T_{upper} = the larger of the two orthogonal fundamental periods of vibration (Section 12.9.2). The mathematical model used to compute T_{upper} shall not include accidental torsion and shall include P-delta effects
V = total design lateral force or shear at the base
V_{EX} = maximum absolute value of elastic base shear computed in the X direction among all three analyses performed in that direction (Section 12.9.2.5)
V_{EY} = maximum absolute value of elastic base shear computed in the Y direction among all three analyses performed in that direction (Section 12.9.2.5)
V_{IX} = inelastic base shear in the X direction (Section 12.9.2.5)
V_{IY} = inelastic base shear in the Y direction (Section 12.9.2.5)
V_t = design value of the seismic base shear as determined in Section 12.9.1.4.1
V_X = ELF base shear in the X direction (Section 12.9.2.5)
V_x = seismic design shear in story x as determined in Section 12.8.4
V_Y = ELF base shear in the Y direction (Section 12.9.2.5)
$\tilde{V}$ = reduced base shear accounting for the effects of soil structure interaction as determined in Section 19.3.1
$\tilde{V}_1$ = portion of the reduced base shear, $\tilde{V}_1$ contributed by the fundamental mode, Section 19.3, in kip (kN)
ΔV = reduction in V as determined in Section 19.3.1, in kip (kN)
ΔV_1 = reduction in V_1 as determined in Section 19.3.1, in kip (kN)
v_s = shear wave velocity at small shear strains (greater than $10^{-3}\%$ strain); see Section 19.2.1, in ft/s (m/s)
$\bar{v}_s$ = average shear wave velocity at small shear strains in top 100 ft (30 m); see Sections 20.3.3 and 20.4.1
v_{si} = the shear wave velocity of any soil or rock layer i (between 0 and 100 ft (between 0 and 30 m)); see Section 20.4.1
v_{so} = average shear wave velocity for the soils beneath the foundation at small strain levels, Section 19.2.1.1 in ft/s (m/s)
W = effective seismic weight of the building as defined in Section 12.7.2. For calculation of seismic-isolated building period, W is the total effective seismic weight of the building as defined in Sections 19.2 and 19.3, in kip (kN)
W = effective seismic weight of the building as defined in Sections 19.2 and 19.3, in kip (kN)
W_c = gravity load of a component of the building
W_P = component operating weight, in lb (N)
w_{px} = weight tributary to the diaphragm at level x
w = moisture content (in percent), ASTM D2216
w_i, w_n, w_x = portion of W that is located at or assigned to level i, n, or x, respectively
x = level under consideration, 1 designates the first level above the base
z = height in structure of point of attachment of component with respect to the base; see Section 13.3.1
z_s = mode shape factor, Section 12.10.3.2.1
β = ratio of shear demand to shear capacity for the story between levels x and $x-1$
$\bar{\beta}$ = fraction of critical damping for the coupled structure–foundation system, determined in Section 19.2.1
β_0 = foundation damping factor as specified in Section 19.2.1.2
Γ_{m1}, Γ_{m2} = first and higher modal contribution factors, respectively, Section 12.10.3.2.1
γ = average unit weight of soil, in lb/ft^3 (N/m^3)
Δ = design story drift as determined in Section 12.8.6
$\Delta_{fallout}$ = the relative seismic displacement (drift) at which glass fallout from the curtain wall, storefront, or partition occurs
Δ_a = allowable story drift as specified in Section 12.12.1
Δ_{ADVE} = average drift of adjoining vertical elements of the seismic force-resisting system over the story below the diaphragm under consideration, under tributary lateral load equivalent to that used in the computation of δ_{MDD} Fig. 12.3-1, in in. (mm)
δ_{MDD} = computed maximum in-plane deflection of the diaphragm under lateral load, Fig. 12.3-1, in in. (mm)
δ_{max} = maximum displacement at level x, considering torsion, Section 12.8.4.3
δ_M = maximum inelastic response displacement, considering torsion, Section 12.12.3
δ_{MT} = total separation distance between adjacent structures on the same property, Section 12.12.3
δ_{avg} = the average of the displacements at the extreme points of the structure at level x, Section 12.8.4.3
δ_x = deflection of level x at the center of the mass at and above level x, Eq. (12.8-15)
δ_{xc} = deflection of level x at the center of the mass at and above level x determined by an elastic analysis, Section 12.8.6
δ_{xm} = modal deflection of level x at the center of the mass at and above level x as determined by Section 19.3.2
$\bar{\delta}_x, \bar{\delta}_{x1}$ = deflection of level x at the center of the mass at and above level x, Eqs. (19.2-13) and (19.3-3), in in. (mm)
θ = stability coefficient for P-delta effects as determined in Section 12.8.7
ηx = Force scale factor in the X direction (12.9.2.5)
ηy = Force scale factor in the Y direction (12.9.2.5)
ρ = a redundancy factor based on the extent of structural redundancy present in a building as defined in Section 12.3.4
ρ_s = spiral reinforcement ratio for precast, prestressed piles in Section 14.2.3.2.6
λ = time effect factor
Ω_0 = overstrength factor as defined in Tables 12.2-1, 15.4.-1, and 15.4-2
Ω_v = Diaphragm shear overstrength factor (Section 14.2.4.1.3)

11.4 SEISMIC GROUND MOTION VALUES

11.4.1 Near-Fault Sites. Sites satisfying either of the following conditions shall be classified as near fault:

1. 9.5 miles (15 km) of the surface projection of a known active fault capable of producing $M_w 7$ or larger events, or
2. 6.25 miles (10 km) of the surface projection of a known active fault capable of producing $M_w 6$ or larger events.

EXCEPTIONS:

1. Faults with estimated slip rate along the fault less than 0.04 in. (1 mm) per year shall not be considered.
2. The surface projection shall not include portions of the fault at depths of 6.25 mi (10 km) or greater.

11.4.2 Mapped Acceleration Parameters. The parameters S_S and S_1 shall be determined from the 0.2- and 1-s spectral response accelerations shown in Figs. 22-1, 22-3, 22-5, 22-6, 22-7, and 22-8 for S_S and Figs. 22-2, 22-4, 22-5, 22-6, 22-7, and 22-8 for S_1. Where S_1 is less than or equal to 0.04 and S_S is less than or equal to

We refer to hazard maps for spectral accelerations
Seismic event: maximum considered earthquake

a site in Nablus,
Ss=0.50g, S1=0.20g

Site class: we get from geotechnical report
We refer to Table 20.3-1, look for the values of shear wave velocities

The site in Nablus has C site class

0.15, the structure is permitted to be assigned to Seismic Design Category A and is only required to comply with Section 11.7.

> **User Note:** Electronic values of mapped acceleration parameters and other seismic design parameters are provided at the U.S. Geological Survey (USGS) website at https://doi.org/10.5066/F7NK3C76.

11.4.3 Site Class. Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E, or F in accordance with Chapter 20. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D, subject to the requirements of Section 11.4.4, shall be used unless the authority having jurisdiction or geotechnical data determine that Site Class E or F soils are present at the site.

For situations in which site investigations, performed in accordance with Chapter 20, reveal rock conditions consistent with Site Class B, but site-specific velocity measurements are not made, the site coefficients F_a, F_v, and F_{PGA} shall be taken as unity (1.0).

11.4.4 Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters. The MCE_R spectral response acceleration parameters for short periods (S_{MS}) and at 1 s (S_{M1}), adjusted for site class effects, shall be determined by Eqs. (11.4-1) and (11.4-2), respectively.

$$S_{MS} = F_a S_S$$

$$S_{M1} = F_v S_1$$

=1.3*0.5=0.65
=1.5*0.20=0.30

where

S_S = the mapped MCE_R spectral response acceleration parameter at short periods as determined in accordance with Section 11.4.2, and

S_1 = the mapped MCE_R spectral response acceleration parameter at a period of 1 s as determined in accordance with Section 11.4.2

where site coefficients F_a and F_v are defined in Tables 11.4-1 and 11.4-2, respectively. Where Site Class D is selected as the default site class per Section 11.4.3, the value of F_a shall not be less than 1.2. Where the simplified design procedure of Section 12.14 is used, the value of F_a shall be determined in accordance with Section 12.14.8.1, and the values for F_v, S_{MS}, and S_{M1} need not be determined.

Table 11.4-1 Short-Period Site Coefficient, F_a

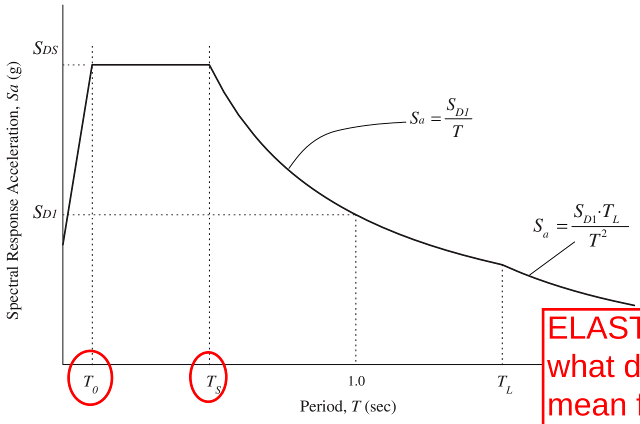
Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at Short Period $S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S = 1.25$	$S_S \ge 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_s.

