min}}$ are respectively the larger and smaller in plan dimension of the building, measured in orthogonal directions.

Structural Redundancy:

A high degree of redundancy accompanied by redistribution capacity through ductility is desirable, enabling a more widely spread energy dissipation across the entire structure and an increased total dissipated energy. The use of evenly distributed structural elements increases redundancy. Structural systems of higher static indeterminacy may result in higher response reduction factor *R*.

Horizontal Bi-directional Resistance and Stiffness:

Horizontal earthquake motion is a bi-directional phenomenon and thus the building structure needs to resist horizontal action in any direction. The structural elements of lateral force resisting system should be arranged in an orthogonal (in plan) pattern, ensuring similar resistance and stiffness characteristics in both main directions. The stiffness characteristics of the structure should also limit the development of excessive displacements that might lead to either instabilities due to second order effects or excessive damages.

Torsional Resistance and Stiffness

Besides lateral resistance and stiffness, building structures should possess adequate torsional resistance and stiffness in order to limit the development of torsional motions which tend to stress the different structural elements in a non-uniform way. In this respect, arrangements in which the main elements resisting the seismic action are distributed close to the periphery of the building present clear advantages.

Diaphragm Behaviour

In buildings, floors (including the roof) act as horizontal diaphragms that collect and transmit the inertia forces to the vertical structural systems and ensure that those systems act together in resisting the horizontal seismic action. Floor systems and the roof should be provided with in-plane stiffness and resistance and with effective connection to the vertical structural systems. Particular care should be taken in cases of non-compact or very elongated in-plan shapes and in cases of large floor openings, especially if the latter are located in the vicinity of the main vertical structural elements, thus hindering such effective connection between the vertical and horizontal structure. The in-plane stiffness of the floors shall be sufficiently large in comparison with the lateral stiffness of the vertical structural elements, so that the deformation of the floor shall have a small effect on the distribution of the forces among the vertical structural elements.

Foundation

The design and construction of the foundation and of its connection to the superstructure shall ensure that the whole building is subjected to a uniform seismic excitation. For buildings with individual foundation elements (footings or piles), the use of a foundation slab or tie-beams between these elements in both main directions is recommended, as described in Chapter 3.

2.5.3 Investigation and Assessment of Site Conditions

2.5.3.1 Site investigation

Appropriate site investigations should be carried out to identify the ground conditions influencing the seismic action.

The ground conditions at the building site should normally be free from risks of ground rupture, slope instability and permanent settlements caused by liquefaction or densification during an earthquake. The possibility of such phenomena should be investigated in accordance with standard procedures described in Chapter 3 of this Part.

The intent of the site investigation is to classify the Site into one of types SA, SB, SC, SD, SE, S_1 and S_2 as defined in Sec 2.5.3.2. Such classification is based on site profile and evaluated soil properties (shear wave velocity, Standard Penetration Resistance, undrained shear strength, soil type). The site class is used to determine the effect of local soil conditions on the earthquake ground motion.

For sites representing special soil type S_1 or S_2, site specific special studies for the ground motion should be done. Soil type S_1, having very low shear wave velocity and low material damping, can produce anomalous seismic site amplification and soil-structure interaction effects. For S_2 soils, possibility of soil failure should be studied.

For a structure belonging to Seismic Design Category C or D (Sec 2.5.5.2), site investigation should also include determination of soil parameters for the assessment of the following:

(a) Slope instability.
(b) Potential for Liquefaction and loss of soil strength.
(c) Differential settlement.
(d) Surface displacement due to faulting or lateral spreading.
(e) Lateral pressures on basement walls and retaining walls due to earthquake ground motion.

Liquefaction potential and possible consequences should be evaluated for design earthquake ground motions consistent with peak ground accelerations. Any Settlement due to densification of loose granular soils under design earthquake motion should be studied. The occurrence and consequences of geologic hazards such as slope instability or surface faulting should also be considered. The dynamic lateral earth pressure on basement walls and retaining walls during earthquake ground shaking is to be considered as an earthquake load for use in design load combinations

2.5.3.2 Site classification

Site will be classified as type SA, SB, SC, SD, SE, S_1 and S_2 based on the provisions of this Section. Classification will be done in accordance with Table 6.2.13 based on the soil properties of upper 30 meters of the site profile. Average soil properties will be determined as given in the following equations:

$$\bar{V}_s = \sum_{i=1}^{n} d_i / \sum_{i=1}^{n} \frac{d_i}{V_{si}} \qquad (6.2.31)$$

$$\bar{N} = \sum_{i=1}^{n} d_i / \sum_{i=1}^{n} \frac{d_i}{N_i} \qquad (6.2.32)$$

$$\bar{S}_u = \sum_{i=1}^{k} d_{ci} / \sum_{i=1}^{k} \frac{d_{ci}}{S_{ui}} \qquad (6.2.33)$$

Where,

n= Number of soil layers in upper 30 m

d_i = Thickness of layer i

V_{ci} = Shear wave velocity of layer i

N_i = Field (uncorrected) Standard Penetration Value for layer i

k = Number of cohesive soil layers in upper 30 m

d_{ci} = Thickness of cohesive layer i

s_{ui} = Undrained shear strength of cohesive layer i

The site profile up to a depth of 30 m is divided into n number of distinct soil or rock layers. Where some of the layers are cohesive, k is the number of cohesive layers. Hence $\sum_{i=1}^{n} d_i$ = 30 m, while $\sum^{k}{}_{i=1} d_{ci}$ < 30 m if $k€\ n$ in other words if there are both cohesionless and cohesive layers. The standard penetration value N as directly measured in the field without correction will be used.

The site classification should be done using average shear wave velocity $\bar{V}_c$ if this can be estimated, otherwise the value of $\bar{N}$ may be used.

Table 6.2.13: Site Classification Based on Soil Properties

Site Class	Description of soil profile up to 30 meters depth	Average Soil Properties in top 30 meters		
		Shear wave velocity, $\bar{V}_s$ (m/s)	SPT Value, $\bar{N}$ (blows/30cm)	Undrained shear strength, $\bar{S}_u$ (kPa)
SA	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	--	--
SB	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
SC	Deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
SD	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
SE	A soil profile consisting of a surface alluvium layer with V_s values of type SC or SD and thickness varying between about 5 m and 20 m, underlain by stiffer material with V_s > 800 m/s.	--	--	--
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	--	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types SA to SE or S_1	--	--	--

2.5.4 Earthquake Ground Motion

2.5.4.1 Regional seismicity

Bangladesh can be affected by moderate to strong earthquake events due to its proximity to the collision boundary of the Northeast moving Indian plate and Eurasian Plate. Strong historical earthquakes with magnitude greater than 7.0 have affected parts of Bangladesh in the last 150 years, some of them had their epicenters within the country. A brief description of the local geology, tectonic features and earthquake occurrence in the region is given in Appendix B.

2.5.4.2 Seismic zoning

The intent of the seismic zoning map is to give an indication of the Maximum Considered Earthquake (MCE) motion at different parts of the country. In probabilistic terms, the MCE motion may be considered to correspond to having a 2% probability of exceedance within a period of 50 years. The country has been divided into four seismic zones with different levels of ground motion. Table 6.2.14 includes a description of the four seismic zones. Figure 6.2.24 presents a map of Bangladesh showing the boundaries of the four zones. Each zone has a seismic zone coefficient (Z) which represents the maximum considered peak ground acceleration (PGA) on very stiff soil/rock (site class SA) in units of g (acceleration due to gravity). The zone coefficients (Z) of the four zones are: Z=0.12 (Zone 1), Z=0.20 (Zone 2), Z=0.28 (Zone 3) and Z=0.36 (Zone 4). Table 6.2.15 lists zone coefficients for some important towns of Bangladesh. The most severe earthquake prone zone, Zone 4 is in the northeast which includes Sylhet and has a maximum PGA value of 0.36g. Dhaka city falls in the moderate seismic intensity zone with Z=0.2, while Chittagong city falls in a severe intensity zone with Z=0.28.

2.5.4.3 Design response spectrum

The earthquake ground motion for which the building has to be designed is represented by the design response spectrum. Both static and dynamic analysis methods are based on this response spectrum. This spectrum represents the spectral acceleration for which the building has to be designed as a function of the building period, taking into account the ground motion intensity. The spectrum is based on elastic analysis but in order to account for energy dissipation due to inelastic deformation and benefits of structural redundancy, the spectral accelerations are reduced by the response modification factor *R*. For important structures, the spectral accelerations are increased by the importance factor *I*. The design basis earthquake (DBE) ground motion is selected at a ground shaking level that is 2/3 of the maximum considered earthquake (MCE) ground motion. The effect of local soil conditions on the response spectrum is incorporated in the normalized acceleration response spectrum C_s. The spectral acceleration for the design earthquake is given by the following equation:

$$S_a = \frac{2}{3}\frac{ZI}{R} C_s \tag{6.2.34}$$

Where,

S_a = Design spectral acceleration (in units of g which shall not be less than 0.67þZIS

þ = Coefficient used to calculate lower bound for S_a. Recommended value for þ is 0.11

Z = Seismic zone coefficient, as defined in Sec 2.5.4.2

I = Structure importance factor, as defined in Sec 2.5.5.1

R = Response reduction factor which depends on the type of structural system given in Table 6.2.19. The ratio $\frac{I}{R}$ cannot be greater than one.

C_c = Normalized acceleration response spectrum, which is a function of structure (building) period and soil type (site class) as defined by Equations 6.2.35a to 6.2.35d.

$$C_s = S\left(1 + \frac{T}{T_B}(2.5\eta - 1)\right) \text{for } 0 \le T \le T_B \tag{6.2.35a}$$

$$C_s = 2.5S\eta \quad \text{for } T_B \le T \le T_C \tag{6.2.35b}$$

$$C_s = 2.5S\eta\left(\frac{T_C}{T}\right) \quad \text{for } T_C \le T \le T_D \tag{6.2.35c}$$

$$C_s = 2.5S\eta\left(\frac{T_C T_D}{T^2}\right) \quad \text{for } T_D \le T \le 4\text{sec} \tag{6.2.35d}$$

C_c depends on *S* and values of T_B, T_C and T_D, (Figure 6.2.25) which are all functions of the site class. Constant C_s value between periods T_B and T_C represents constant spectral acceleration.

S = Soil factor which depends on site class and is given in Table 6.2.16

T = Structure (building) period as defined in Sec 2.5.7.2

T_B = Lower limit of the period of the constant spectral acceleration branch given in Table 6.2.16 as a function of site class.

T_C = Upper limit of the period of the constant spectral acceleration branch given in Table 6.2.16 as a function of site class

T_D = Lower limit of the period of the constant spectral displacement branch given in Table 6.2.16 as a function of site class

η = Damping correction factor as a function of damping with a reference value of η=1 for 5% viscous damping. It is given by the following expression:

$$\eta = \sqrt{10/(5+\xi)} \geq 0.55 \qquad (6.2.36)$$

Where, ξ is the viscous damping ratio of the structure, expressed as a percentage of critical damping. The value of η cannot be smaller than 0.55.

The anticipated (design basis earthquake) peak ground acceleration (PGA) for rock or very stiff soil (site class SA) is $\frac{2}{3}\mathrm{Z}$. However, for design, the ground motion is modified through the use of response reduction factor R and importance factor I, resulting in $\mathrm{PGA_{rock}} = \frac{2}{3}(\frac{\mathrm{ZI}}{\mathrm{R}})$. Figure 6.2.26 shows the normalized acceleration response spectrum C_s for 5% damping, which may be defined as the 5% damped spectral acceleration (obtained by Eq. 6.2.34) normalized with respect to $\mathrm{PGA_{rock}}$. This Figure demonstrates the significant influence of site class on the response spectrum.

Design Spectrum for Elastic Analysis

For site classes SA to SE, the design acceleration response spectrum for elastic analysis methods is obtained using Eq. 6.2.34 to compute S_a (in units of g) as a function of period T. The design acceleration response spectrum represents the expected ground motion (Design Basis Earthquake) divided by the factor R/I.

Design Spectrum for Inelastic Analysis

For inelastic analysis methods, the anticipated ground motion (Design Basis Earthquake) is directly used. Corresponding real design acceleration response spectrum is used, which is obtained by using *R*=1 and *I*=1 in Eq. 6.2.34. The 'real design acceleration response spectrum' is equal to 'design acceleration response spectrum' multiplied by *R*/*I*.

Site-Specific Design Spectrum

For site class S_1 and S_2, site-specific studies are needed to obtain design response spectrum. For important projects, site-specific studies may also be carried out to determine spectrum instead of using Eq. 6.2.34. The objective of such site-specific ground-motion analysis is to determine ground motions for local seismic and site conditions with higher confidence than is possible using simplified equations.