Table 11.4-2 Long-Period Site Coefficient, F_v

Site Class	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period $S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \ge 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2[a]	2.0[a]	1.9[a]	1.8[a]	1.7[a]
E	4.2	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of S_1.
[a]Also, see requirements for site-specific ground motions in Section 11.4.8.



ELASTIC
what does this mean for us?

FIGURE 11.4-1 Design Response Spectrum

11.4.5 Design Spectral Acceleration Parameters. Design earthquake spectral response acceleration parameters at short periods, S_{DS}, and at 1-s periods, S_{D1}, shall be determined from Eqs. (11.4-3) and (11.4-4), respectively. Where the alternate simplified design procedure of Section 12.14 is used, the value of S_{DS} shall be determined in accordance with Section 12.14.8.1, and the value for S_{D1} need not be determined.

$$S_{DS} = \frac{2}{3} S_{MS}$$

$$S_{D1} = \frac{2}{3} S_{M1}$$

=2/3*0.65=0.43
=2/3*0.30=0.20

11.4.6 Design Response Spectrum. Where a design response spectrum is required by this standard and site-specific ground motion procedures are not used, the design response spectrum curve shall be developed as indicated in Fig. 11.4-1 and as follows:

1. For periods less than T_0, the design spectral response acceleration, S_a, shall be taken as given in Eq. (11.4-5):

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right) \qquad (11.4\text{-}5)$$

2. For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration, S_a, shall be taken as equal to S_{DS}.
3. For periods greater than T_S and less than or equal to T_L, the design spectral response acceleration, S_a, shall be taken as given in Eq. (11.4-6):

$$S_a = \frac{S_{D1}}{T} \tag{11.4-6}$$

4. For periods greater than T_L, S_a shall be taken as given in Eq. (11.4-7):

$$S_a = \frac{S_{D1} T_L}{T^2} \tag{11.4-7}$$

where

S_{DS} = the design spectral response acceleration parameter at short periods
S_{D1} = the design spectral response acceleration parameter at a 1-s period
T = the fundamental period of the structure, s
$T_0 = 0.2(S_{D1}/S_{DS})$
$T_S = S_{D1}/S_{DS}$, and
T_L = long-period transition period(s) shown in Figs. 22-14 through 22-17.

11.4.7 Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum. Where an MCE_R response spectrum is required, it shall be determined by multiplying the design response spectrum by 1.5.

11.4.8 Site-Specific Ground Motion Procedures. A site response analysis shall be performed in accordance with Section 21.1 for structures on Site Class F sites, unless exempted in accordance with Section 20.3.1. A ground motion hazard analysis shall be performed in accordance with Section 21.2 for the following:

1. seismically isolated structures and structures with damping systems on sites with S_1 greater than or equal to 0.6,
2. structures on Site Class E sites with S_s greater than or equal to 1.0, and
3. structures on Site Class D and E sites with S_1 greater than or equal to 0.2.

EXCEPTION: A ground motion hazard analysis is not required for structures other than seismically isolated structures and structures with damping systems where:

1. Structures on Site Class E sites with S_S greater than or equal to 1.0, provided the site coefficient F_a is taken as equal to that of Site Class C.
2. Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$.
3. Structures on Site Class E sites with S_1 greater than or equal to 0.2, provided that T is less than or equal to T_s and the equivalent static force procedure is used for design.

It shall be permitted to perform a site response analysis in accordance with Section 21.1 and/or a ground motion hazard analysis in accordance with Section 21.2 to determine ground motions for any structure.

When the procedures of either Section 21.1 or 21.2 are used, the design response spectrum shall be determined in accordance with Section 21.3, the design acceleration parameters shall be determined in accordance with Section 21.4, and, if required, the MCE_G peak ground acceleration parameter shall be determined in accordance with Section 21.5.

11.5 IMPORTANCE FACTOR AND RISK CATEGORY

11.5.1 Importance Factor. An Importance Factor, I_e, shall be assigned to each structure in accordance with Table 1.5-2.

11.5.2 Protected Access for Risk Category IV. Where operational access to a Risk Category IV structure is required through an adjacent structure, the adjacent structure shall conform to the requirements for Risk Category IV structures. Where operational access is less than 10 ft (3.048 m) from an interior lot line or another structure on the same lot, protection from potential falling debris from adjacent structures shall be provided by the owner of the Risk Category IV structure.

11.6 SEISMIC DESIGN CATEGORY

Structures shall be assigned a Seismic Design Category in accordance with this Table 1.5-1

Risk Category I, II, or III structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Risk Category IV structures located where the mapped spectral response acceleration parameter at 1-s period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F. All other structures shall be assigned to a Seismic Design Category based on their Risk Category and the design spectral response acceleration parameters, S_{DS} and S_{D1}, determined in accordance with Section 11.4.5. Each building and structure shall be assigned to the more severe Seismic Design Category in accordance with Table 11.6-1 or 11.6-2, irrespective

TABLE 11.6-1 Seismic Design Category Based on Short-Period Response Acceleration Parameter

	Risk Category	
Value of S_{DS}	I or II or III	IV
$S_{DS} < 0.167$	A	A
$0.167 \leq S_{DS} < 0.33$	B	C
$0.33 \leq S_{DS} < 0.50$	C	D
$0.50 \leq S_{DS}$	D	D

TABLE 11.6-2 Seismic Design Category Based on 1-s Period Response Acceleration Parameter

	Risk Category	
Value of S_{D1}	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \leq S_{D1} < 0.133$	B	C
$0.133 \leq S_{D1} < 0.20$	C	D
$0.20 \leq S_{D1}$	D	D

For the building, we assign the higher SDC, so for a building to be constructed on this site, SDC is D of the fundamental period of vibration of the structure, T. The provisions in Chapter 19 shall not be used to modify the spectral response acceleration parameters for determining Seismic Design Category.

Where S_1 is less than 0.75, the Seismic Design Category is permitted to be determined from Table 11.6-1 alone where all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, determined in accordance with Section 12.8.2.1 is less than $0.8T_s$, where T_s is determined in accordance with Section 11.4.6.
2. In each of two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Eq. (12.8-2) is used to determine the seismic response coefficient C_s.
4. The diaphragms are rigid in accordance with Section 12.3; or, for diaphragms that are not rigid, the horizontal distance between vertical elements of the seismic force-resisting system does not exceed 40 ft (12.192 m).

Where the alternate simplified design procedure of Section 12.14 is used, the Seismic Design Category is permitted to be determined from Table 11.6-1 alone, using the value of S_{DS} determined in Section 12.14.8.1, except that where S_1 is greater than or equal to 0.75, the Seismic Design Category shall be E.

11.7 DESIGN REQUIREMENTS FOR SEISMIC DESIGN CATEGORY A

Buildings and other structures assigned to Seismic Design Category A need only comply with the requirements of Section 1.4. Nonstructural components in SDC A are exempt from seismic design requirements. In addition, tanks assigned to Risk Category IV shall satisfy the freeboard requirement in Section 15.6.5.1.

11.8 GEOLOGIC HAZARDS AND GEOTECHNICAL INVESTIGATION

11.8.1 Site Limitation for Seismic Design Categories E and F. A structure assigned to Seismic Design Category E or F shall not be located where a known potential exists for an active fault to cause rupture of the ground surface at the structure.

11.8.2 Geotechnical Investigation Report Requirements for Seismic Design Categories C through F. A geotechnical investigation report shall be provided for a structure assigned to Seismic Design Category C, D, E, or F in accordance with this section. An investigation shall be conducted, and a report shall be submitted that includes an evaluation of the following potential geologic and seismic hazards:

a. Slope instability,
b. Liquefaction,
c. Total and differential settlement, and
d. Surface displacement caused by faulting or seismically induced lateral spreading or lateral flow.

The report shall contain recommendations for foundation designs or other measures to mitigate the effects of the previously mentioned hazards.

EXCEPTION: Where approved by the authority having jurisdiction, a site-specific geotechnical report is not required where prior evaluations of nearby sites with similar soil conditions provide direction relative to the proposed construction.

11.8.3 Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F. The geotechnical investigation report for a structure assigned to Seismic Design Category D, E, or F shall include all of the following, as applicable:

1. The determination of dynamic seismic lateral earth pressures on basement and retaining walls caused by design earthquake ground motions.
2. The potential for liquefaction and soil strength loss evaluated for site peak ground acceleration, earthquake magnitude, and source characteristics consistent with the MCE_G peak ground acceleration. Peak ground acceleration shall be determined based on either (1) a site-specific study taking into account soil amplification effects as specified in Section 11.4.8 or (2) the peak ground acceleration PGA_M, from Eq. (11.8-1).

$$PGA_M = F_{PGA} \cdot PGA \qquad (11.8\text{-}1)$$

where

PGA_M = MCE_G peak ground acceleration adjusted for site class effects.
PGA = Mapped MCE_G peak ground acceleration shown in Figs. 22-9 through 22-13.
F_{PGA} = Site coefficient from Table 11.8-1.

where Site Class D is selected as the default site class per Section 11.4.3, the value of F_{PGA} shall not be less than 1.2.

3. Assessment of potential consequences of liquefaction and soil strength loss, including, but not limited to, estimation of total and differential settlement, lateral soil movement, lateral soil loads on foundations, reduction in foundation soil-bearing capacity and lateral soil reaction, soil downdrag and reduction in axial and lateral soil reaction for pile foundations, increases in soil lateral pressures on retaining walls, and flotation of buried structures.
4. Discussion of mitigation measures such as, but not limited to, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements and forces, ground stabilization, or any combination of these measures and how they shall be considered in the design of the structure.

TABLE 11.8-1 Site Coefficient F_{PGA}

	Mapped Maximum Considered Geometric Mean (MCE_G) Peak Ground Acceleration, PGA					
Site Class	$PGA \le 0.1$	$PGA = 0.2$	$PGA = 0.3$	$PGA = 0.4$	$PGA = 0.5$	$PGA \ge 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.2	1.2	1.2	1.2	1.2
D	1.6	1.4	1.3	1.2	1.1	1.1
E	2.4	1.9	1.6	1.4	1.2	1.1
F	See Section 11.4.8					

Note: Use straight-line interpolation for intermediate values of PGA.

11.9 VERTICAL GROUND MOTIONS FOR SEISMIC DESIGN

11.9.1 General. If the option to incorporate the effects of vertical seismic ground motions is exercised in lieu of the requirements of Section 12.4.2.2, the requirements of this section are permitted to be used in the determination of the vertical design earthquake ground motions. The requirements of Section 11.9 shall only apply to structures in Seismic Design Categories C, D, E, and F.

11.9.2 MCE_R Vertical Response Spectrum. Where a vertical response spectrum is required by this standard and site-specific procedures are not used, the MCE_R vertical response spectral acceleration, S_{aMv}, shall be developed as follows:

1. For vertical periods less than or equal to 0.025 s, S_{aMv} shall be determined in accordance with Eq. (11.9-1) as follows:

$$S_{aMv} = 0.3C_v S_{MS} \qquad (11.9\text{-}1)$$

2. For vertical periods greater than 0.025 s and less than or equal to 0.05 s, S_{aMv} shall be determined in accordance with Eq. (11.9-2) as follows:

$$S_{aMv} = 20C_v S_{MS}(T_v - 0.025) + 0.3C_v S_{MS} \qquad (11.9\text{-}2)$$

3. For vertical periods greater than 0.05 s and less than or equal to 0.15 s, S_{aMv} shall be determined in accordance with Eq. (11.9-3) as follows:

$$S_{aMv} = 0.8C_v S_{MS} \qquad (11.9\text{-}3)$$

4. For vertical periods greater than 0.15 s and less than or equal to 2.0 s, S_{aMv} shall be determined in accordance with Eq. (11.9-4) as follows:

$$S_{aMv} = 0.8C_v S_{MS}\left(\frac{0.15}{T_v}\right)^{0.75} \qquad (11.9\text{-}4)$$

where

C_v = is defined in terms of S_S in Table 11.9-1,
S_{MS} = the MCE_R spectral response acceleration parameter at short periods, and
T_v = the vertical period of vibration.

TABLE 11.9-1 Values of Vertical Coefficient C_v

Mapped MCE_R Spectral Response Parameter at Short Periods[a]	Site Class A, B	Site Class C	Site Class D, E, F
$S_S \geq 2.0$	0.9	1.3	1.5
$S_S = 1.0$	0.9	1.1	1.3
$S_S = 0.6$	0.9	1.0	1.1
$S_S = 0.3$	0.8	0.8	0.9
$S_S \leq 0.2$	0.7	0.7	0.7

[a]Use straight-line interpolation for intermediate values of S_S.

S_{aMv} shall not be less than one-half of the corresponding S_{aM} for horizontal components determined in accordance with the general or site-specific procedures of Section 11.4 or Chapter 21, respectively.

For vertical periods greater than 2.0 s, S_{aMv} shall be developed from a site-specific procedure; however, the resulting ordinate of S_{aMv} shall not be less than one-half of the corresponding S_a for horizontal components determined in accordance with the general or site-specific procedures of Section 11.4 or Chapter 21, respectively.

In lieu of using the above procedure, a site-specific study is permitted to be performed to obtain S_{aMv} at vertical periods less than or equal to 2.0 s, but the value so determined shall not be less than 80% of the S_{aMv} value determined from Eqs. (11.9-1) through (11.9-4).

11.9.3 Design Vertical Response Spectrum. The design vertical response spectral acceleration, S_{av}, shall be taken as two-thirds of the value of S_{aMv} determined in Section 11.9.2.

11.10 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

See Chapter 23 for the list of consensus standards and other documents that shall be considered part of this standard to the extent referenced in this chapter.

This page intentionally left blank

CHAPTER 12
SEISMIC DESIGN REQUIREMENTS FOR BUILDING STRUCTURES

12.1 STRUCTURAL DESIGN BASIS

12.1.1 Basic Requirements. The seismic analysis and design procedures to be used in the design of building structures and their members shall be as prescribed in this section. 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. The design seismic forces and their distribution over the height of the building structure shall be established in accordance with one of the applicable procedures indicated in Section 12.6, and the corresponding internal forces and deformations in the members of the structure shall be determined. An approved alternative procedure shall not be used to establish the seismic forces and their distribution unless the corresponding internal forces and deformations in the members are determined using a model consistent with the procedure adopted.

EXCEPTION: As an alternative, the simplified design procedures of Section 12.14 are permitted to be used in lieu of the requirements of Sections 12.1 through 12.12, subject to all of the limitations contained in Section 12.14.

12.1.2 Member Design, Connection Design, and Deformation Limit. Individual members, including those not part of the seismic force-resisting system, shall be provided with adequate strength to resist the shears, axial forces, and moments determined in accordance with this standard, and connections shall develop the strength of the connected members or the forces indicated in Section 12.1.1. The deformation of the structure shall not exceed the prescribed limits where the structure is subjected to the design seismic forces.

12.1.3 Continuous Load Path and Interconnection. A continuous load path, or paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the final point of resistance. All parts of the structure between separation joints shall be interconnected to form a continuous path to the seismic force-resisting system, and the connections shall be capable of transmitting the seismic force (F_p) induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements that have a design strength capable of transmitting a seismic force of 0.133 times the short-period design spectral response acceleration parameter, S_{DS}, times the weight of the smaller portion or 5% of the portion's weight, whichever is greater. This connection force does not apply to the overall design of the seismic force-resisting system. Connection design forces need not exceed the maximum forces that the structural system can deliver to the connection.

12.1.4 Connection to Supports. A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder, or truss, either directly to its supporting elements or to slabs designed to act as diaphragms. Where the connection is through a diaphragm, then the member's supporting element must also be connected to the diaphragm. The connection shall have a minimum design strength of 5% of the dead plus live load reaction.

12.1.5 Foundation Design. The foundation shall be designed to resist the forces developed and to accommodate the movements imparted to the structure and foundation by the design ground motions. The dynamic nature of the forces, the expected ground motion, the design basis for strength and energy dissipation capacity of the structure, and the dynamic properties of the soil shall be included in the determination of the foundation design criteria. The design and construction of foundations shall comply with Section 12.13.

When calculating load combinations using either the load combinations specified in Sections 2.3 or 2.4, the weights of foundations shall be considered dead loads in accordance with Section 3.1.2. The dead loads are permitted to include overlying fill and paving materials.

12.1.6 Material Design and Detailing Requirements. Structural elements, including foundation elements, shall conform to the material design and detailing requirements set forth in Chapter 14.

12.2 STRUCTURAL SYSTEM SELECTION

Based on our SDC

12.2.1 Selection and Limitations. Except as noted in Section 12.2.1.1, the basic lateral and vertical seismic force-resisting system shall conform to one of the types indicated in Table 12.2-1 or a combination of systems as permitted in Sections 12.2.2, 12.2.3, and 12.2.4. Each system is subdivided by the types of vertical elements used to resist lateral seismic forces. The structural systems used shall be in accordance with the structural system limitations and the limits on structural height, h_n, contained in Table 12.2-1. The appropriate response modification coefficient, R; overstrength factor, Ω_0; and deflection amplification factor, C_d, indicated in Table 12.2-1 shall be used in determining the base shear, element design forces, and design story drift.