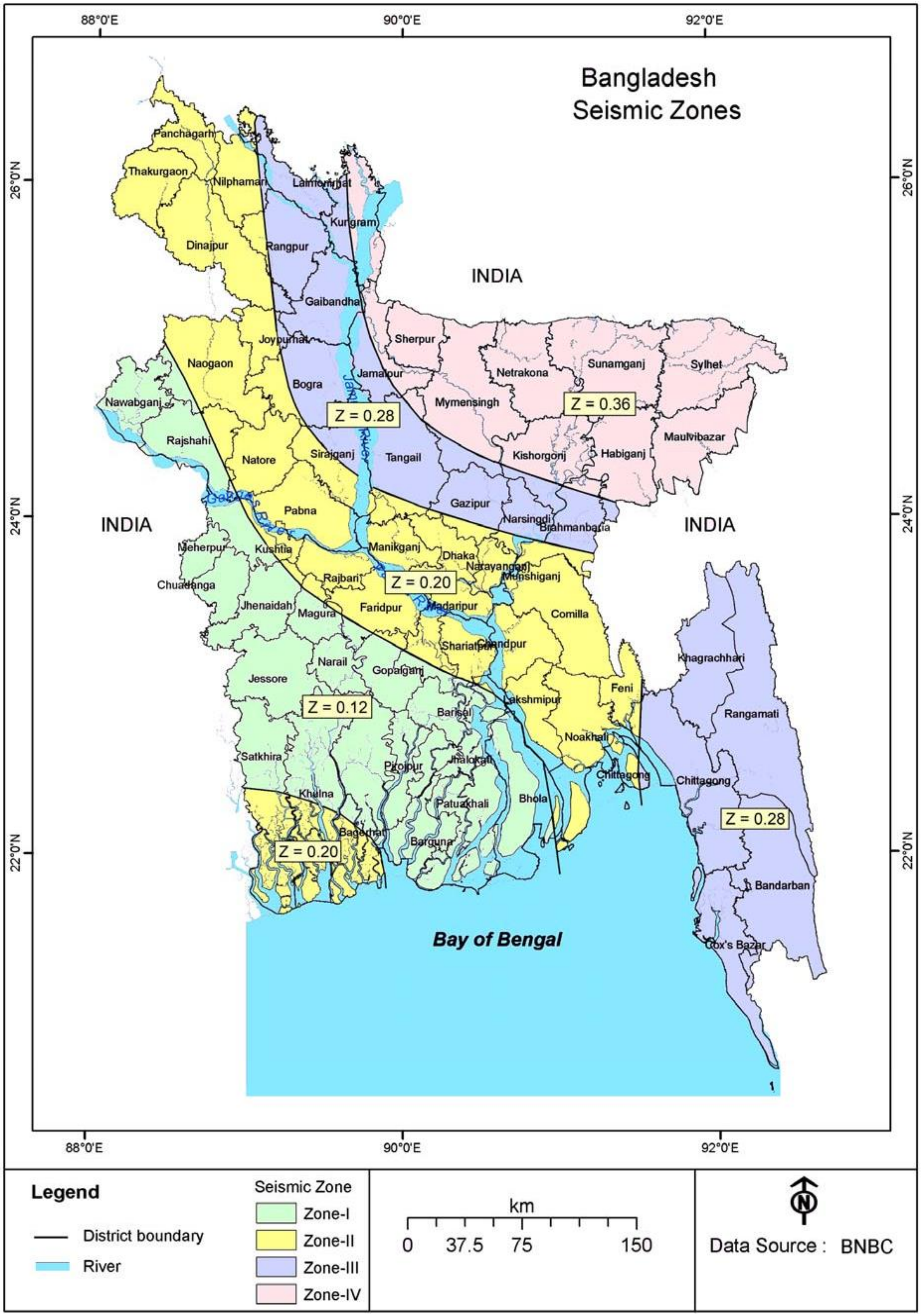


Figure 6.2.24 Seismic zoning map of Bangladesh

Table 6.2.14: Description of Seismic Zones

Seismic Zone	Location	Seismic Intensity	Seismic Zone Coefficient, Z
1	Southwestern part including Barisal, Khulna, Jessore, Rajshahi	Low	0.12
2	Lower Central and Northwestern part including Noakhali, Dhaka, Pabna, Dinajpur, as well as Southwestern corner including Sundarbans	Moderate	0.20
3	Upper Central and Northwestern part including Brahmanbaria, Sirajganj, Rangpur	Severe	0.28
4	Northeastern part including Sylhet, Mymensingh, Kurigram	Very Severe	0.36

Table 6.2.15: Seismic Zone Coefficient *Z* for Some Important Towns of Bangladesh

Town	Z	Town	Z	Town	Z	Town	Z
Bagerhat	0.12	Gaibandha	0.28	Magura	0.12	Patuakhali	0.12
Bandarban	0.28	Gazipur	0.20	Manikganj	0.20	Pirojpur	0.12
Barguna	0.12	Gopalganj	0.12	Maulvibazar	0.36	Rajbari	0.20
Barisal	0.12	Habiganj	0.36	Meherpur	0.12	Rajshahi	0.12
Bhola	0.12	Jaipurhat	0.20	Mongla	0.12	Rangamati	0.28
Bogra	0.28	Jamalpur	0.36	Munshiganj	0.20	Rangpur	0.28
Brahmanbaria	0.28	Jessore	0.12	Mymensingh	0.36	Satkhira	0.12
Chandpur	0.20	Jhalokati	0.12	Narail	0.12	Shariatpur	0.20
Chapainababganj	0.12	Jhenaidah	0.12	Narayanganj	0.20	Sherpur	0.36
Chittagong	0.28	Khagrachari	0.28	Narsingdi	0.28	Sirajganj	0.28
Chuadanga	0.12	Khulna	0.12	Natore	0.20	Srimangal	0.36
Comilla	0.20	Kishoreganj	0.36	Naogaon	0.20	Sunamganj	0.36
Cox's Bazar	0.28	Kurigram	0.36	Netrakona	0.36	Sylhet	0.36
Dhaka	0.20	Kushtia	0.20	Nilphamari	0.12	Tangail	0.28
Dinajpur	0.20	Lakshmipur	0.20	Noakhali	0.20	Thakurgaon	0.20
Faridpur	0.20	Lalmanirhat	0.28	Pabna	0.20		
Feni	0.20	Madaripur	0.20	Panchagarh	0.20		

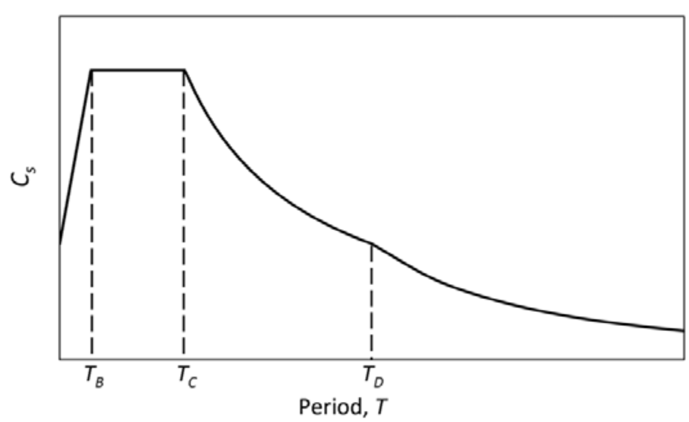


Figure 6.2.25 Typical shape of the elastic response spectrum coefficient C_s

Table 6.2.16: Site Dependent Soil Factor and Other Parameters Defining Elastic Response Spectrum

Soil type	S	T_B(s)	T_C (s)	T_D (s)
SA	1.0	0.15	0.40	2.0
SB	1.2	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.4	0.15	0.50	2.0

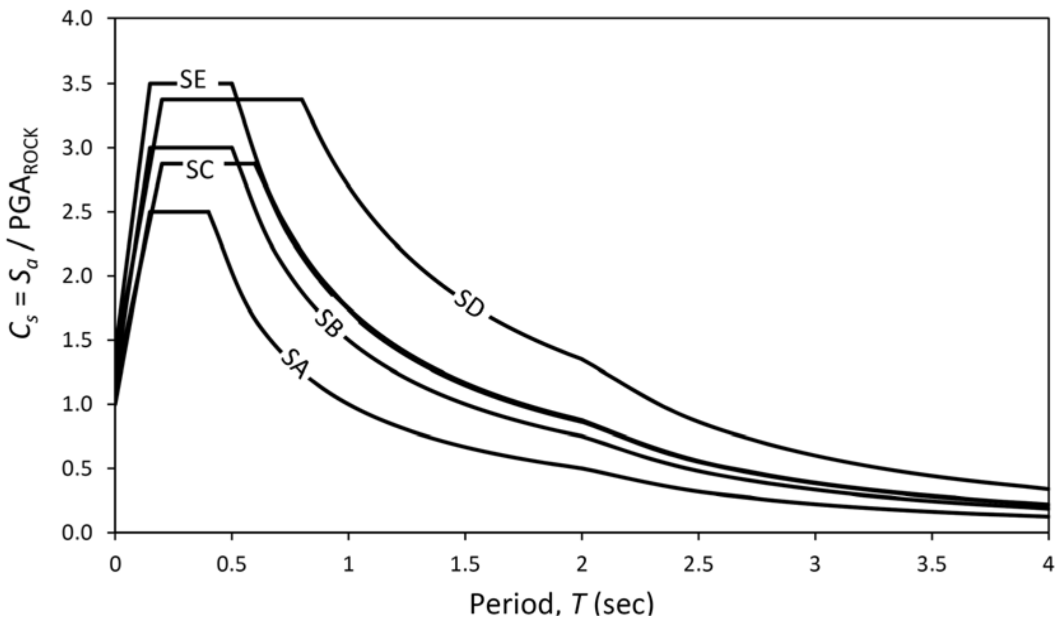


Figure 6.2.26 Normalized design acceleration response spectrum for different site classes.

2.5.5 Building Categories

2.5.5.1 Importance factor

Buildings are classified in four occupancy categories in Chapter 1 (Table 6.1.1), depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse. Depending on occupancy category, buildings may be designed for higher seismic forces using importance factor greater than one. Table 6.2.17 defines different occupancy categories and corresponding importance factor.

Table 6.2.17: Importance Factors for Buildings and Structures for Earthquake design

Occupancy Category	Importance factor I
I, II	1.00
III	1.25
IV	1.50

2.5.5.2 Seismic design category

Buildings shall be assigned a seismic design category among B, C or D based on seismic zone, local site conditions and importance class of building, as given in Table 6.2.18. Seismic design category D has the most stringent seismic design detailing, while seismic design category B has the least seismic design detailing requirements.

Table 6.2.18: Seismic Design Category of Buildings

Site Class	Occupancy Category I, II and III				Occupancy Category IV			
	Zone 1	Zone 2	Zone 3	Zone 4	Zone 1	Zone 2	Zone 3	Zone 4
SA	B	C	C	D	C	D	D	D
SB	B	C	D	D	C	D	D	D
SC	B	C	D	D	C	D	D	D
SD	C	D	D	D	D	D	D	D
SE, S_1, S_2	D	D	D	D	D	D	D	D

2.5.5.3 Building irregularity

Buildings with irregularity in plan or elevation suffer much more damage in earthquakes than buildings with regular configuration. A building may be considered as irregular, if at least one of the conditions given below are applicable:

2.5.5.3.1 Plan irregularity: Following are the different types of irregularities that may exist in the plan of a building.

(i) Torsion irregularity

To be considered for rigid floor diaphragms, when the maximum storey drift (Δ_{mas}) as shown in Figure 6.2.27(a), computed including accidental torsion, at one end of the structure is more than 1.2 times the average ($\Delta_{\text{avg}} = \frac{\Delta_{\text{max}}+\Delta_{\text{min}}}{2}$) of the storey drifts at the two ends of the structure. If $\Delta_{\text{mas}}\Sigma\ 1.4\Delta_{\text{avg}}$ then the irregularity is termed as extreme torsional irregularity.

(ii) Re-entrant corners

Both projections of the structure beyond a re-entrant comer [Figure 6.2.27(b)] are greater than 15 percent of its plan dimension in the given direction.

(iii) Diaphragm Discontinuity

Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out [Figure 6.2.27(c)] or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next.

(iv) Out- of-Plane Offsets

Discontinuities in a lateral force resistance path, such as out of-plane offsets of vertical elements, as shown in Figure 6.2.27(d).

(v) Non-parallel Systems

The vertical elements resisting the lateral force are not parallel to or symmetric [Figure 6.2.27(e)] about the major orthogonal axes of the lateral force resisting elements.

2.5.5.3.2 Vertical Irregularity: Following are different types of irregularities that may exist along vertical elevations of a building.

(i) Stiffness Irregularity - Soft Storey

A soft storey is one in which the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average lateral stiffness of the three storeys above irregularity [Figure 6.2.28(a)]. An extreme soft storey is defined where its lateral stiffness is less than 60% of that in the storey above or less than 70% of the average lateral stiffness of the three storeys above.

(ii) Mass Irregularity

The seismic weight of any storey is more than twice of that of its adjacent storeys [Figure 6.2.28(b)]. This irregularity need not be considered in case of roofs.

(iii) Vertical Geometric Irregularity

This irregularity exists for buildings with setbacks with dimensions given in Figure 6.2.28(c).

(iv) Vertical In-Plane Discontinuity in Vertical Elements Resisting Lateral Force

An in-plane offset of the lateral force resisting elements greater than the length of those elements Figure 6.2.28(d).

(v) Discontinuity in Capacity - Weak Storey

A weak storey is one in which the storey lateral strength is less than 80% of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction [Figure 6.2.28(e)]. An extreme weak storey is one where the storey lateral strength is less than 65% of that in the storey above.

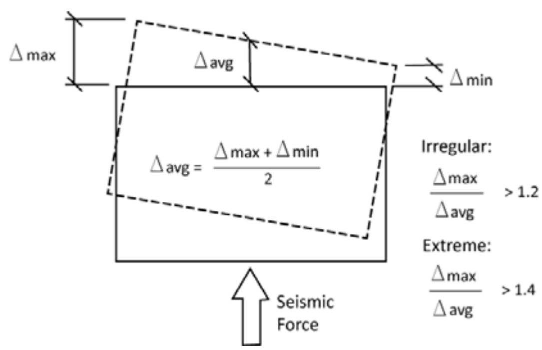
2.5.5.4 Type of structural systems

The basic lateral and vertical seismic force-resisting system shall conform to one of the types A to G indicated in Table 6.2.19. Each type is again subdivided by the types of vertical elements used to resist lateral seismic forces. A combination of systems may also be permitted as stated in Sec 2.5.5.5.

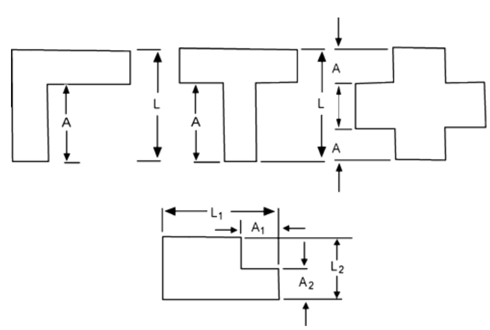
The structural system to be used shall be in accordance with the seismic design category indicated in Table 6.2.18. Structural systems that are not permitted for a certain seismic design category are indicated by “NP”. Structural systems that do not have any height restriction are indicated by “NL”. Where there is height limit, the maximum height in meters is given.

The response reduction factor, *R*, and the deflection amplification factor, C_d indicated in Table 6.2.19 shall be used in determining the design base shear and design story drift. The selected seismic force-resisting system shall be designed and detailed in accordance with the specific requirements for the system.

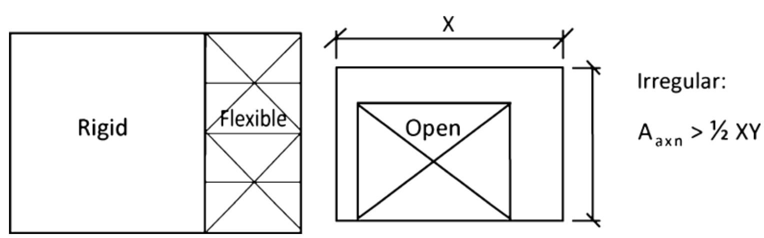
Seismic force resisting systems that are not given in Table 6.2.19 may be permitted if substantial analytical and test data are submitted that establish the dynamic characteristics and demonstrate the lateral force resistance and energy dissipation capacity to be equivalent to the structural systems listed in Table 6.2.19 for equivalent response modification coefficient, *R*, and deflection amplification factor, C_d values.



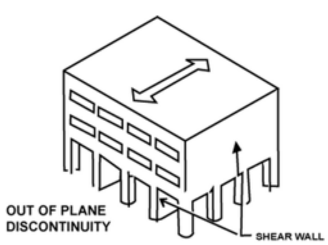
(a) Torsional Irregularity



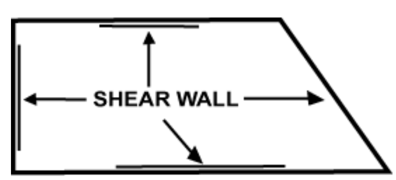
(b) Re-entrant corners (A/L>0.15)



(c) Diaphragm discontinuity



(d) Out-of-plane offsets of shear wall



(e) Non-parallel systems of shear wall

Figure 6.2.27 Different types of plan irregularities of buildings

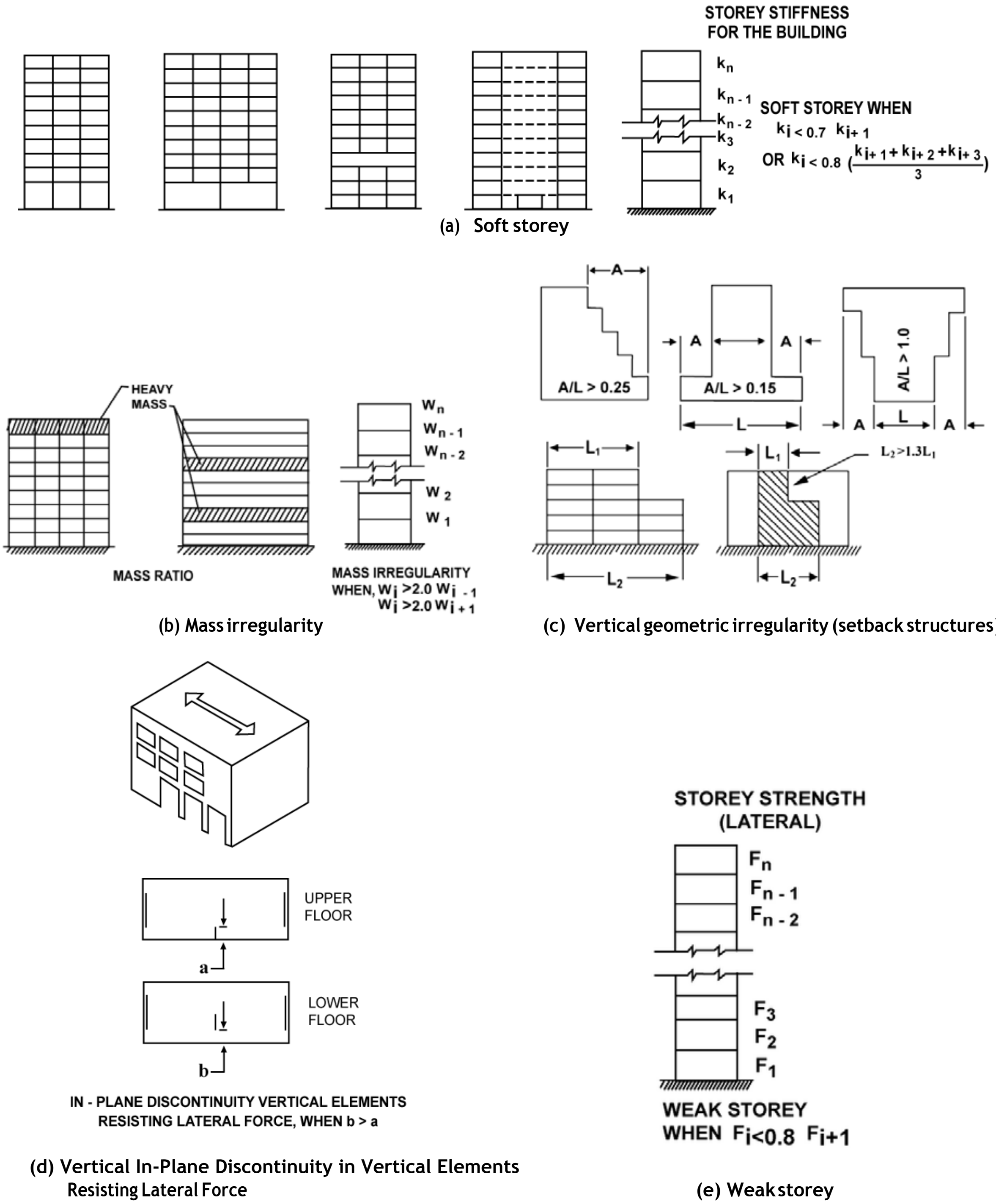


Figure 6.2.28 Different types of vertical irregularities of buildings

Table 6.2.19: Response Reduction Factor, Deflection Amplification Factor and Height Limitations for Different Structural Systems

Seismic Force-Resisting System	Response Reduction Factor, *R*	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Seismic Design Category B	Seismic Design Category C	Seismic Design Category D
				Height limit (m)		
A. BEARING WALL SYSTEMS (no frame)						
1. Special reinforced concrete shear walls	5	2.5	5	NL	NL	50
2. Ordinary reinforced concrete shear walls	4	2.5	4	NL	NL	NP
3. Ordinary reinforced masonry shear walls	2	2.5	1.75	NL	50	NP
4. Ordinary plain masonry shear walls	1.5	2.5	1.25	**18**	NP	NP
B. BUILDING FRAME SYSTEMS (with bracing or shear wall)						
1. Steel eccentrically braced frames, moment resisting connections at columns away from links	8	2	4	NL	NL	50
2. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	7	2	4	NL	NL	50
3. Special steel concentrically braced frames	6	2	5	NL	NL	50
4. Ordinary steel concentrically braced frames	3.25	2	3.25	NL	NL	11
5. Special reinforced concrete shear walls	6	2.5	5	NL	NL	50
6. Ordinary reinforced concrete shear walls	5	2.5	4.25	NL	NL	NP
7. Ordinary reinforced masonry shear walls	2	2.5	2	NL	50	NP
8. Ordinary plain masonry shear walls	1.5	2.5	1.25	**18**	NP	NP
C. MOMENT RESISTING FRAME SYSTEMS (no shear wall)						
1. Special steel moment frames	8	3	5.5	NL	NL	NL
2. Intermediate steel moment frames	4.5	3	4	NL	NL	35
3. Ordinary steel moment frames	3.5	3	3	NL	NL	NP
4. Special reinforced concrete moment frames	8	3	5.5	NL	NL	NL
5. Intermediate reinforced concrete moment frames	5	3	4.5	NL	NL	NP
6. Ordinary reinforced concrete moment frames	3	3	2.5	NL	NP	NP
D. DUAL SYSTEMS: SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)						
1. Steel eccentrically braced frames	8	2.5	4	NL	NL	NL
2. Special steel concentrically braced frames	7	2.5	5.5	NL	NL	NL
3. Special reinforced concrete shear walls	7	2.5	5.5	NL	NL	NL
4. Ordinary reinforced concrete shear walls	6	2.5	5	NL	NL	NP

Seismic Force-Resisting System	Response Reduction Factor, R	System Overstrength Factor, Ω_o	Deflection Amplification Factor, C_d	Seismic Design Category B	Seismic Design Category C	Seismic Design Category D
				Height limit (m)		
E. DUAL SYSTEMS: INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES (with bracing or shear wall)						
1. Special steel concentrically braced frames	6	2.5	5	NL	NL	11
2. Special reinforced concrete shear walls	6.5	2.5	5	NL	NL	50
3. Ordinary reinforced masonry shear walls	3	3	3	NL	50	NP
4. Ordinary reinforced concrete shear walls	5.5	2.5	4.5	NL	NL	NP
F. DUAL SHEAR WALL-FRAME SYSTEM: ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	4.5	2.5	4	NL	NP	NP
G. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE	3	3	3	NL	NL	NP

Notes:

1. Seismic design category, NL = No height restriction, NP = Not permitted. Number represents maximum allowable height (m).
2. Dual Systems include buildings which consist of both moment resisting frame and shear walls (or braced frame) where both systems resist the total design forces in proportion to their lateral stiffness.
3. See Sec. 10.20 of Chapter 10 of this Part for additional values of R and C_d and height limits for some other types of steel structures not covered in this Table.
4. Where data specific to a structure type is not available in this Table, reference may be made to Table 12.2-1 of ASCE 7-05.

2.5.5.5 Combination of structural systems

2.5.5.5.1 Combinations of Structural Systems in Different Directions: Different seismic force-resisting systems are permitted to be used to resist seismic forces along each of the two orthogonal axes of the structure. Where different systems are used, the respective R and C_d coefficients shall apply to each system, including the limitations on system use contained in Table 6.2.19.

2.5.5.5.2 Combinations of Structural Systems in the Same Direction: Where different seismic force-resisting systems are used in combination to resist seismic forces in the same direction of structural response, other than those combinations considered as dual systems, the more stringent system limitation contained in Table 6.2.19 shall apply. The value of *R* used for design in that direction shall not be greater than the least value of *R* for any of the systems utilized in that direction. The deflection amplification factor, C_d in the direction under consideration at any story shall not be less than the largest value of this factor for the *R* factor used in the same direction being considered.

2.5.5.6 Provisions for Using System Overstrength Factor, Ω_o

2.5.5.6.1 Combinations of Elements Supporting Discontinuous Walls or Frames.

Columns, beams, trusses, or slabs supporting discontinuous walls or frames of structures having horizontal irregularity Type IV of Table 6.1.5 or vertical irregularity Type IV of Table 6.1.4 shall have the design strength to resist the maximum axial force that can develop in accordance with the load combinations with overstrength factor of Section 2.5.13.4. The connections of such discontinuous elements to the supporting members shall be adequate to transmit the forces for which the discontinuous elements were required to be designed.