Each selected seismic force-resisting system shall be designed and detailed in accordance with the specific requirements for the

Table 12.2-1 Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n (ft) Limits[d] — Seismic Design Category B	C	D[e]	E[e]	F[f]
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls[g,h]	14.2	5	2½	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls[g]	14.2	4	2½	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls[g]	14.2	2	2½	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
5. Intermediate precast shear walls[g]	14.2	4	2½	4	NL	NL	40[i]	40[i]	40[i]
6. Ordinary precast shear walls[g]	14.2	3	2½	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
13. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2	NL	35	NP	NP	NP
14. Ordinary plain AAC masonry shear walls	14.4	1½	2½	1½	NL	NP	NP	NP	NP
15. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	3	4	NL	NL	65	65	65
16. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	3	4	NL	NL	65	65	65
17. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2½	2	NL	NL	35	NP	NP
18. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3½	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]
4. Special reinforced concrete shear walls[g,h]	14.2	6	2½	5	NL	NL	160	160	100
5. Ordinary reinforced concrete shear walls[g]	14.2	5	2½	4½	NL	NL	NP	NP	NP
6. Detailed plain concrete shear walls[g]	14.2 and 14.2.2.7	2	2½	2	NL	NP	NP	NP	NP
7. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
8. Intermediate precast shear walls[g]	14.2	5	2½	4½	NL	NL	40[i]	40[i]	40[i]
9. Ordinary precast shear walls[g]	14.2	4	2½	4	NL	NP	NP	NP	NP
10. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4	NL	NL	160	160	100
11. Steel and concrete composite special concentrically braced frames	14.3	5	2	4½	NL	NL	160	160	100
12. Steel and concrete composite ordinary braced frames	14.3	3	2	3	NL	NL	NP	NP	NP
13. Steel and concrete composite plate shear walls	14.3	6½	2½	5½	NL	NL	160	160	100
14. Steel and concrete composite special shear walls	14.3	6	2½	5	NL	NL	160	160	100
15. Steel and concrete composite ordinary shear walls	14.3	5	2½	4½	NL	NL	NP	NP	NP
16. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100
17. Intermediate reinforced masonry shear walls	14.4	4	2½	4	NL	NL	NP	NP	NP

18. Ordinary reinforced masonry shear walls	14.4	2	2½	2	NL	160	NP	NP	NP
19. Detailed plain masonry shear walls	14.4	2	2½	2	NL	NP	NP	NP	NP
20. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
21. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
22. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	7	2½	4½	NL	NL	65	65	65
23. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	7	2½	4½	NL	NL	65	65	65
24. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2½	2½	2½	NL	NL	35	NP	NP
25. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	160	160	100
26. Steel special plate shear walls	14.1	7	2	6	NL	NL	160	160	100
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35[k]	NP[k]	NP[k]
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP[l]	NP[l]	NP[l]
5. Special reinforced concrete moment frames[m]	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Steel and concrete composite special moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Steel and concrete composite intermediate moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Steel and concrete composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Steel and concrete composite ordinary moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP
12. Cold-formed steel—special bolted moment frame[n]	14.1	3½	3[o]	3½	35	35	35	35	35
D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Steel eccentrically braced frames	14.1	8	2½	4	NL	NL	NL	NL	NL
2. Steel special concentrically braced frames	14.1	7	2½	5½	NL	NL	NL	NL	NL
3. Special reinforced concrete shear walls[g,h]	14.2	7	2½	5½	NL	NL	NL	NL	NL
4. Ordinary reinforced concrete shear walls[g]	14.2	6	2½	5	NL	NL	NP	NP	NP
5. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4	NL	NL	NL	NL	NL
6. Steel and concrete composite special concentrically braced frames	14.3	6	2½	5	NL	NL	NL	NL	NL
7. Steel and concrete composite plate shear walls	14.3	7½	2½	6	NL	NL	NL	NL	NL
8. Steel and concrete composite special shear walls	14.3	7	2½	6	NL	NL	NL	NL	NL
9. Steel and concrete composite ordinary shear walls	14.3	6	2½	5	NL	NL	NP	NP	NP
10. Special reinforced masonry shear walls	14.4	5½	3	5	NL	NL	NL	NL	NL
11. Intermediate reinforced masonry shear walls	14.4	4	3	3½	NL	NL	NP	NP	NP
12. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	NL	NL	NL
13. Steel special plate shear walls	14.1	8	2½	6½	NL	NL	NL	NL	NL
E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Steel special concentrically braced frames[p]	14.1	6	2½	5	NL	NL	35	NP	NP
2. Special reinforced concrete shear walls[g,h]	14.2	6½	2½	5	NL	NL	160	100	100
3. Ordinary reinforced masonry shear walls	14.4	3	3	2½	NL	160	NP	NP	NP
4. Intermediate reinforced masonry shear walls	14.4	3½	3	3	NL	NL	NP	NP	NP

continues

Table 12.2-1 *(Continued)* Design Coefficients and Factors for Seismic Force-Resisting Systems

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, $\Omega_0{}^b$	Deflection Amplification Factor, $C_d{}^c$	Structural System Limitations Including Structural Height, h_n (ft) Limits[d] — Seismic Design Category: B	C	D^e	E^e	F^f
5. Steel and concrete composite special concentrically braced frames	14.3	5½	2½	4½	NL	NL	160	100	NP
6. Steel and concrete composite ordinary braced frames	14.3	3½	2½	3	NL	NL	NP	NP	NP
7. Steel and concrete composite ordinary shear walls	14.3	5	3	4½	NL	NL	NP	NP	NP
8. Ordinary reinforced concrete shear walls[g]	14.2	5½	2½	4½	NL	NL	NP	NP	NP
F. SHEAR WALL-FRAME INTERACTIVE SYSTEM WITH ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS[g]	12.2.5.8 and 14.2	4½	2½	4	NL	NP	NP	NP	NP
G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:	12.2.5.2								
1. Steel special cantilever column systems	14.1	2½	1¼	2½	35	35	35	35	35
2. Steel ordinary cantilever column systems	14.1	1¼	1¼	1¼	35	35	NP[i]	NP[i]	NP[i]
3. Special reinforced concrete moment frames[m]	12.2.5.5 and 14.2	2½	1¼	2½	35	35	35	35	35
4. Intermediate reinforced concrete moment frames	14.2	1½	1¼	1½	35	35	NP	NP	NP
5. Ordinary reinforced concrete moment frames	14.2	1	1¼	1	35	NP	NP	NP	NP
6. Timber frames	14.5	1½	1½	1½	35	35	35	NP	NP
H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE, EXCLUDING CANTILEVER COLUMN SYSTEMS	14.1	3	3	3	NL	NL	NP	NP	NP

[a]Response modification coefficient, R, for use throughout the standard. Note that R reduces forces to a strength level, not an allowable stress level.
[b]Where the tabulated value of the overstrength factor, Ω_0, is greater than or equal to 2½, Ω_0 is permitted to be reduced by subtracting the value of $1/2$ for structures with flexible diaphragms.
[c]Deflection amplification factor, C_d, for use in Sections 12.8.6, 12.8.7, and 12.9.1.2.
[d]NL = Not Limited, and NP = Not Permitted. For metric units, use 30.5 m for 100 ft and use 48.8 m for 160 ft.
[e]See Section 12.2.5.4 for a description of seismic force-resisting systems limited to buildings with a structural height, h_n, of 240 ft (73.2 m) or less.
[f]See Section 12.2.5.4 for seismic force-resisting systems limited to buildings with a structural height, h_n, of 160 ft (48.8 m) or less.
[g]In Section 2.3 of ACI 318. A shear wall is defined as a structural wall.
[h]In Section 2.3 of ACI 318. The definition of "special structural wall" includes precast and cast-in-place construction.
[i]An increase in structural height, h_n, to 45 ft (13.7 m) is permitted for single-story storage warehouse facilities.
[j]Steel ordinary concentrically braced frames are permitted in single-story buildings up to a structural height, h_n, of 60 ft (18.3 m) where the dead load of the roof does not exceed 20 lb/ft^2 (0.96 kN/m^2) and in penthouse structures.
[k]See Section 12.2.5.7 for limitations in structures assigned to Seismic Design Categories D, E, or F.
[l]See Section 12.2.5.6 for limitations in structures assigned to Seismic Design Categories D, E, or F.
[m]In Section 2.3 of ACI 318. The definition of "special moment frame" includes precast and cast-in-place construction.
[n]Cold-formed steel—special bolted moment frames shall be limited to one story in height in accordance with ANSI/AISI S400.
[o]Alternately, the seismic load effect including overstrength, E_{mh}, is permitted to be based on the expected strength determined in accordance with ANSI/AISI S400.
[p]Ordinary moment frame is permitted to be used in lieu of intermediate moment frame for Seismic Design Categories B or C.

system as set forth in the applicable reference document listed in Table 12.2-1 and the additional requirements set forth in Chapter 14.

Nothing contained in this section shall prohibit the use of alternative procedures for the design of individual structures that demonstrate acceptable performance in accordance with the requirements of Section 1.3.1.3 of this standard.

12.2.1.1 Alternative Structural Systems. Use of seismic force-resisting systems not contained in Table 12.2-1 shall be permitted contingent on submittal to and approval by the Authority Having Jurisdiction and independent structural design review of an accompanying set of design criteria and substantiating analytical and test data. The design criteria shall specify any limitations on system use, including Seismic Design Category and height; required procedures for designing the system's components and connections; required detailing; and the values of the response modification coefficient, R; overstrength factor, Ω_0; and deflection amplification factor, C_d. The submitted data shall establish the system's nonlinear dynamic characteristics and demonstrate that the design criteria result in a probability of collapse conditioned on the occurrence of MCE_R shaking not greater than 10% for Risk Category II structures. The conditional probability of collapse shall be determined based on a nonlinear analytical evaluation of the system and shall account for sources of uncertainty in quality of the design criteria, modeling fidelity, laboratory test data, and ground motions. Structural design review shall conform to the criteria of Section 16.5.

12.2.1.2 Elements of Seismic Force-Resisting Systems. Elements of seismic force-resisting systems, including members and their connections, shall conform to the detailing requirements specified in Table 12.2-1 for the selected structural system.