2.5.5.6.2 Increase in Forces Due to Irregularities for Seismic Design Categories D through E.

For structures assigned to Seismic Design Category D or E and having a horizontal structural irregularity of Type I.a, I.b, II, III, or IV in Table 6.1.5 or a vertical structural irregularity of Type IV in Table 6.1.4, the design forces determined from Section 2.5.7 shall be increased 25 percent for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements. Collectors and their connections also shall be designed for these increased forces unless they are designed for the load combinations with overstrength factor of Section 2.5.5.4, in accordance with Section 2.5.13.4.

2.5.5.6.3 Collector Elements Requiring Load Combinations with Overstrength Factor for Seismic Design Categories C through E.

In structures assigned to Seismic Design Category C, D or E, collector elements, splices, and their connections to resisting elements shall resist the load combinations with overstrength of Section 2.5.13.4.

2.5.5.6.4 Batter Piles.

Batter piles and their connections shall be capable of resisting forces and moments from the load combinations with overstrength factor of Section 2.5.13.4. Where vertical and batter piles act jointly to resist foundation forces as a group, these forces shall be distributed to the individual piles in accordance with their relative horizontal and vertical rigidities and the geometric distribution of the piles within the group.

2.5.6 Static Analysis Procedure

Although analysis of buildings subjected to dynamic earthquake loads should theoretically require dynamic analysis procedures, for certain type of building structures subjected to earthquake shaking, simplified static analysis procedures may also provide reasonably good results. The equivalent static force method is such a procedure for determining the seismic lateral forces acting on the structure. This type of analysis may be applied to buildings whose seismic response is not significantly affected by contributions from modes higher than the fundamental mode in each direction. This requirement is deemed to be satisfied in buildings which fulfill the following two conditions:

(a) The building period in the two main horizontal directions is smaller than both $4T_C$ (T_C is defined in Sec 2.5.4.3) and 2 seconds.

(b) The building does not possess irregularity in elevation as defined in Sec 2.5.5.3.

2.5.7 Equivalent Static Analysis

The evaluation of the seismic loads starts with the calculation of the design base shear which is derived from the design response spectrum presented in Sec 2.5.4.3. This Section presents different computations relevant to the equivalent static analysis procedure.

2.5.7.1 Design base shear

The seismic design base shear force in a given direction shall be determined from the following relation:

$$V = S_a W \tag{6.2.37}$$

Where,

S_a = Lateral seismic force coefficient calculated using Eq. 6.2.34 (Sec 2.5.4.3). It is the design spectral acceleration (in units of g) corresponding to the building period T (computed as per Sec 2.5.7.2).

W = Total seismic weight of the building defined in Sec 2.5.7.3

Alternatively, for buildings with natural period less than or equal to 2.0 sec., the seismic design base shear can be calculated using ASCE 7-02 with seismic design parameters as given in Appendix C. However, the minimum value of S_a should not be less than 0.044 $S_{DS}I$. The values of S_{DS} are provided in Table 6.C.4 Appendix C.

2.5.7.2 Building period

The fundamental period T of the building in the horizontal direction under consideration shall be determined using the following guidelines:

(a) Structural dynamics procedures (such as Rayleigh method or modal eigenvalue analysis), using structural properties and deformation characteristics of resisting elements, may be used to determine the fundamental period T of the building in the direction under consideration. This period shall not exceed the approximate fundamental period determined by Eq. 6.2.38 by more than 40 percent.

(b) The building period T (in secs) may be approximated by the following formula:

$$T = C_t(h_n)^m \tag{6.2.38}$$

Where,

h_n = Height of building in metres from foundation or from top of rigid basement. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But it includes the basement storeys, when they are not so connected. C_t and m are obtained from Table 6.2.20

(c) For masonry or concrete shear wall structures, the approximate fundamental period, T in sec may be determined as follows:

$$T = \frac{0.0062}{\sqrt{C_w}} h_n \tag{6.2.39}$$

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} \tag{6.2.40}$$

Where,

A_B = area of base of structure

A_i = web area of shear wall "i"

D_i = length of shear wall "i"

h_i = height of shear wall "i"

x = number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

Table 6.2.20: Values for Coefficients to Estimate Approximate Period

Structure type	C_t	m	
Concrete moment-resisting frames	0.0466	0.9	Note: Consider moment resisting frames as frames which resist 100% of seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting under seismic forces.
Steel moment-resisting frames	0.0724	0.8	
Eccentrically braced steel frame	0.0731	0.75	
All other structural systems	0.0488	0.75	

2.5.7.3 Seismic weight

Seismic weight, W, is the total dead load of a building or a structure, including partition walls, and applicable portions of other imposed loads listed below:

(a) For live load up to and including 3 kN/m^2, a minimum of 25 percent of the live load shall be applicable.

(b) For live load above 3 kN/m^2, a minimum of 50 percent of the live load shall be applicable.

(c) Total weight (100 percent) of permanent heavy equipment or retained liquid or any imposed load sustained in nature shall be included.

Where the probable imposed loads (mass) at the time of earthquake are more correctly assessed, the designer may go for higher percentage of live load.

2.5.7.4 Vertical distribution of lateral forces

In the absence of a more rigorous procedure, the total seismic lateral force at the base level, in other words the base shear V, shall be considered as the sum of lateral forces F_s induced at different floor levels, these forces may be calculated as:

$$F_x = V \frac{w_x h_x^{\,k}}{\sum_{i=1}^{n} w_i h_i^{\,k}} \tag{6.2.41}$$

Where,

F_s = Part of base shear force induced at level *x*

w_i and w_s = Part of the total effective seismic weight of the structure (*W*) assigned to level *i* or *x*

h_i and h_s = the height from the base to level *i* or *x*

k = 1 For structure period ≤ 0.5s

= 2 for structure period ≥ 2.5s

= linear interpolation between 1 and 2 for other periods.

n = number of stories

2.5.7.5 Storey shear and its horizontal distribution

The design storey shear V_s, at any storey x is the sum of the forces F_s in that storey and all other stories above it, given by Eq. 6.2.42:

$$V_x = \sum_{i=x}^{n} F_i \tag{6.2.42}$$

Where, F_i = Portion of base shear induced at level *i*, as determined by Eq. 6.2.41.

If the floor diaphragms can be considered to be infinitely rigid in the horizontal plane, the shear V_s shall be distributed to the various elements of the lateral force resisting system in proportion to their relative lateral stiffness. For flexible diaphragms, the distribution of forces to the vertical elements shall account for the position and distribution of the masses supported.

Allowance shall also be made for the increased shear arising due to horizontal torsional moment as specified in Sec 2.5.7.6

2.5.7.6 Horizontal torsional moments

Design shall accommodate increase in storey shear forces resulting from probable horizontal torsional moments on rigid floor diaphragms. Computation of such moments shall be as follows:

2.5.7.6.1 In-built torsional effects: When there is in-built eccentricity between centre of mass and centre of rigidity (lateral resistance) at floor levels, rigid diaphragms at each level will be subject to torsional moment M_t.

2.5.7.6.2 Accidental torsional effects: In order to account for uncertainties in the location of masses and in the spatial variation of the seismic motion, accidental torsional effects need to be always considered. The accidental moment M_{ta} is determined assuming the storey mass to be displaced from the calculated centre of mass a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration. The accidental torsional moment M_{tai} at level i is given as:

$$M_{tai} = e_{ai} F_i \tag{6.2.43}$$

Where,

e_{ai} = accidental eccentricity of floor mass at level i applied in the same direction at all floors = $\pm 0.05 L_i$

L_i = floor dimension perpendicular to the direction of seismic force considered.

Where torsional irregularity exists (Sec 2.5.5.3.1) for Seismic Design Category C or D, the irregularity effects shall be accounted for by increasing the accidental torsion M_{ta} at each level by a torsional amplification factor, A_s as illustrated in Figure 6.2.29 determined from the following equation:

$$A_s = \left[\frac{ð_{max}}{1.2ð_{avg}}\right]^2 \leq 3.0 \tag{6.2.44}$$

Where,

$ð_{mas}$ = Maximum displacement at level-x computed assuming $A_x = 1$.

$ð_{avg}$ = Average displacements at extreme points of the building at level-x computed assuming $A_x = 1$.

The accidental torsional moment need not be amplified for structures of light-frame construction. Also the torsional amplification factor (A_S) should not exceed 3.0.

2.5.7.6.3 Design for torsional effects: The torsional design moment at a given storey shall be equal to the accidental torsional moment M_{ta} plus the inbuilt torsional moment M_t (if any). Where earthquake forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass (for accidental torsion) need not be applied in both of the orthogonal directions at the same time, but shall be applied in only one direction that produces the greater effect.

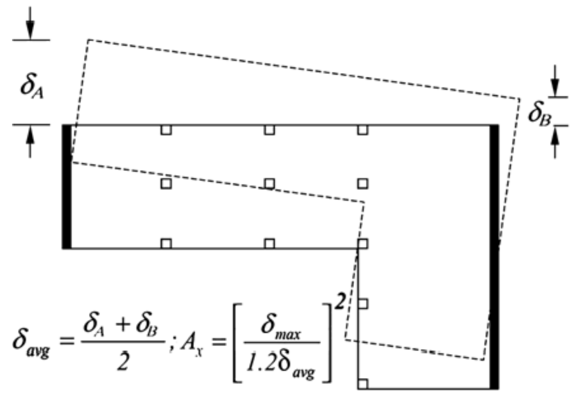


Figure 6.2.29 Torsional amplification factor A_x for plan irregularity.

2.5.7.7 Deflection and storey drift

The deflections (δ_S) of level x at the center of the mass shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad (6.2.45)$$

Where,

C_d = Deflection amplification factor given in Table 6.2.19

δ_{xe} = Deflection determined by an elastic analysis

I = Importance factor defined in Table 6.2.17

The design storey drift at storey x shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration:

$$\Delta_x = \delta_x - \delta_{x-1} \qquad (6.2.46)$$

2.5.7.8 Overturning effects

The structure shall be designed to resist overturning effects caused by the seismic forces determined in Sec 2.5.7.4. At any story, the increment of overturning moment in the story under consideration shall be distributed to the various vertical force resisting elements in the same proportion as the distribution of the horizontal shears to those elements. The overturning moments at level x, M_s shall be determined as follows:

$$M_x = \sum_{i=x}^{n} F_i \left(h_i - h_x\right) \qquad (6.2.47)$$

Where,

F_i = Portion of the seismic base shear, V induced at level i

h_i, h_s = Height from the base to level i or x.

The foundations of structures, except inverted pendulum-type structures, shall be permitted to be designed for three-fourths of the foundation overturning design moment, M_o determined using above equation.

2.5.7.9 P-delta effects

The P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered if the stability coefficient (θ) determined by the following equation is not more than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad (6.2.48)$$

Where,

P_s = Total vertical design load at and above level x; where computing P_s, no individual load factor need exceed 1.0

Δ = Design story drift occurring simultaneously with V_s

V_s = Storey shear force acting between levels x and $x - 1$

h_{cs} = Storey height below level x

C_d = Deflection amplification factor given in Table 6.2.19

The stability coefficient 8 shall not exceed 8_{mas} determined as follows:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25 \qquad (6.2.49)$$

Where, þ is the ratio of shear demand to shear capacity for the story between levels x and $x - 1$. This ratio is permitted to be conservatively taken as 1.0.

Where, the stability coefficient 8 is greater than 0.10 but less than or equal to 8_{mas}, the incremental factor related to P-delta effects on displacements and member forces shall be determined by rational analysis. Alternatively, it is permitted to multiply displacements and member forces by $\frac{1}{(1-8)}$.

Where, 8 is greater than 8_{mas}, the structure is potentially unstable and shall be redesigned.

Where, the P-delta effect is included in an automated analysis, Eq. 6.2.49 shall still be satisfied, however, the value of 8 computed from Eq. 6.2.48 using the results of the P-delta analysis is permitted to be divided by $(1 + 8)$ before checking Eq. 6.2.49.

2.5.8 Dynamic Analysis Methods

Dynamic analysis method involves applying principles of structural dynamics to compute the response of the structure to applied dynamic (earthquake) loads.

2.5.8.1 Requirement for dynamic analysis

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

(a) Regular buildings with height greater than 40 m in Zones 2, 3, 4 and greater than 90 m in Zone 1.

(b) Irregular buildings (as defined in Sec 2.5.5.3) with height greater than 12 m in Zones 2, 3, 4 and greater than 40 m in Zone 1.

For irregular buildings, smaller than 40 m in height in Zone 1, dynamic analysis, even though not mandatory, is recommended.

2.5.8.2 Methods of analysis

Dynamic analysis may be carried out through the following two methods:

(i) Response Spectrum Analysis method is a linear elastic analysis method using modal analysis procedures, where the structure is subjected to spectral accelerations corresponding to a design acceleration response spectrum. The design earthquake ground motion in this case is represented by its response spectrum.

(ii) Time History Analysis method is a numerical integration procedure where design ground motion time histories (acceleration record) are applied at the base of the structure. Time history analysis procedures can be two types: linear and non-linear.

2.5.9 Response Spectrum Analysis (RSA)

A response spectrum analysis shall consist of the analysis of a linear mathematical model of the structure to determine the maximum accelerations, forces, and displacements resulting from the dynamic response to ground shaking represented by the design acceleration response spectrum (presented in Sec 2.5.4.3). Response spectrum analysis is also called a modal analysis procedure because it considers different modes of vibration of the structure and combines effects of different modes.