EXCEPTION: Substitute elements that do not conform to the requirements specified in Table 12.2-1 shall be permitted contingent on submittal to and approval by the authority having jurisdiction of all of the following:

a. In-depth description of the methodology used to evaluate equivalency of the substitute element for the seismic force-resisting system of interest, or reference to published documentation describing the methodology in depth.
b. Justification of the applicability of the equivalency methodology, including but not limited to consideration of the similarity of the forces transferred across the connection between the substitute and conforming elements and the balance of the seismic force-resisting system, and the similarity between the substitute and conforming element on the distribution of forces and displacements in the balance of the structure.
c. A design procedure for the substitute elements, including procedures to determine design strength stiffness, detailing, connections, and limitations to applicability and use.
d. Requirements for the manufacturing, installation, and maintenance of the substitute elements.
e. Experimental evidence demonstrating that the hysteretic characteristics of the conforming and substitute elements are similar through deformation levels anticipated in response to MCE_R shaking. The evaluation of experimental evidence shall include assessment of the ratio of the measured maximum strength to design strength; the ratio of the measured initial stiffness to design stiffness; the ultimate deformation capacity; and the cyclic strength and stiffness deterioration characteristics of the conforming and substitute elements.
f. Evidence of independent structural design review, in accordance with Section 16.5 or review by a third party acceptable to the authority having jurisdiction, of conformance to the requirements of this section.

12.2.2 Combinations of Framing 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, C_d, and Ω_0 coefficients shall apply to each system, including the structural system limitations contained in Table 12.2-1.

12.2.3 Combinations of Framing Systems in the Same Direction. Where different seismic force-resisting systems are used in combination to resist seismic forces in the same direction, other than those combinations considered as dual systems, the most stringent applicable structural system limitations contained in Table 12.2-1 shall apply and the design shall comply with the requirements of this section.

12.2.3.1 R, C_d, and Ω_0 Values for Vertical Combinations. Where a structure has a vertical combination in the same direction, the following requirements shall apply:

1. Where the lower system has a lower response modification coefficient, R, the design coefficients (R, Ω_0, and C_d) for the upper system are permitted to be used to calculate the forces and drifts of the upper system. For the design of the lower system, the design coefficients (R, Ω_0, and C_d) for the lower system shall be used. Forces transferred from the upper system to the lower system shall be increased by multiplying by the ratio of the higher response modification coefficient to the lower response modification coefficient.
2. Where the upper system has a lower response modification coefficient, the design coefficients (R, Ω_0, and C_d) for the upper system shall be used for both systems.

EXCEPTIONS:

1. Rooftop structures not exceeding two stories in height and 10% of the total structure weight.
2. Other supported structural systems with a weight equal to or less than 10% of the weight of the structure.
3. Detached one- and two-family dwellings of light-frame construction.

12.2.3.2 Two-Stage Analysis Procedure. A two-stage equivalent lateral force procedure is permitted to be used for structures that have a flexible upper portion above a rigid lower portion, provided that the design of the structure complies with all of the following:

a. The stiffness of the lower portion shall be at least 10 times the stiffness of the upper portion.
b. The period of the entire structure shall not be greater than 1.1 times the period of the upper portion considered as a separate structure supported at the transition from the upper to the lower portion.
c. The upper portion shall be designed as a separate structure using the appropriate values of R and ρ.
d. The lower portion shall be designed as a separate structure using the appropriate values of R and ρ. The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the

R/ρ of the upper portion over R/ρ of the lower portion. This ratio shall not be less than 1.0.

e. The upper portion is analyzed with the equivalent lateral force or modal response spectrum procedure, and the lower portion is analyzed with the equivalent lateral force procedure.

12.2.3.3 R, C_d, and Ω_0 Values for Horizontal Combinations. The value of the response modification coefficient, R, used for design in the direction under consideration shall not be greater than the least value of R for any of the systems used in that direction. The deflection amplification factor, C_d, and the overstrength factor, Ω_0, shall be consistent with R required in that direction.

EXCEPTION: Resisting elements are permitted to be designed using the least value of R for the different structural systems found in each independent line of resistance if the following three conditions are met: (1) Risk Category I or II building, (2) two stories or fewer above grade plane, and (3) use of light-frame construction or flexible diaphragms. The value of R used for design of diaphragms in such structures shall not be greater than the least value of R for any of the systems used in that same direction.

12.2.4 Combination Framing Detailing Requirements. Structural members common to different framing systems used to resist seismic forces in any direction shall be designed using the detailing requirements of Chapter 12 required by the highest response modification coefficient, R, of the connected framing systems.

12.2.5 System-Specific Requirements. The structural framing system shall also comply with the following system-specific requirements of this section.

12.2.5.1 Dual System. For a dual system, the moment frames shall be capable of resisting at least 25% of the design seismic forces. The total seismic force resistance is to be provided by the combination of the moment frames and the shear walls or braced frames in proportion to their rigidities.

12.2.5.2 Cantilever Column Systems. Cantilever column systems are permitted as indicated in Table 12.2-1 and as follows. The required axial strength of individual cantilever column elements, considering only the load combinations that include seismic load effects, shall not exceed 15% of the available axial strength, including slenderness effects.

Foundation and other elements used to provide overturning resistance at the base of cantilever column elements shall be designed to resist the seismic load effects, including overstrength of Section 12.4.3.

12.2.5.3 Inverted Pendulum-Type Structures. Regardless of the structural system selected, inverted pendulums as defined in Section 11.2 shall comply with this section. Supporting columns or piers of inverted pendulum-type structures shall be designed for the bending moment calculated at the base determined using the procedures given in Section 12.8 and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the base.

12.2.5.4 Increased Structural Height Limit for Steel Eccentrically Braced Frames, Steel Special Concentrically Braced Frames, Steel Buckling-Restrained Braced Frames, Steel Special Plate Shear Walls, and Special Reinforced Concrete Shear Walls. The limits on structural height, h_n, in Table 12.2-1 are permitted to be increased from 160 ft (50 m) to 240 ft (75 m) for structures assigned to Seismic Design Categories D or E and from 100 ft (30 m) to 160 ft (50 m) for structures assigned to Seismic Design Category F, provided that the seismic force-resisting systems are limited to steel eccentrically braced frames, steel special concentrically braced frames, steel buckling-restrained braced frames, steel special plate shear walls, or special reinforced concrete cast-in-place shear walls and both of the following requirements are met:

1. The structure shall not have an extreme torsional irregularity as defined in Table 12.3-1 (horizontal structural irregularity Type 1b).
2. The steel eccentrically braced frames, steel special concentrically braced frames, steel buckling-restrained braced frames, steel special plate shear walls, or special reinforced cast-in-place concrete shear walls in any one plane shall resist no more than 60% of the total seismic forces in each direction, neglecting accidental torsional effects.

12.2.5.5 Special Moment Frames in Structures Assigned to Seismic Design Categories D through F. For structures assigned to Seismic Design Categories D, E, or F, where a special moment frame is required by Table 12.2-1 because of the structural system limitations, the frame shall be continuous to the base.

A special moment frame that is used but not required by Table 12.2-1 is permitted to be discontinued above the base and supported by a more rigid system with a lower response modification coefficient, R, provided that the requirements of Sections 12.2.3.1 and 12.3.3.4 are met.

12.2.5.6 Steel Ordinary Moment Frames

12.2.5.6.1 Seismic Design Category D or E

a. Single-story steel ordinary moment frames in structures assigned to Seismic Design Category D or E are permitted up to a structural height, h_n, of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior walls more than 35 ft (10.6 m) above the base tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

   EXCEPTION: Single-story structures with steel ordinary moment frames whose purpose is to enclose equipment or machinery and whose occupants are engaged in maintenance or monitoring of that equipment, machinery, or their associated processes shall be permitted to be of unlimited height where the sum of the dead and equipment loads supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior wall system, including exterior columns more than 35 ft (10.6 m) above the base, shall not exceed 20 psf (0.96 kN/m^2). For determining compliance with the exterior wall or roof load limits, the weight of equipment or machinery, including cranes, not self-supporting for all loads shall be assumed to be fully tributary to the area of the adjacent exterior wall or roof not to exceed 600 ft^2 (55.8 m^2), regardless of its height above the base of the structure.

b. Steel ordinary moment frames in structures assigned to Seismic Design Category D or E not meeting the limitations set forth in Section 12.2.5.6.1.a are permitted within light-frame construction up to a structural height, h_n, of 35 ft (10.6 m) where neither the roof dead load nor the dead load of any floor above the base supported by and tributary to the moment frames exceeds 35 psf (1.68 kN/m^2). In addition, the dead load of the exterior walls tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

After assigning the structural systems for our building, we have to check for types of irregularity