2.5.9.1 Modeling (RSA)

A mathematical model of the structure shall be constructed that represents the spatial distribution of mass and stiffness throughout the structure. For regular structures with independent orthogonal seismic-force-resisting systems, independent two-dimensional models are permitted to be constructed to represent each system. For irregular structures or structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the structure. Where the diaphragms are not rigid compared to the vertical elements of the lateral-force-resisting system, the model should include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response. The structure shall be considered to be fixed at the base or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations. In addition, the model shall comply with the following:

(a) Stiffness properties of concrete and masonry elements shall consider the effects of cracked sections

(b) The contribution of panel zone deformations to overall story drift shall be included for steel moment frame resisting systems.

2.5.9.2 Number of modes (RSA)

An analysis shall be conducted using the masses and elastic stiffnesses of the seismic-force-resisting system to determine the natural modes of vibration for the structure including the period of each mode, the modal shape vector $, the modal participation factor P and modal mass M. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of two orthogonal directions.

2.5.9.3 Modal story shears and moments (RSA)

For each mode, the story shears, story overturning moments, and the shear forces and overturning moments in vertical elements of the structural system at each level due to the seismic forces shall be computed. The peak lateral force F_{ik} induced at level i in mode k is given by:

$$F_{ik} = A_k \$_{ik} P_k W_i \tag{6.2.50}$$

Where,

A_k = Design horizontal spectral acceleration corresponding to period of vibration T_k of mode k obtained from design response spectrum (Sec 2.5.4.3)

$\$_{ik}$ = Modal shape coefficient at level i in mode k

P_k = Modal participation factor of mode k

W_i = Weight of floor i.

2.5.9.4 Structure response (RSA)

In the response spectrum analysis method, the base shear V_{rc}; each of the story shear, moment, and drift quantities; and the deflection at each level shall be determined by combining their modal values. The combination shall be carried out by taking the square root of the sum of the squares (SRSS) of each of the modal values or by the complete quadratic combination (CQC) technique. The complete quadratic combination shall be used where closely spaced periods in the translational and torsional modes result in cross-correlation of the modes.

The distribution of horizontal shear shall be in accordance with the requirements of Sec 2.5.7.5. It should be noted that amplification of accidental torsion as per Sec 2.5.7.6 is not required where accidental torsional effects are included in the dynamic analysis model by offsetting the centre of mass in each story by the required amount.

A base shear, V shall also be calculated using the equivalent static force procedure in Sec 2.5.7. Where the base shear, V_{rc} is less than 85 percent of V all the forces but not the drifts obtained by response spectrum analysis shall be multiplied by the ratio $\frac{0.85V}{V_{rc}}$.

The displacements and drifts obtained by response spectrum analysis shall be multiplied by C_d/I to obtain design displacements and drifts, as done in equivalent static analysis procedure (Sec 2.5.7.7). The P-delta effects shall be determined in accordance with Sec 2.5.7.9.

2.5.10 Linear Time History Analysis (LTHA)

A linear time history analysis (LTHA) shall consist of an analysis of a linear mathematical model of the structure to determine its response, through direct numerical integration of the differential equations of motion, to a number of ground motion acceleration time histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with the provisions of this Section. For the purposes of analysis, the structure shall be permitted to be considered to be fixed at the base or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations. The acceleration time history (ground motion) is applied at the base of the structure. The advantage of this procedure is that the time dependent behavior of the structural response is obtained.

2.5.10.1 Modeling (LTHA)

Mathematical models shall conform to the requirements of modeling described in Sec 2.5.9.1.

2.5.10.2 Ground motion (LTHA)

At least three appropriate ground motions (acceleration time history) shall be used in the analysis. Ground motion shall conform to the requirements of this Section.

Two-dimensional analysis: Where two-dimensional analyses are performed, each ground motion shall consist of a horizontal acceleration time history selected from an actual recorded event. Appropriate acceleration histories shall be obtained from records of events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of appropriate ground motion records are not available, appropriate simulated ground motion time histories shall be used to make up the total number required. The ground motions shall be scaled such that for each period between 0.2T and 1.5T (where T is the natural period of the structure in the fundamental mode for the direction considered) the average of the five-percent-damped response spectra for the each acceleration time history is not less than the corresponding ordinate of the design acceleration response spectrum, determined in accordance with Sec2.5.4.3.

Three-dimensional analysis: Where three-dimensional analysis is performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration time histories (in two orthogonal horizontal directions) that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of recorded ground motion pairs are not available, appropriate simulated ground motion pairs shall be used to make up the total number required. For each pair of horizontal ground motion components, an SRSS spectrum shall be constructed by taking the square root of the sum of the squares of the five-percent-damped response spectra for the components (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that for each period between 0.2T and 1.5T (where T is the natural period of the fundamental mode of the structure) the average of the SRSS spectra from all horizontal component pairs is not less than 1.3 times the corresponding ordinate of the design response spectrum, determined in accordance with Sec 2.5.4.3.

2.5.10.3 Structure response (LTHA)

For each scaled acceleration time history, the maximum values of base shear and other structure response quantities shall be obtained from the time history analysis. For three dimensional analysis, orthogonal pair of scaled motions are applied simultaneously. A base shear, *V*, shall also be calculated using the equivalent static force procedure described in Sec 2.5.7.1. Where the maximum base shear, V_{th} computed by linear time history analysis, is less than *V*, all response quantities (storey shear, moments, drifts, floor deflections, member forces etc) obtained by time history analysis shall be increased by multiplying with the ratio, $\frac{V}{V_{th}}$. If number of earthquake records (or pairs) used in the analysis is less than seven, the maximum structural response obtained corresponding to different earthquake records shall be considered as the design value. If the number is at least seven, then the average of maximum structural responses for different earthquake records shall be considered as the design value.

The displacements and drifts obtained as mentioned above shall be multiplied by $\frac{C_d}{I}$ to obtain design displacements and drifts, as done in equivalent static analysis procedure (Sec 2.5.7.7).

2.5.11 Non-Linear Time History Analysis (NTHA)

Nonlinear time history analysis (NTHA) shall consist of analysis of a mathematical model of the structure which incorporates the nonlinear hysteretic behavior of the structure's components to determine its response, through methods of numerical integration, to ground acceleration time histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with the requirements of this Section. For the purposes of analysis, the structure shall be permitted to be considered to be fixed at the base or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness of foundations. The acceleration time history (ground motion) is applied at the base of the structure. The advantage of this procedure is that actual time dependent behavior of the structural response considering inelastic deformations in the structure can be obtained.

2.5.11.1 Modeling (NTHA)

A mathematical model of the structure shall be constructed that represents the spatial distribution of mass throughout the structure. The hysteretic behavior of elements shall be modeled consistent with suitable laboratory test data and shall account for all significant yielding, strength degradation, stiffness degradation, and hysteretic pinching indicated by such test data. Strength of elements shall be based on expected values considering material over-strength, strain hardening, and hysteretic strength degradation. As a minimum, a bilinear force deformation relationship should be used at the element level. In reinforced concrete and masonry buildings, the elastic stiffness should correspond to that of cracked sections. Linear properties, consistent with the provisions of Chapter 5 shall be permitted to be used for those elements demonstrated by the analysis to remain within their linear range of response. The structure shall be assumed to have a fixed base or, alternatively, it shall be permitted to use realistic assumptions with regard to the stiffness and load carrying characteristics of the foundations consistent with site-specific soils data and rational principles of engineering mechanics.

For regular structures with independent orthogonal seismic-force-resisting systems, independent two dimensional models shall be permitted to be constructed to represent each system. For structures having plan irregularity or structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis at each level of the structure shall be used. Where the diaphragms are not rigid compared to the vertical elements of the lateral-force-resisting system, the model shall include representation of the diaphragm's flexibility and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response.

2.5.11.2 Ground motion (NTHA)

The actual time-dependent inelastic deformation of the structure is modeled. For inelastic analysis method, the real design acceleration response spectrum (Sec 2.5.4.3) is obtained using Eq. 6.2.34 with R=1 and I=1. The real design acceleration response spectrum is the true representation of the expected ground motion (design basis earthquake) including local soil effects and corresponds to a peak ground acceleration (PGA) value of $\frac{2}{3}ZS$.

At least three appropriate acceleration time histories shall be used in the analysis. Ground motion shall conform to the requirements of this Section.

<u>Two-dimensional analysis</u>

Where two-dimensional analyses are performed, each ground motion shall consist of a horizontal acceleration time history selected from an actual recorded event. Appropriate acceleration histories shall be obtained from records of events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of appropriate ground motion records are not available, appropriate simulated ground motion time histories shall be used to make up the total number required. The ground motions shall be scaled such that for each period between $0.2T$ and $1.5T$ (where T is the natural period of the structure in the fundamental mode for the direction considered) the average of the five-percent-damped response spectra for each acceleration time history is not less than the corresponding ordinate of the real design acceleration response spectrum, as defined here.

<u>Three-dimensional analysis</u>

Where three-dimensional analysis is performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration time histories (in two orthogonal horizontal directions) that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of recorded ground motion pairs are not available, appropriate simulated ground motion pairs shall be used to make up the total number required. For each pair of horizontal ground motion components, an SRSS spectrum shall be constructed by taking the square root of the sum of the squares of the five-percent-damped response spectra for the components (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that for each period between $0.2T$ and $1.5T$ (where T is the natural period of the fundamental mode of the structure) the average of the SRSS spectra from all horizontal component pairs is not less than 1.3 times the corresponding ordinate of the real design acceleration response spectrum.

2.5.11.3 Structure response (NTHA)

For each scaled acceleration time history, the maximum values of base shear and other structure response quantities shall be obtained from the nonlinear time history analysis. For three dimensional analysis, orthogonal pair of scaled motions are applied simultaneously. If number of earthquake records (or pairs) used in the analysis is less than seven, the maximum structural response obtained corresponding to different earthquake records shall be considered as the design value. If the number is at least seven, then the average of maximum structural responses for different earthquake records shall be considered as the design value. Since real expected earthquake motion input and model incorporating real nonlinear behavior of the structure is used, the results as obtained are directly used (no scaling as in LTHA or RSA is required) for interpretation and design.

2.5.11.4 Structure member design (NTHA)

The adequacy of individual members and their connections to withstand the design deformations predicted by the analyses shall be evaluated based on laboratory test data for similar components. The effects of gravity and other loads on member deformation capacity shall be considered in these evaluations. Member deformation shall not exceed two thirds of the smaller of: the value that results in loss of ability to carry gravity loads or the value at which member strength has deteriorated to less than 67 percent of peak strength.

2.5.11.5 Design review (NTHA)

Special care and expertise is needed in the use of nonlinear dynamic analysis based design. Checking of the design by competent third party is recommended. A review of the design of the seismic-force-resisting system and the supporting structural analyses shall be performed by an independent team consisting of design professionals with experience in seismic analysis methods and the theory and application of nonlinear seismic analysis and structural behavior under extreme cyclic loads. The design review shall include the following: (i) Review of development of ground motion time histories (ii) Review of acceptance criteria (including laboratory test data) used to demonstrate the adequacy of structural elements and systems to withstand the calculated force and deformation demands (iii) Review of structural design.

2.5.12 Non-Linear Static Analysis (NSA)

Nonlinear static analysis (NSA), also popularly known as pushover analysis, is a simplified method of directly evaluating nonlinear response of structures to strong earthquake ground shaking. It is an alternative to the more complex nonlinear time history analysis (NTHA). The building is subjected to monotonically increasing static horizontal loads under constant gravity load.

2.5.12.1 Modeling (NSA)

A mathematical model of the structure shall be constructed to represent the spatial distribution of mass and stiffness of the structural system considering the effects of element nonlinearity for deformation levels that exceed the proportional limit. P-Delta effects shall also be included in the analysis.

For regular structures with independent orthogonal seismic-force-resisting systems, independent two-dimensional models may be used to represent each system. For structures having plan irregularities or structures without independent orthogonal systems, a three-dimensional model incorporating a minimum of three degrees of freedom for each level of the structure, consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis, shall be used. Where the diaphragms are not rigid compared to the vertical elements of the seismic-force-resisting system, the model should include representation of the diaphragm flexibility.

Unless analysis indicates that an element remains elastic, a nonlinear force deformation model shall be used to represent the stiffness of the element before onset of yield, the yield strength, and the stiffness properties of the element after yield at various levels of deformation. Strengths of elements shall not exceed expected values considering material over-strength and strain hardening. The properties of elements and components after yielding shall account for strength and stiffness degradation due to softening, buckling, or fracture as indicated by principles of mechanics or test data.

A control point shall be selected for the model. For normal buildings, the control point shall be at the center of mass of the highest level (roof) of the structure.

2.5.12.2 Analysis procedure (NSA)

The lateral forces shall be applied at the center of mass of each level and shall be proportional to the distribution obtained from a modal analysis for the fundamental mode of response in the direction under consideration. The lateral loads shall be increased incrementally in a monotonic manner.