Be careful and check

Table 12.3-1 Horizontal Structural Irregularities

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity:** Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 16.3.4	D, E, and F B, C, D, E, and F C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity:** Extreme torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with $A_x = 1.0$, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.3.4.2 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 16.3.4	E and F D D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity:** Reentrant corner irregularity is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity:** Diaphragm discontinuity irregularity is defined to exist where there is a diaphragm with an abrupt discontinuity or variation in stiffness, including one that has a cutout or open area greater than 50% of the gross enclosed diaphragm area, or a change in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**Out-of-Plane Offset Irregularity:** Out-of-plane offset irregularity is defined to exist where there is a discontinuity in a lateral force-resistance path, such as an out-of-plane offset of at least one of the vertical elements.	12.3.3.3 12.3.3.4 12.7.3 Table 12.6-1 16.3.4	B, C, D, E, and F D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**Nonparallel System Irregularity:** Nonparallel system irregularity is defined to exist where vertical lateral force-resisting elements are not parallel to the major orthogonal axes of the seismic force-resisting system.	12.5.3 12.7.3 Table 12.6-1 16.3.4	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

12.2.5.6.2 Seismic Design Category F. Single-story steel ordinary moment frames in structures assigned to Seismic Design Category F are permitted up to a structural height, h_n, of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior walls tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

12.2.5.7 Steel Intermediate Moment Frames

12.2.5.7.1 Seismic Design Category D

a. Single-story steel intermediate moment frames in structures assigned to Seismic Design Category D are permitted up to a structural height, h_n, of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior walls more than 35 ft (10.6 m) above the base tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

   EXCEPTION: Single-story structures with steel intermediate moment frames whose purpose is to enclose equipment or machinery and whose occupants are engaged in maintenance or monitoring of that equipment, machinery, or their associated processes shall be permitted to be of unlimited height where the sum of the dead and equipment loads supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior wall system, including exterior columns more than 35 ft (10.6 m) above the base, shall not exceed 20 psf (0.96 kN/m^2). For determining compliance with the exterior wall or roof load limits, the weight of equipment or machinery, including cranes, not self-supporting for all loads shall be assumed to be fully tributary to the area of the adjacent exterior wall or roof not to exceed 600 ft^2 (55.8 m^2), regardless of its height above the base of the structure.

b. Steel intermediate moment frames in structures assigned to Seismic Design Category D not meeting the limitations set forth in Section 12.2.5.7.1.a are permitted up to a structural height, h_n, of 35 ft (10.6 m).

12.2.5.7.2 Seismic Design Category E

a. Single-story steel intermediate moment frames in structures assigned to Seismic Design Category E are permitted up to a structural height, h_n, of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior walls more than 35 ft (10.6 m) above the base tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

EXCEPTION: Single-story structures with steel intermediate moment frames whose purpose is to enclose equipment or machinery and whose occupants are engaged in maintenance or monitoring of that equipment, machinery, or their associated processes shall be permitted to be of unlimited height where the sum of the dead and equipment loads supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior wall system, including exterior columns more than 35 ft (10.6 m) above the base, shall not exceed 20 psf (0.96 kN/m^2). For determining compliance with the exterior wall or roof load limits, the weight of equipment or machinery, including cranes, not self-supporting for all loads shall be assumed fully tributary to the area of the adjacent exterior wall or roof not to exceed 600 ft^2 (55.8 m^2), regardless of its height above the base of the structure.

b. Steel intermediate moment frames in structures assigned to Seismic Design Category E not meeting the limitations set forth in Section 12.2.5.7.2.a are permitted up to a structural height, h_n, of 35 ft (10.6 m) where neither the roof dead load nor the dead load of any floor above the base supported by and tributary to the moment frames exceeds 35 psf (1.68 kN/m^2). In addition, the dead load of the exterior walls tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

12.2.5.7.3 Seismic Design Category F

a. Single-story steel intermediate moment frames in structures assigned to Seismic Design Category F are permitted up to a structural height, h_n, of 65 ft (20 m) where the dead load supported by and tributary to the roof does not exceed 20 psf (0.96 kN/m^2). In addition, the dead load of the exterior walls tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).
b. Steel intermediate moment frames in structures assigned to Seismic Design Category F not meeting the limitations set forth in Section 12.2.5.7.3.a are permitted within light-frame construction up to a structural height, h_n, of 35 ft (10.6 m) where neither the roof dead load nor the dead load of any floor above the base supported by and tributary to the moment frames exceeds 35 psf (1.68 kN/m^2). In addition, the dead load of the exterior walls tributary to the moment frames shall not exceed 20 psf (0.96 kN/m^2).

12.2.5.8 Shear Wall–Frame Interactive Systems. The shear strength of the shear walls of the shear wall–frame interactive system shall be at least 75% of the design story shear at each story. The frames of the shear wall–frame interactive system shall be capable of resisting at least 25% of the design story shear in every story.

12.3 DIAPHRAGM FLEXIBILITY, CONFIGURATION IRREGULARITIES, AND REDUNDANCY

12.3.1 Diaphragm Flexibility. The structural analysis shall consider the relative stiffnesses of diaphragms and the vertical elements of the seismic force-resisting system. Unless a diaphragm can be idealized as either flexible or rigid in accordance with Sections 12.3.1.1, 12.3.1.2, or 12.3.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semirigid modeling assumption).

12.3.1.1 Flexible Diaphragm Condition. Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:

a. In structures where the vertical elements are steel braced frames; steel and concrete composite braced frames; or concrete, masonry, steel, or steel and concrete composite shear walls.
b. In one- and two-family dwellings.
c. In structures of light-frame construction where all of the following conditions are met:
   1. Topping of concrete or similar materials is not placed over wood structural panel diaphragms except for nonstructural topping no greater than 1 1/2 in. (38 mm) thick.
   2. Each line of vertical elements of the seismic force-resisting system complies with the allowable story drift of Table 12.12-1.

12.3.1.2 Rigid Diaphragm Condition. Diaphragms of concrete slabs or concrete-filled metal deck with span-to-depth ratios of 3 or less in structures that have no horizontal irregularities are permitted to be idealized as rigid.

12.3.1.3 Calculated Flexible Diaphragm Condition. Diaphragms not satisfying the conditions of Sections 12.3.1.1 or 12.3.1.2 are permitted to be idealized as flexible provided:

$$\frac{\delta_{MDD}}{\Delta_{ADVE}} > 2 \qquad (12.3\text{-}1)$$

where δ_{MDD} and Δ_{ADVE} are as shown in Fig. 12.3-1. The loading used in this calculation shall be that prescribed in Section 12.8.

12.3.2 Irregular and Regular Classification. Structures shall be classified as having a structural irregularity based on the criteria in this section. Such classification shall be based on their structural configurations.

12.3.2.1 Horizontal Irregularity. Structures that have one or more of the irregularity types listed in Table 12.3-1 shall be designated as having a horizontal structural irregularity. Such structures assigned to the Seismic Design Categories listed in Table 12.3-1 shall comply with the requirements in the sections referenced in that table.

12.3.2.2 Vertical Irregularity. Structures that have one or more of the irregularity types listed in Table 12.3-2 shall be designated as having a vertical structural irregularity. Such structures assigned to the Seismic Design Categories listed in Table 12.3-2 shall comply with the requirements in the sections referenced in that table.

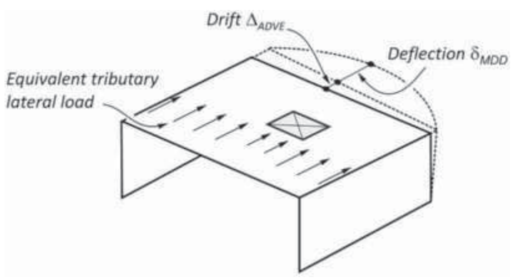


FIGURE 12.3-1 Flexible Diaphragm

what does it mean for us (in terms of lateral force distribution) rigid or flexible?

Be careful and check

Table 12.3-2 Vertical Structural Irregularities

Type	Description	Reference Section	Seismic Design Category Application
1a.	**Stiffness–Soft Story Irregularity:** Stiffness–soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
1b.	**Stiffness–Extreme Soft Story Irregularity:** Stiffness–extreme soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F
2.	**Weight (Mass) Irregularity:** Weight (mass) irregularity is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 12.6-1	D, E, and F
3.	**Vertical Geometric Irregularity:** Vertical geometric irregularity is defined to exist where the horizontal dimension of the seismic force-resisting system in any story is more than 130% of that in an adjacent story.	Table 12.6-1	D, E, and F
4.	**In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity:** In-plane discontinuity in vertical lateral force-resisting element irregularity is defined to exist where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on supporting structural elements.	12.3.3.3 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F D, E, and F
5a.	**Discontinuity in Lateral Strength–Weak Story Irregularity:** Discontinuity in lateral strength–weak story irregularity is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 Table 12.6-1	E and F D, E, and F
5b.	**Discontinuity in Lateral Strength–Extreme Weak Story Irregularity:** Discontinuity in lateral strength–extreme weak story irregularity is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 12.3.3.2 Table 12.6-1	D, E, and F B and C D, E, and F

EXCEPTIONS:

1. Vertical structural irregularities of Types 1a, 1b, and 2 in Table 12.3-2 do not apply where no story drift ratio under design lateral seismic force is greater than 130% of the story drift ratio of the next story above. Torsional effects need not be considered in the calculation of story drifts. The story–drift ratio relationship for the top two stories of the structure are not required to be evaluated.
2. Vertical structural irregularities of Types 1a, 1b, and 2 in Table 12.3-2 are not required to be considered for one-story buildings in any Seismic Design Category or for two-story buildings assigned to Seismic Design Categories B, C, or D.