At the j^{th} increment of lateral loading, the total lateral force applied to the model shall be characterized by the term V_j. The incremental increases in applied lateral force should be in steps that are sufficiently small to permit significant changes in individual element behavior (such as yielding, buckling or failure) to be detected. The first increment in lateral loading shall result in linear elastic behavior. At each loading step, the total applied lateral force, V_j the lateral displacement of the control point, δ_j and the forces and deformations in each element shall be recorded. The analysis shall be continued until the displacement of the control point is at least 150 percent of the target displacement determined in accordance with Sec.2.5.12.3. The structure shall be designed so that the total applied lateral force does not decrease in any load increment for control point displacements less than or equal to 125 percent of the target displacement.

2.5.12.3 Effective period and target displacement (NSA)

A bilinear curve shall be fitted to the capacity curve, such that the first segment of the bilinear curve coincides with the capacity curve at 60 percent of the effective yield strength, the second segment coincides with the capacity curve at the target displacement, and the area under the bilinear curve equals the area under the capacity curve, between the origin and the target displacement. The effective yield strength, V_y corresponds to the total applied lateral force at the intersection of the two line segments. The effective yield displacement, δ_y corresponds to the control point displacement at the intersection of the two line segments. The effective fundamental period, T_e of the structure in the direction under consideration shall be determined using Eq. 6.2.51 as follows:

$$T_e = T_1\sqrt{\frac{V_1/\delta_1}{V_y/\delta_y}} \tag{6.2.51}$$

Where, V_1, δ_1, and T_1 are determined for the first increment of lateral load. The target displacement of the control point, δ_T shall be determined as follows:

$$\delta_T = C_0 C_1 S_a \left(\frac{T_e}{2\pi}\right)^2 g \tag{6.2.52}$$

Where, the spectral acceleration, S_a, is determined at the effective fundamental period, T_e, using Eq. 6.2.34, g is the acceleration due to gravity. The coefficient C_o shall be calculated as:

$$C_o = \frac{\sum_{i=1}^{n} w_i \phi_i}{\sum_{i=1}^{n} w_i \phi_i^2} \tag{6.2.53}$$

Where,

w_i = the portion of the seismic weight, W, at level i, and

ϕ_i = the amplitude of the shape vector at level i.

Where the effective fundamental period, T_e, is greater than T_C (defined in Sec. 2.5.4.3), the coefficient C_1 shall be taken as 1.0. Otherwise, the value of the coefficient C_1 shall be calculated as follows:

$$C_1 = \frac{1}{R_d}\left(1 + \frac{(R_d - 1)T_s}{T_e}\right) \tag{6.2.54}$$

Where, R_d is given as follows:

$$R_d = \frac{S_a}{V_y/W} \tag{6.2.55}$$

2.5.12.4 Structure member design (NSA)

For each nonlinear static analysis the design response parameters, including the individual member forces and member deformations shall be taken as the values obtained from the analysis at the step at which the target displacement is reached.

The adequacy of individual members and their connections to withstand the member forces and member deformations shall be evaluated based on laboratory test data for similar components. The effects of gravity and other loads on member deformation capacity shall be considered in these evaluations. The deformation of a member supporting gravity loads shall not exceed (i) two-thirds of the deformation that results in loss of ability to support gravity loads, and (ii) two-thirds of the deformation at which the member strength has deteriorated to less than 70 percent of the peak strength of the component model. The deformation of a member not required for gravity load support shall not exceed two-thirds of the value at which member strength has deteriorated to less than 70 percent of the peak strength of the component model.

2.5.12.5 Design review (NSA)

Checking of the design by competent third party is recommended. An independent team composed of at least two members with experience in seismic analysis methods and the theory and application of nonlinear seismic analysis and structural behavior under earthquake loading, shall perform a review of the design of the seismic force resisting system and the supporting structural analyses. The design review shall include (i) review of any site-specific seismic criteria (if developed) employed in the analysis (ii) review of the determination of the target displacement and effective yield strength of the structure (iii) review of adequacy of structural elements and systems to withstand the calculated force and deformation demands, together with laboratory and other data (iv) review of structural design.

2.5.13 Earthquake Load Effects and Load Combinations

The seismic load effect, E, shall be determined in accordance with the following:

1. For use in load combination 5 in Section 2.7.3 or load combination 5 and 6 in Section 2.7.2, E shall be determined in accordance with the following equation,

   $E = E_h + E_v$

2. For use in load combination 7 in Section 2.7.3 or load combination 8 in Section 2.7.2, E shall be determined in accordance with following equation,

   $E = E_h - E_v$

Where,

E = total seismic load effect

E_h = effect of horizontal seismic forces as defined in Sections 2.5.7 or 2.5.9

E_v = effect of vertical seismic forces as defined in Section 2.5.13.2

2.5.13.1 Horizontal earthquake loading, E_h

The horizontal seismic load effect, E_h, shall be taken as the horizontal load effects of seismic base shear V (Sec 2.5.7 or 2.5.9) or component forces F_c (Sec 2.5.15).

The directions of application of horizontal seismic forces for design shall be those which will produce the most critical load effects. Earthquake forces act in both principal directions of the building simultaneously. In order to account for that,

(a) For structures of Seismic Design Category B, the design horizontal seismic forces are permitted to be applied independently in each of two orthogonal directions and orthogonal interaction effects are permitted to be neglected

(b) Structures of Seismic Design Category C and D shall, as a minimum, conform to the requirements of (a) for Seismic Design Category B and in addition the requirements of this Section. The structure of Seismic Design Category C with plan irregularity type V and Seismic Design Category D shall be designed for 100% of the horizontal seismic forces in one principal direction combined with 30% of the horizontal seismic forces in the orthogonal direction. Possible combinations are:

“±100% in x-direction ±30% in y-direction” or

“±30% in x-direction ±100% in y-direction”

The combination which produces most unfavourable effect for the particular action effect shall be considered. This approach may be applied to equivalent static analysis, response spectrum analysis and linear time history analysis procedure.

(c) Where three-dimensional analysis of a spatial structure model is performed as in 3D time history analysis, simultaneous application of accelerations in two directions shall be considered where the ground motions shall satisfy the conditions stated in Sections 2.5.10.2 or 2.5.11.2.

2.5.13.2 Vertical earthquake loading, E_v

The maximum vertical ground acceleration shall be taken as 50 percent of the expected horizontal peak ground acceleration (PGA). The vertical seismic load effect E_v may be determined as:

$$E_v = 0.50(a_h)D \qquad (6.2.56)$$

Where,

a_h = expected horizontal peak ground acceleration (in *g*) for design = $(2/3)ZS$

D = effect of dead load, *S* = site dependent soil factor (see Table 6.2.16).

2.5.13.3 Combination of earthquake loading with other loadings

When earthquake effect is included in the analysis and design of a building or structure, the provisions set forth in Sec 2.7 shall be followed to combine earthquake load effects, both horizontal and vertical, with other loading effects to obtain design forces etc.

2.5.13.4 Seismic Load Effect Including Overstrength Factor

Where specifically required, conditions requiring overstrength factor, Ω_o, applications shall be determined in accordance with the following,

1. For use in load combination 5 in Section 2.7.3 or load combinations 5 and 6 in Section 2.7.2, *E* shall be taken equal to E_m as determined in accordance with the following equation,

   $E_m = E_{mh} + E_v$

2. For use in load combination 7 in Section 2.7.3 or load combination 8 in Section 2.7.2, *E* shall be taken equal to E_m as determined in accordance with the following equation,

   $E_m = E_{mh} - E_v$

   where

   E_m = total seismic load effect including overstrength factor

   E_{mh} = effect of horizontal seismic forces as defined in Sections 2.5.7 or 2.5.9 including structural overstrength.

   E_v = effect of vertical seismic forces as defined in Section 2.5.13.2

The horizontal seismic load effect with overstrength factor, E_{mh}, shall be determined in accordance with the following equation:

$E_{mh} = \Omega_o E_h$

Where, Ω_o is the system overstrength factor as defined in Table 6.2.19. Like E_h, directional combinations as defined in Sec. 2.5.13.1.(b) is also applicable for calculating E_{mh}. The value of E_{mh} need not exceed the maximum force that can develop in the structure or element as determined by a rational, plastic mechanism analysis or nonlinear response analysis (static or dynamic) utilizing realistic expected values of material strengths.

2.5.13.5 Allowable Stress Increase for Load Combinations with Overstrength

Where allowable stress design methodologies are used with the seismic load effect defined in Section 2.5.13.4 applied in load combinations 5, 6, or 8 of Section 2.7.2, allowable stresses are permitted to be determined using an allowable stress increase of 1.2. This increase shall not be combined with increases in allowable stresses or load combination reductions otherwise permitted elsewhere by this standard.

2.5.13.6 Minimum Upward Force for Horizontal Cantilevers for Seismic Design Categories D through E

In structures assigned to Seismic Design Category D, or E, horizontal cantilever structural components shall be designed for a minimum net upward force of 0.2 times the dead load in addition to the applicable load combinations of Section 2.7.

2.5.14 Drift and Deformation

2.5.14.1 Storey drift limit

The design storey drift (Δ) of each storey, as determined in Sections 2.5.7, 2.5.9 or 2.5.10 shall not exceed the allowable storey drift (Δ_a) as obtained from Table 6.2.21 for any story.

For structures with significant torsional deflections, the maximum drift shall include torsional effects. For structures assigned to Seismic Design Category C or D having torsional irregularity, the design storey drift, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the storey under consideration. For seismic force-resisting systems comprised solely of moment frames in Seismic Design Categories D, the allowable storey drift for such linear elastic analysis procedures shall not exceed Δ_a/q where q is termed as a structural redundancy factor. The value of redundancy factor q may be considered as 1.0 with exception of structures of very low level of redundancy where q may be considered as 1.3.

For nonlinear time history analysis (NTHA), the storey drift obtained (Sec 2.5.11) shall not exceed 1.25 times the storey drift limit specified above for linear elastic analysis procedures.

2.5.14.2 Diaphragm deflection

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

Table 6.2.21: Allowable Storey Drift Limit (Δ_a)

Structure	Occupancy Category		
	I and II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{cs}$	$0.020h_{cs}$	$0.015h_{cs}$
Masonry cantilever shear wall structures	$0.010h_{cs}$	$0.010h_{cs}$	$0.010h_{cs}$
Other masonry shear wall structures	$0.007h_{cs}$	$0.007h_{cs}$	$0.007h_{cs}$
All other structures	$0.020h_{cs}$	$0.015h_{cs}$	$0.010h_{cs}$

Notes:

1. h_{cs} is the story height below Level x.
2. There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the storey drifts.
3. Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.
4. Occupancy categories are defined in Table 6.1.1

2.5.14.3 Separation between adjacent structures

Buildings shall be protected from earthquake-induced pounding from adjacent structures or between structurally independent units of the same building maintaining safe distance between such structures as follows:

(i) for buildings, or structurally independent units, that do not belong to the same property, the distance from the property line to the potential points of impact shall not be less than the computed maximum horizontal displacement (Sec 2.5.7.7) of the building at the corresponding level.

(ii) for buildings, or structurally independent units, belonging to the same property, if the distance between them is not less than the square root of the sum- of the squares (SRSS) of the computed maximum horizontal displacements (Sec 2.5.7.7) of the two buildings or units at the corresponding level.

(iii) if the floor elevations of the building or independent unit under design are the same as those of the adjacent building or unit, the above referred minimum distance may be reduced by a factor of 0.7

2.5.14.4 Special deformation requirement for seismic design category D

For structures assigned to Seismic Design Category D, every structural component not included in the seismic force-resisting system in the direction under consideration shall be designed to be adequate for the gravity load effects and the seismic forces resulting from displacement to the design story drift (Δ) as determined in accordance with Sec 2.5.7.7. Even where elements of the structure are not intended to resist seismic forces, their protection may be important. Where determining the moments and shears induced in components that are not included in the seismic force-resisting system in the direction under consideration, the stiffening effects of adjoining rigid structural and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used.

2.5.15 Seismic Design For Nonstructural Components

This Section establishes minimum design criteria for nonstructural components that are permanently attached to structures and for their supports and attachments. The following components are exempt from the requirements of this Section.

(1) Architectural components in Seismic Design Category B, other than parapets supported by bearing walls or shear walls, where the component importance factor, I_c is equal to 1.0.

(2) Mechanical and electrical components in Seismic Design Category B.

(3) Mechanical and electrical components in Seismic Design Category C where the importance factor, I_c is equal to 1.0.

(4) Mechanical and electrical components in Seismic Design Category D where the component importance factor, I_c is equal to 1.0 and either (a) flexible connections between the components and associated ductwork, piping, and conduit are provided, or (b) components are mounted at 1.2 m or less above a floor level and weigh 1780 N or less.

(5) Mechanical and electrical components in Seismic Design Category C or D where the component importance factor, I_c is equal to 1.0 and (a) flexible connections between the components and associated ductwork, piping, and conduit are provided, and (b) the components weigh 89 N or less or, for distribution systems, which weigh 73 N/m or less.

Where the individual weight of supported components and non-building structures with periods greater than 0.06 seconds exceeds 25 percent of the total seismic weight W, the structure shall be designed considering interaction effects between the structure and the supported components.