12.3.3 Limitations and Additional Requirements for Systems with Structural Irregularities

12.3.3.1 Prohibited Horizontal and Vertical Irregularities for Seismic Design Categories D through F. Structures assigned to Seismic Design Category E or F that have horizontal irregularity Type 1b of Table 12.3-1 or vertical irregularities Type 1b, 5a, or 5b of Table 12.3-2 shall not be permitted. Structures assigned to Seismic Design Category D that have vertical irregularity Type 5b of Table 12.3-2 shall not be permitted.

12.3.3.2 Extreme Weak Stories. Structures with a vertical irregularity Type 5b, as defined in Table 12.3-2, shall not be more than two stories or 30 ft (9 m) in structural height, h_n.

EXCEPTION: The limit does not apply where the "weak" story is capable of resisting a total seismic force equal to Ω_0 times the design force prescribed in Section 12.8.

12.3.3.3 Elements Supporting Discontinuous Walls or Frames. Structural elements supporting discontinuous walls or frames of structures that have horizontal irregularity Type 4 of Table 12.3-1 or vertical irregularity Type 4 of Table 12.3-2 shall be designed to resist the seismic load effects, including overstrength of Section 12.4.3. The connections of such discontinuous walls or frames to the supporting members shall be adequate to transmit the forces for which the discontinuous walls or frames were required to be designed.

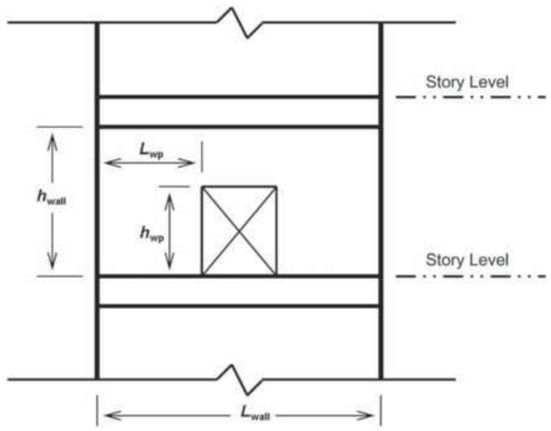
12.3.3.4 Increase in Forces Caused by Irregularities for Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E, or F and having a horizontal structural irregularity of Type 1a, 1b, 2, 3, or 4 in Table 12.3-1 or a vertical structural irregularity of Type 4 in Table 12.3-2, the design forces determined from Section 12.10.1.1 shall be increased 25% for the following elements of the seismic force-resisting system:

1. Connections of diaphragms to vertical elements and to collectors and
2. Collectors and their connections, including connections to vertical elements, of the seismic force-resisting system.

EXCEPTION: Forces calculated using the seismic load effects, including overstrength of Section 12.4.3, need not be increased.

Table 12.3-3 Requirements for Each Story Resisting More than 35% of the Base Shear

Lateral Force-Resisting Element	Requirement
Braced frames	Removal of an individual brace, or connection thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Moment frames	Loss of moment resistance at the beam-to-column connections at both ends of a single beam would not result in more than a 33% reduction in story strength; nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Shear walls or wall piers with a height-to-length ratio greater than 1.0	Removal of a shear wall or wall pier with a height-to-length ratio greater than 1.0 within any story, or collector connections thereto, would not result in more than a 33% reduction in story strength; nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b). The shear wall and wall pier height-to-length ratios are determined as shown in Fig. 12.3-2.
Cantilever columns	Loss of moment resistance at the base connections of any single cantilever column would not result in more than a 33% reduction in story strength; nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Other	No requirements.



Notes: h_{wall} = height of shear wall; h_{wp} = height of wall pier; L_{wall} = length of shear wall; L_{wp} = length of wall pier. Shear wall height-to-length ratio: h_{wall}/L_{wall}. Wall pier height-to-length ratio: h_{wp}/L_{wp}.

FIGURE 12.3-2 Shear Wall and Wall Pier Height-to-Length Ratio Determination

12.3.4 Redundancy. A redundancy factor, ρ, shall be assigned to the seismic force-resisting system in each of two orthogonal directions for all structures in accordance with this section.

12.3.4.1 Conditions Where Value of ρ is 1.0. The value of ρ is permitted to equal 1.0 for the following:

1. Structures assigned to Seismic Design Category B or C;
2. Drift calculation and P-delta effects;
3. Design of nonstructural components;
4. Design of nonbuilding structures that are not similar to buildings;
5. Design of collector elements, splices, and their connections for which the seismic load effects, including overstrength of Section 12.4.3, are used;
6. Design of members or connections where the seismic load effects, including overstrength of Section 12.4.3, are required for design;
7. Diaphragm loads determined using Eq. (12.10-1), including the limits imposed by Eqs. (12.10-2) and (12.10-3);
8. Structures with damping systems designed in accordance with Chapter 18; and
9. Design of structural walls for out-of-plane forces, including their anchorage.

12.3.4.2 Redundancy Factor, ρ, for Seismic Design Categories D through F. For structures assigned to Seismic Design Category D and having extreme torsional irregularity as defined in Table 12.3-1, Type 1b, ρ shall equal 1.3. For other structures assigned to Seismic Design Category D and for structures assigned to Seismic Design Categories E or F, ρ shall equal 1.3 unless one of the following two conditions is met, whereby ρ is permitted to be taken as 1.0. A reduction in the value of ρ from 1.3 is not permitted for structures assigned to Seismic Design Category D that have an extreme torsional irregularity (Type 1b). Seismic Design Categories E and F are not also specified because extreme torsional irregularities are prohibited (see Section 12.3.3.1).

a. Each story resisting more than 35% of the base shear in the direction of interest shall comply with Table 12.3-3.
b. Structures are regular in plan at all levels provided that the seismic force-resisting systems consist of at least two bays of seismic force-resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35% of the base shear. The number of bays for a shear wall shall be calculated as the length of shear wall divided by the story height or two times the length of shear wall divided by the story height, h_{sx}, for light-frame construction.

12.4 SEISMIC LOAD EFFECTS AND COMBINATIONS

12.4.1 Applicability. All members of the structure, including those not part of the seismic force-resisting system, shall be designed using the seismic load effects of Section 12.4 unless otherwise exempted by this standard. Seismic load effects are the axial, shear, and flexural member forces resulting from application of horizontal and vertical seismic forces as set forth in Section 12.4.2. Where required, seismic load effects shall include overstrength, as set forth in Section 12.4.3.

12.4.2 Seismic Load Effect. The seismic load effect, E, shall be determined in accordance with the following:

1. For use in load combination 6 in Section 2.3.6 or load combinations 8 and 9 in Section 2.4.5, E shall be determined in accordance with Eq. (12.4-1) as follows:

$$\boxed{E = E_h + E_v} \quad (12.4\text{-}1)$$

2. For use in load combination 7 in Section 2.3.6 or load combination 10 in Section 2.4.5, E shall be determined in accordance with Eq. (12.4-2) as follows:

$$\boxed{E = E_h - E_v} \quad (12.4\text{-}2)$$

(1.2 + 0.2Sds)*D + Eh + L + 0.2*S

(0.9 - 0.2Sds)*D + Eh

where

E = seismic load effect,
E_h = effect of horizontal seismic forces as defined in Section 12.4.2.1, and
E_v = vertical seismic effect applied in the vertical downward direction as determined in Section 12.4.2.2. E_v shall be subject to reversal to the upward direction in accordance with the applicable load combinations.

12.4.2.1 Horizontal Seismic Load Effect. The horizontal seismic load effect, E_h, shall be determined in accordance with Eq. (12.4-3) as follows:

$$E_h = \rho Q_E \tag{12.4-3}$$

where

Q_E = effects of horizontal seismic forces from V or F_p (where required by Section 12.5.3 or 12.5.4, such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other) and
ρ = redundancy factor, as defined in Section 12.3.4.

12.4.2.2 Vertical Seismic Load Effect. The vertical seismic load effect, E_v, shall be determined in accordance with Eq. (12.4-4a) as follows:

$$E_v = 0.2 S_{DS} D \tag{12.4-4a}$$

where

S_{DS} = design spectral response acceleration parameter at short periods obtained from Section 11.4.5, and
D = effect of dead load.

EXCEPTIONS:

1. Where the option to incorporate the effects of vertical seismic ground motions using the provisions of Section 11.9 is required elsewhere in this standard, the vertical seismic load effect, E_v, shall be determined in accordance with Eq. (12.4-4b) as follows:

$$E_v = 0.3 S_{av} D \tag{12.4-4b}$$

   where

   S_{av} = design vertical response spectral acceleration obtained from Section 11.9.3, and
   D = effect of dead load.

2. The vertical seismic load effect, E_v, is permitted to be taken as zero for either of the following conditions:
   a. In Eqs. (12.4-1), (12.4-2), (12.4-5), and (12.4-6) for structures assigned to Seismic Design Category B.
   b. In Eq. (12.4-2) where determining demands on the soil–structure interface of foundations.