Testing shall be permitted to be used in lieu of analysis methods outlined in this Chapter to determine the seismic capacity of components and their supports and attachments.

2.5.15.1 Component importance factor

All components shall be assigned a component importance factor. The component importance factor, I_c shall be taken as 1.5 if any of the following conditions apply:

(1) The component is required to function after an earthquake,

(2) The component contains hazardous materials, or

(3) The component is in or attached to a occupancy category IV building and it is needed for continued operation of the facility.

All other components shall be assigned a component importance factor, I_c equal to 1.0.

2.5.15.2 Component force transfer

Components shall be attached such that the component forces are transferred to the structure. Component attachments that are intended to resist seismic forces shall be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effects of gravity. A continuous load path of sufficient strength and stiffness between the component and the supporting structure shall be verified. Local elements of the supporting structure shall be designed for the component forces where such forces control the design of the elements or their connections. In this instance, the component forces shall be those determined in Sec 2.5.15.3, except that modifications to F_p and R_p due to anchorage conditions need not be considered. The design documents shall include sufficient information concerning the attachments to verify compliance with the requirements of these Provisions.

2.5.15.3 Seismic design force

The seismic design force, F_c, applied in the horizontal direction shall be centered at the component's center of gravity and distributed relative to the component's mass distribution and shall be determined as follows:

$$F_c = \frac{\alpha_c a_h W_c I_c}{R_c}\left(1 + 2\frac{z}{h}\right) \qquad (6.2.57)$$

Where,

$0.75 a_h W_c I_c \leq F_c \leq 1.5 a_h W_c I_c$

α_c = component amplification factor which varies from 1.0 to 2.5 (Table 6.2.22 or Table 6.2.23).

a_h = expected horizontal peak ground acceleration (in g) for design = 0.67ZS

W_c = weight of component

R_c = component response reduction factor which varies from 1.0 to 12.0 (Table 6.2.22 or Table 6.2.23)

z = height above the base of the point of attachment of the component, but z shall not be taken less than 0 and the value of z/h need not exceed 1.0

h = roof height of structure above the base

The force F_c shall be independently applied in at least two orthogonal horizontal directions in combination with service loads associated with the component. In addition, the component shall also be designed for a concurrent vertical force of $\pm 0.5 a_h W_c$.

Where non-seismic loads on nonstructural components exceed F_c such loads shall govern the strength design, but the seismic detailing requirements and limitations shall apply.

2.5.15.4 Seismic relative displacements

The relative seismic displacement, D_c for two connection points on the same structure A, one at a height h_s and other at height h_y, for use in component design shall be determined as follows:

$$D_c = \delta_{xA} - \delta_{yA} \qquad (6.2.58)$$

D_c shall not exceed $D_{c\ mas}$ given by:

$$D_{c\,\max} = \frac{\left(h_x - h_y\right)\Delta_{aA}}{h_{sx}} \qquad (6.2.59)$$

Where,

$\delta_{sÆ}$ = Deflection at level x of structure A

$\delta_{yÆ}$ = Deflection at level y of structure A

$\Delta_{aÆ}$ = Allowable story drift for structure A

h_x = Height (above base) of level x to which upper connection point is attached.

h_y = Height (above base) of level y to which lower connection point is attached.

h_{sx} = Story height used in the definition of the allowable drift Δ_a

For two connection points on separate structures, A and B, or separate structural systems, one at level x and the other at level y, D_c shall be determined as follows:

$$D_c = |\delta_{xA}| + |\delta_{yB}| \tag{6.2.60}$$

D_c shall not exceed $D_{c\,max}$ given by:

$$D_{c\,max} = \frac{h_x \Delta_{aA}}{h_{sx}} + \frac{h_y \Delta_{aB}}{h_{sx}} \tag{6.2.61}$$

Where,

δ_{yB} = Deflection at level y of structure B

Δ_{aB} = Allowable story drift for structure B

The effects of relative seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate.

2.5.16 Design For Seismically Isolated Buildings

Buildings that use special seismic isolation systems for protection against earthquakes shall be called seismically isolated or base isolated buildings. Seismically isolated structure and every portion thereof shall be designed and constructed in accordance with the requirements of provisions presented in this Section.

2.5.16.1 General requirements for isolation system

The isolation system to be used in seismically isolated structures shall satisfy the following requirements:

(1) Design of isolation system shall consider variations in seismic isolator material properties over the projected life of structure including changes due to ageing, contamination, exposure to moisture, loadings, temperature, creep, fatigue, etc.

(2) Isolated structures shall resist design wind loads at all levels above the isolation interface. At the isolation interface, a wind restraint system shall be provided to limit lateral displacement in the isolation system to a value equal to that required between floors of the structure above the isolation interface.

(3) The fire resistance rating for the isolation system shall be consistent with the requirements of columns, walls, or other such elements in the same area of the structure.

(4) The isolation system shall be configured to produce a lateral restoring force such that the lateral force at the total design displacement is at least 0.025 W greater than the lateral force at 50% of the total design displacement.

(5) The isolation system shall not be configured to include a displacement restraint that limits lateral displacement due to the maximum considered earthquake to less than the total maximum displacement unless it is demonstrated by analysis that such engagement of restraint does not result in unsatisfactory performance of thestructure.

(6) Each element of the isolation system shall be designed to be stable under the design vertical load when subjected to a horizontal displacement equal to the total maximum displacement.

(7) The factor of safety against global structural overturning at the isolation interface shall not be less than 1.0 for required load combinations. All gravity and seismic loading conditions shall be investigated. Seismic forces for overturning calculations shall be based on the maximum considered earthquake and the vertical restoring force shall be based on the seismic weight above the isolation interface.

(8) Local uplift of individual units of isolation system is permitted if the resulting deflections do not cause overstress or instability of the isolator units or other elements of the structure.

(9) Access for inspection and replacement of all components of the isolation system shall be provided.

(10) The designer of the isolation system shall establish a quality control testing program for isolator units. Each isolator unit before installation shall be tested under specified vertical and horizontal loads.

(11) After completion of construction, a design professional shall complete a final series of inspections or observations of structure separation areas and components that cross the isolation interface. Such

inspections and observations shall confirm that existing conditions allow free and unhindered displacement of the structure to maximum design levels and that all components that cross the isolation interface as installed are able to accommodate the stipulated displacements.

(12) The designer of the isolation system shall establish a periodic monitoring, inspection, and maintenance program for such system.

(13) Remodeling, repair, or retrofitting at the isolation interface, including that of components that cross the isolation interface, shall be performed under the direction of a design professional experienced in seismic isolation systems.

Table 6.2.22: Coefficients a_c and R_c for Architectural Components

Architectural Component or Element	a_c[a]	R_c
Interior Nonstructural Walls and Partitions		
Plain (unreinforced) masonry walls	1.0	1.5
All other walls and partitions	1.0	2.5
Cantilever Elements (Unbraced or braced to structural frame below its center of mass) Parapets and cantilever interior nonstructural walls	2.5	2.5
Chimneys and stacks where laterally braced or supported by the structural frame	2.5	2.5
Cantilever Elements (Braced to structural frame above its center of mass) Parapets	1.0	2.5
Chimneys and Stacks	1.0	2.5
Exterior Nonstructural Walls	1.0	2.5
Exterior Nonstructural Wall Elements and Connections		
Wall Element	1.0	2.5
Body of wall panel connections	1.0	2.5
Fasteners of the connecting system	1.25	1.0
Veneer		
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
Penthouses (except where framed by an extension of the building frame)	2.5	3.5
Ceilings		
All	1.0	2.5
Cabinets		
Storage cabinets and laboratory equipment	1.0	2.5
Access Floors		
Special access floors	1.0	2.5
All other	1.0	1.5
Appendages and Ornamentations	2.5	2.5
Signs and Billboards	2.5	2.5
Other Rigid Components		
High deformability elements and attachments	1.0	3.5

Architectural Componentor Element	a_c[a]	R_c
Limited deformability elements and attachments	1.0	2.5
Low deformability materials and attachments	1.0	1.5
Other Flexible Components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability materials and attachments	2.5	1.5

[a] A lower value for α_c is permitted where justified by detailed dynamic analysis. The value for α_c shall not be less than 1.0. The value of α_c equal to 1.0 is for rigid components and rigidly attached components. The value of α_c equal to 2.5 is for flexible components and flexibly attached components.

Table 6.2.23: Coefficients a_c and R_c for Mechanical and Electrical Components

Mechanical and Electrical Components	a_c[a]	R_c
Air-side HVAC, fans, air handlers, air conditioning units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal framing.	2.5	6.0
Wet-side HVAC, boilers, furnaces, atmospheric tanks and bins, chillers, water heaters, heat exchangers, evaporators, air separators, manufacturing or process equipment, and other mechanical components constructed of high-deformability materials.	1.0	2.5
Engines, turbines, pumps, compressors, and pressure vessels not supported on skirts and not within the scope of Chapter 15.	1.0	2.5
Skirt-supported pressure vessels	2.5	2.5
Elevator and escalator components.	1.0	2.5
Generators, batteries, inverters, motors, transformers, and other electrical components constructed of high deformability materials.	1.0	2.5
Motor control centers, panel boards, switch gear, instrumentation cabinets, and other components constructed of sheet metal framing.	2.5	6.0
Communication equipment, computers, instrumentation, and controls.	1.0	2.5
Roof-mounted chimneys, stacks, cooling and electrical towers laterally braced below their center of mass.	2.5	3.0
Roof-mounted chimneys, stacks, cooling and electrical towers laterally braced above their center of mass.	1.0	2.5
Lighting fixtures.	1.0	1.5
Other mechanical or electrical components.	1.0	1.5
Vibration Isolated Components and Systems[b]		
Components and systems isolated using neoprene elements and neoprene isolated floors with built-in or separate elastomeric snubbing devices or resilient perimeter stops.	2.5	2.5
Spring isolated components and systems and vibration isolated floors closely restrained using built-in or separate elastomeric snubbing devices or resilient perimeter stops.	2.5	2.0
Internally isolated components and systems.	2.5	2.0
Suspended vibration isolated equipment including in-line duct devices and suspended internally isolated components.	2.5	2.5

Mechanical and Electrical Components	a_c[a]	R_c
Air-side HVAC, fans, air handlers, air conditioning units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal framing.	2.5	6.0
Wet-side HVAC, boilers, furnaces, atmospheric tanks and bins, chillers, water heaters, heat exchangers, evaporators, air separators, manufacturing or process equipment, and other mechanical components constructed of high-deformability materials.	1.0	2.5
Engines, turbines, pumps, compressors, and pressure vessels not supported on skirts and not within the scope of Chapter 15.	1.0	2.5
Skirt-supported pressure vessels	2.5	2.5
Distribution Systems		
Piping in accordance with ASME B31, including in-line components with joints made by welding or brazing.	2.5	12.0
Piping in accordance with ASME B31, including in-line components, constructed of high or limited deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings.	2.5	6.0
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing.	2.5	9.0
Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high- or limited-deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings.	2.5	4.5
Piping and tubing constructed of low-deformability materials, such as cast iron, glass, and non-ductile plastics.	2.5	3.0
Ductwork, including in-line components, constructed of high-deformability materials, with joints made by welding or brazing.	2.5	9.0
Ductwork, including in-line components, constructed of high- or limited-deformability materials with joints made by means other than welding or brazing.	2.5	6.0
Ductwork, including in-line components, constructed of low-deformability materials, such as cast iron, glass, and non-ductile plastics.	2.5	3.0
Electrical conduit, bus ducts, rigidly mounted cable trays, and plumbing.	1.0	2.5
Manufacturing or process conveyors (non-personnel).	2.5	3.0
Suspended cable trays.	2.5	6.0

[a] A lower value for α_c is permitted where justified by detailed dynamic analysis. The value for α_c shall not be less than 1.0. The value of α_c equal to 1.0 is for rigid components and rigidly attached components. The value of α_c equal to 2.5 is for flexible components and flexibly attached components.

[b] Components mounted on vibration isolators shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_c$ if the nominal clearance (air gap) between the equipment support frame and restraint is greater than 6 mm. If the nominal clearance specified on the construction documents is not greater than 6 mm, the design force may be taken as F_c.

2.5.16.2 Equivalent static analysis

The equivalent static analysis procedure is permitted to be used for design of a seismically isolated structure provided that:

(1) The structure is located on Site Class SA, SB, SC, SD or SE site;

(2) The structure above the isolation interface is not more than four stories or 20 m in height

(3) Effective period of the isolated structure at the maximum displacement, T_M, is less than or equal to 3.0 sec.

(4) The effective period of the isolated structure at the design displacement, T_D, is greater than three times the elastic, fixed-base period of the structure above the isolation system as determined in Sec. 2.5.7.2

(5) The structure above the isolation system is of regular configuration; and

(6) The isolation system meets all of the following criteria:

(a) The effective stiffness of the isolation system at the design displacement is greater than one third of the effective stiffness at 20 percent of the design displacement,

(b) The isolation system is capable of producing a restoring force as specified in Sec. 2.5.16.1,

(c) The isolation system does not limit maximum considered earthquake displacement to less than the total maximum displacement.

Where the equivalent lateral force procedure is used to design seismically isolated structures, the requirements of this Section shall apply.

2.5.16.2.1 Displacement of isolation system: The isolation system shall be designed and constructed to withstand minimum lateral earthquake displacements that act in the direction of each of the main horizontal axes of the structure and such displacements shall be calculated as follows:

$$D_D = \frac{S_a g}{4\pi^2}\left(\frac{T_D^2}{B_D}\right) \qquad (6.2.62)$$

Where,

S_a = Design spectral acceleration (in units of g), calculated using Eq. 6.2.34 for period T_D and assuming $R=1$, $I=1$, $\eta=1$ (Sec 2.5.4.3) for the design basis earthquake (DBE).

g = acceleration due to gravity

B_D= damping coefficient related to the effective damping β_D of the isolation system at the design displacement, as set forth in Table 6.2.24.

T_D= effective period of seismically isolated structure at the design displacement in the direction under consideration, as prescribed by Eq. 6.2.63:

$$T_D = 2\pi\sqrt{\frac{W}{k_{D\min}\, g}} \qquad (6.2.63)$$

Where,

W = seismic weight above the isolation interface

k_{Dmin} = minimum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

Table 6.2.24: Damping Coefficient, B_D or B_M

Effective Damping, β_D or β_M [a, b] (%)	B_D or B_M
≤ 2	0.8
5	1.0
10	1.2
20	1.5
30	1.7
40	1.9
≥ 50	2.0

[a] The damping coefficient shall be based on the effective damping of the isolation system

[b] The damping coefficient shall be based on linear interpolation for effective damping values other than those given.

The maximum displacement of the isolation system, D_M, in the most critical direction of horizontal response shall be calculated in accordance with the following formula:

$$D_M = \frac{S_{aM} g}{4\pi^2}\left(\frac{T_M^2}{B_M}\right) \qquad (6.2.64)$$

Where:

S_{aM} = Maximum spectral acceleration (in units of g), calculated using Eq. 6.2.34 for period T_D and assuming $R=1$, $I=1$, $\eta=1$ (Sec 2.5.4.3) for the maximum considered earthquake (MCE).

B_M = numerical coefficient related to the effective damping β_M of the isolation system at the maximum displacement, as set forth in Table 6.2.24.

T_M = effective period of seismic-isolated structure at the maximum displacement in the direction under consideration as prescribed by:

$$T_M = 2\pi\sqrt{\frac{W}{k_{M\,\min}\, g}} \qquad (6.2.65)$$

Where,

$k_{M\,\min}$ = minimum effective stiffness of the isolation system at the maximum displacement in the horizontal direction under consideration.

The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, of elements of the isolation system shall include additional displacement due to inherent and accidental torsion calculated considering the spatial distribution of the lateral stiffness of the isolation system and the most disadvantageous location of eccentric mass.

2.5.16.2.2 Lateral seismic forces: The structure above the isolation system shall be designed and constructed to withstand a minimum lateral force, V_s, using all of the appropriate provisions for a non-isolated structure. The importance factor for all isolated structures shall be considered as 1.0, also the response reduction factor R_I considered here (for computing design seismic forces) is in the range of 1.0 to 2.0. V_s shall be determined in accordance with Eq. 6.2.66 as follows:

$$V_s = \frac{k_{D\,\max} D_D}{R_I} \qquad (6.2.66)$$

Where,

$k_{D\ mas}$ = maximum effective stiffness of the isolation system at the design displacement in the horizontal direction under consideration.

D_D = design displacement at the center of rigidity of the isolation system in the direction under consideration as prescribed by Eq. 6.2.62.

R_I = response reduction factor related to the type of seismic-force-resisting system above the isolation system. R_I shall be based on the type of seismic-force-resisting system used for the structure above the isolation system and shall be taken as the lesser of $\frac{3}{8}R$ (Table 6.2.19) or 2.0, but need not be taken less than 1.0.

In no case shall V_s be taken less than the following:

(1) The lateral force required by Sec 2.5.7 for a fixed-base structure of the same weight, W, and a period equal to the isolated period, T_D;

(2) The base shear corresponding to the factored design wind load; and

(3) The lateral force required to fully activate the isolation system (e.g., the yield level of a softening system, the ultimate capacity of a sacrificial wind-restraint system, or the break-away friction level of a sliding system) multiplied by 1.5.

The isolation system, the foundation, and all structural elements below the isolation system shall be designed and constructed to withstand a minimum lateral force, V_b using all of the appropriate provisions for a non-isolated structure. V_b shall be determined in accordance with Eq. 6.2.67 as follows:

$$V_b = k_{Dmas} D_D \tag{6.2.67}$$

In all cases, V_b shall not be taken less than the maximum force in the isolation system at any displacement up to and including the design displacement.

2.5.16.2.3 Vertical distribution of lateral forces: The total lateral force shall be distributed over the height of the structure above the isolation interface in accordance with Eq. 6.2.68 as follows:

$$F_x = V_s \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i} \tag{6.2.68}$$

Where:

V_c = Total seismic lateral design force on elements above the isolation system.

h_i, h_s = Height above the base, to Level i or Level x, respectively.

w_i, w_s = Portion of W that is located at or assigned to Level i or Level x, respectively.

At each Level x the force, F_s shall be applied over the area of the structure in accordance with the distribution of mass at the level. Stresses in each structural element shall be determined by applying to an analytical model the lateral forces, F_s at all levels above the base.

2.5.16.2.4 Storey drift: The storey drift shall be calculated as in Sec 2.5.7.7 except that C_d for the isolated structure shall be taken equal to R_I and importance factor equal to 1.0. The maximum storey drift of the structure above the isolation system shall not exceed $0.015h_{sx}$.

2.5.16.3 Dynamic analysis

Response spectrum analysis may be conducted if the behavior of the isolation system can be considered as equivalent linear. Otherwise, non-linear time history analysis shall be used where the true non-linear behaviour of the isolation system can be modeled. The mathematical models of the isolated structure including the isolation system shall be along guidelines given in Sections 2.5.9.1 and 2.5.11.1, and other requirements given in Sec 2.5.16.

The isolation system shall be modeled using deformational characteristics developed and verified by testing. The structure model shall account for: (i) spatial distribution of isolator units; (ii) consideration of translation in both horizontal directions, and torsion of the structure above the isolation interface considering the most disadvantageous location of eccentric mass; (iii) overturning/uplift forces on individual isolator units; and (iv) effects of vertical load, bilateral load, and the rate of loading if the force-deflection properties of the isolation system are dependent on such attributes.

A linear elastic model of the isolated structure (above isolation system) may be used provided that: (i) stiffness properties assumed for the nonlinear components of the isolation system are based on the maximum effective stiffness of the isolation system, and (ii) all elements of the seismic-force-resisting system of the structure above the isolation system behave linearly.

2.5.16.3.1 Response Spectrum Analysis: Response spectrum analysis shall be performed using a modal damping value for the fundamental mode in the direction of interest not greater than the effective damping of the isolation system or 30 percent of critical, whichever is less. Modal damping values for higher modes shall be selected consistent with those that would be appropriate for response spectrum analysis of the structure above the isolation system assuming a fixed base.

Response spectrum analysis used to determine the total design displacement and the total maximum displacement shall include simultaneous excitation of the model by 100 percent of the ground motion in the critical direction and 30 percent of the ground motion in the perpendicular, horizontal direction. The design basis earthquake shall be used for the design displacement, while the maximum considered earthquake shall be used for the maximum displacement. The maximum displacement of the isolation system shall be calculated as the vectorial sum of the two orthogonal displacements.

For the design displacement, structures that do not require site-specific ground motion evaluation, shall be analyzed using the design acceleration response spectrum in accordance with Sec 2.5.4.3. The maximum design spectrum to be used for the maximum considered earthquake shall not be less than 1.5 times the design acceleration response spectrum.

The response spectrum procedure is based on an equivalent linear model, where the effective stiffness and effective damping is a function of the displacement, this formulation is thus an iterative process. The effective stiffness must be estimated, based on assumed displacement, and then adjusted till obtained displacement agree with assumed displacement.

The design shear at any story shall not be less than the story shear resulting from application of the story forces calculated using Eq. 6.2.68 with a value of V_c equal to the base shear obtained from the response spectrum analysis in the direction of interest.

2.5.16.3.2 Nonlinear Time History Analysis: Where a time history analysis procedure is performed, not fewer than three appropriate ground motions shall be used in the analysis as described below.

Ground motions shall consist of pairs of appropriate horizontal ground motion acceleration components that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. If required number of recorded ground motion pairs are not available, appropriate simulated ground motion pairs shall be used to make up the total number required. For each pair of horizontal ground-motion components, a square root of the sum of the squares (SRSS) spectrum shall be constructed by taking the SRSS of the 5 percent damped response spectra for the scaled components (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that for each period between $0.5T_D$ and $1.25T_M$ (where T_D and T_M are defined in Sec 2.5.16.2.1) the average of the SRSS spectra from all horizontal component pairs does not fall below 1.3 times the corresponding ordinate of the design response spectrum (Sec 2.5.16.4), by more than 10 percent.

Each pair of ground motion components shall be applied simultaneously to the model considering the most disadvantageous location of eccentric mass. The maximum displacement of the isolation system shall be calculated from the vectorial sum of the two orthogonal displacements at each time step.

The parameters of interest shall be calculated for each ground motion used for the time history analysis. If at least seven ground motions are used for the time history analysis, the average value of the response parameter of interest is permitted to be used for design. If fewer than seven ground motions are analyzed, the maximum value of the response parameter of interest shall be used for design.

2.5.16.3.3 Storey drift: Maximum story drift corresponding to the design lateral force including displacement due to vertical deformation of the isolation system shall not exceed the following limits:

1. The maximum story drift of the structure above the isolation system calculated by response spectrum analysis shall not exceed $0.015h_{cs}$.
2. The maximum story drift of the structure above the isolation system calculated by nonlinear time history analysis shall not exceed $0.020h_{cs}$.

The storey drift shall be calculated as in Sec 2.5.7.7 except that C_d for the isolated structure shall be taken equal to R_I and importance factor equal to 1.0.

2.5.16.4 Testing

The deformation characteristics and damping values of the isolation system used in the design and analysis of seismically isolated structures shall be based on test results of isolator units. The tests are for establishing and validating the design properties of the isolation system and shall not be considered as satisfying the manufacturing quality control tests.

The following sequence of tests shall be performed on isolator units for the prescribed number of cycles at a vertical load equal to the average dead load plus one-half the effects due to live load on all isolator units of a common type and size:

(1) Twenty fully reversed cycles of loading at a lateral force corresponding to the wind design force.

(2) Three fully reversed cycles of loading at each of the following increments of the total design displacement-0.25D_D, 0.5D_D, 1.0D_D, and 1.0D_M where D_D and D_M are as determined in Sec 2.5.16.2.1.

(3) Three fully reversed cycles of loading at the total maximum displacement, 1.0D_{TM}.

(4) Not less than ten fully reversed cycles of loading at 1.0 times the total design displacement, 1.0D_{TD}.

For each cycle of each test, the force-deflection and hysteretic behavior of each isolator unit shall be recorded. The effective stiffness is obtained as the secant value of stiffness at design displacement while the effective damping is determined from the area of hysteretic loop at the design displacement.

2.5.16.5 Design review

A design review of the isolation system and related test programs shall be performed by an independent team of design professionals experienced in seismic analysis methods and the application of seismic isolation. Isolation system design review shall include, but need not be limited to, the following:

(1) Review of site-specific seismic criteria including the development of site-specific spectra and ground motion time histories and all other design criteria developed specifically for the project;

(2) Review of the preliminary design including the determination of the total design displacement of the isolation system and the lateral force design level;

(3) Overview and observation of prototype (isolator unit) testing

(4) Review of the final design of the entire structural system and all supporting analyses; and

(5) Review of the isolation system quality control testing program.

2.5.17 Buildings with Soft Storey

Buildings with possible soft storey action at ground level for providing open parking spaces belong to structures with major vertical irregularity [Figure 6.2.28(a)]. Special arrangement is needed to increase the lateral strength and stiffness of the soft/open storey. The following two approaches may be considered:

(1) Dynamic analysis of such building may be carried out incorporating the strength and stiffness of infill walls and inelastic deformations in the members, particularly those in the soft storey, and the members designed accordingly.

(2) Alternatively, when system overstrength factor, Ω_o, is not included in determining seismic load effects, the following design criteria are to be adopted after carrying out the earthquake analysis, neglecting the effect of infill walls in other storeys. Structural elements (e.g columns and beams) of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads neglecting effect of infill walls. Shear walls placed symmetrically in both directions of the building as far away from the centre of the building as feasible are to be designed exclusively for 1.5 times the lateral shear force calculated before.

2.5.18 Non-Building Structures

Calculation of seismic design forces on non-building structures (e.g. chimney, self-supported overhead water/fluid tank, silo, trussed tower, storage tank, cooling tower, monument and other structures not covered in Sec 2.5) shall be in accordance with "Chapter 15: Seismic Design Requirements for Non-Building Structures, Minimum Design Loads for Buildings and Other Structures, ASCE Standard ASCE/SEI 7-05" complying with the requirements of Sec 2.5 of this Code.