12.4.3 Seismic Load Effects Including Overstrength. Where required, the seismic load effects including overstrength shall be determined in accordance with the following:

1. For use in load combination 6 in Section 2.3.6 or load combinations 8 and 9 in Section 2.4.5, E shall be taken as equal to E_m as determined in accordance with Eq. (12.4-5) as follows:

$$E_m = E_{mh} + E_v \tag{12.4-5}$$

2. For use in load combination 7 in Section 2.3.6 or load combination 10 in Section 2.4.5, E shall be taken as equal to E_m as determined in accordance with Eq. (12.4-6) as follows:

$$E_m = E_{mh} - E_v \tag{12.4-6}$$

   where

   E_m = seismic load effect including overstrength;
   E_{mh} = effect of horizontal seismic forces, including overstrength as defined in Section 12.4.3.1 or Section 12.4.3.2; and
   E_v = vertical seismic load effect as defined in Section 12.4.2.2. E_v is an applied load in the vertical downward direction. E_v shall be subject to reversal to the upward direction as per the associated load combinations.

12.4.3.1 Horizontal Seismic Load Effect Including Overstrength. The effect of horizontal seismic forces including overstrength, E_{mh}, shall be determined in accordance with Eq. (12.4-7) as follows:

$$E_{mh} = \Omega_0 Q_E \tag{12.4-7}$$

where

Q_E = effects of horizontal seismic forces from V, F_{px}, or F_p as specified in Sections 12.8.1, 12.10, or 13.3.1 (where required by Section 12.5.3 or 12.5.4, such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other); and
Ω_0 = overstrength factor.

E_{mh} need not be taken as larger than E_{cl} where E_{cl} = the capacity-limited horizontal seismic load effect as defined in Section 11.3.

12.4.3.2 Capacity-Limited Horizontal Seismic Load Effect. Where capacity-limited design is required by the material reference document, the seismic load effect, including overstrength, shall be calculated with the capacity-limited horizontal seismic load effect, E_{cl}, substituted for E_{mh} in the load combinations of Section 2.3.6 and Section 2.4.5.

12.4.4 Minimum Upward Force for Horizontal Cantilevers for Seismic Design Categories D through F. In structures assigned to Seismic Design Category D, E, or F, horizontal cantilever structural members 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 12.4.

12.5 DIRECTION OF LOADING

12.5.1 Direction of Loading Criteria. The directions of application of seismic forces used in the design shall be those that produce the most critical load effects. It is permitted to satisfy this requirement using the procedures of Section 12.5.2 for Seismic Design Category B, Section 12.5.3 for Seismic Design Category C, and Section 12.5.4 for Seismic Design Categories D, E, and F.

12.5.2 Seismic Design Category B. For structures assigned to Seismic Design Category B, the design seismic forces are permitted to be applied independently in each of two orthogonal directions, and orthogonal interaction effects are permitted to be neglected.

12.5.3 Seismic Design Category C. Loading applied to structures assigned to Seismic Design Category C shall, as a minimum, conform to the requirements of Section 12.5.2 for Seismic Design Category B and the requirements of this section.

12.5.3.1 Structures with Nonparallel System Irregularities. Structures that have horizontal structural irregularity of Type 5 in Table 12.3-1 shall use one of the following procedures:

a. **Orthogonal Combination Procedure.** The structure shall be analyzed using the equivalent lateral force analysis procedure of Section 12.8, the modal response spectrum analysis (MRSA) procedure of Section 12.9.1, or the linear response history procedure of Section 12.9.2, as permitted under Section 12.6, with the loading applied independently in any two orthogonal directions. The requirement of Section 12.5.1 is deemed satisfied if members and their foundations are designed for 100% of the forces for one direction plus 30% of the forces for the perpendicular direction. The combination requiring the maximum component strength shall be used.

b. **Simultaneous Application of Orthogonal Ground Motion.** The structure shall be analyzed using the linear response history procedure of Section 12.9.2 or the nonlinear response history procedure of Chapter 16, as permitted by Section 12.6, with orthogonal pairs of ground motion acceleration histories applied simultaneously.

12.5.4 Seismic Design Categories D through F. Structures assigned to Seismic Design Category D, E, or F shall, as a minimum, conform to the requirements of Section 12.5.3. In addition, any column or wall that forms part of two or more intersecting seismic force-resisting systems and is subjected to axial load due to seismic forces acting along either principal plan axis equaling or exceeding 20% of the axial design strength of the column or wall shall be designed for the most critical load effect due to application of seismic forces in any direction. Either of the procedures of Section 12.5.3.a or 12.5.3.b are permitted to be used to satisfy this requirement. Except as required by Section 12.7.3, 2D analyses are permitted for structures with flexible diaphragms.

12.6 ANALYSIS PROCEDURE SELECTION

The structural analysis required by Chapter 12 shall consist of one of the types permitted in Table 12.6-1, based on the structure's Seismic Design Category, structural system, dynamic properties, and regularity, or with the approval of the authority having jurisdiction, an alternative generally accepted procedure is permitted to be used. The analysis procedure selected shall be completed in accordance with the requirements of the corresponding section referenced in Table 12.6-1.

12.7 MODELING CRITERIA

12.7.1 Foundation Modeling. For purposes of determining seismic loads, it is permitted to consider the structure to be fixed at the base. Alternatively, where foundation flexibility is considered, it shall be in accordance with Section 12.13.3 or Chapter 19.

12.7.2 Effective Seismic Weight. The effective seismic weight, W, of a structure shall include the dead load, as defined in Section 3.1, above the base and other loads above the base as listed below:

1. In areas used for storage, a minimum of 25% of the floor live load shall be included.
   EXCEPTIONS:
   a. Where the inclusion of storage loads adds no more than 5% to the effective seismic weight at that level, it need not be included in the effective seismic weight.
   b. Floor live load in public garages and open parking structures need not be included.
2. Where provision for partitions is required by Section 4.3.2 in the floor load design, the actual partition weight or a minimum weight of 10 psf (0.48 kN/m^2) of floor area, whichever is greater.
3. Total operating weight of permanent equipment.
4. Where the flat roof snow load, P_f, exceeds 30 psf (1.44 kN/m^2), 20% of the uniform design snow load, regardless of actual roof slope.
5. Weight of landscaping and other materials at roof gardens and similar areas.

Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Procedure, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9.1, or Linear Response History Analysis, Section 12.9.2[a]	Nonlinear Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding two stories above the base	P	P	P
	Structures of light-frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft (48.8 m) in structural height	P	P	P
	Structures exceeding 160 ft (48.8 m) in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft (48.8 m) in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2	P	P	P
	All other structures	NP	P	P

[a]P: Permitted; NP: Not Permitted; $T_s = S_{D1}/S_{DS}$.

12.7.3 Structural Modeling. A mathematical model of the structure shall be constructed for the purpose of determining member forces and structure displacements resulting from applied loads and any imposed displacements or P-delta effects. The model shall include the stiffness and strength of elements that are significant to the distribution of forces and deformations in the structure and represent the spatial distribution of mass and stiffness throughout the structure.

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. For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.

Structures that have horizontal structural irregularity Type 1a, 1b, 4, or 5 of Table 12.3-1 shall be analyzed using a 3D representation. Where a 3D model is used, a minimum of three degrees of freedom consisting of translation in two orthogonal plan directions and rotation about the vertical axis shall be included at each level of the structure. Where the diaphragms have not been classified as rigid or flexible in accordance with Section 12.3.1, the model shall include representation of the diaphragm's stiffness characteristics and, when dynamic analysis is performed, sufficient degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response. When modal response spectrum or response history analysis is performed, 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.

EXCEPTION: Analysis using a 3D representation is not required for structures with flexible diaphragms that have Type 4 horizontal structural irregularities.

12.7.4 Interaction Effects. Moment-resisting frames that are enclosed or adjoined by elements that are more rigid and not considered to be part of the seismic force-resisting system shall be designed so that the action or failure of those elements will not impair the vertical load and seismic force-resisting capability of the frame. The design shall provide for the effect of these rigid elements on the structural system at structural deformations corresponding to the design story drift (Δ) as determined in Section 12.8.6. In addition, the effects of these elements shall be considered where determining whether a structure has one or more of the irregularities defined in Section 12.3.2.

12.8 EQUIVALENT LATERAL FORCE (ELF) PROCEDURE

12.8.1 Seismic Base Shear. The seismic base shear, V, in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \tag{12.8-1}$$

where

C_s = the seismic response coefficient determined in accordance with Section 12.8.1.1, and
W = the effective seismic weight per Section 12.7.2.

12.8.1.1 Calculation of Seismic Response Coefficient. The seismic response coefficient, C_s, shall be determined in accordance with Eq. (12.8-2).

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \tag{12.8-2}$$

where

S_{DS} = the design spectral response acceleration parameter in the short period range as determined from Section 11.4.5 or 11.4.8;
R = the response modification factor in Table 12.2-1; and
I_e = the Importance Factor determined in accordance with Section 11.5.1.

The value of C_s computed in accordance with Eq. (12.8-2) need not exceed the following:

for $T \le T_L$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \tag{12.8-3}$$

for $T > T_L$

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} \tag{12.8-4}$$

C_s shall not be less than

$$C_s = 0.044 S_{DS} I_e \ge 0.01 \tag{12.8-5}$$

In addition, for structures located where S_1 is equal to or greater than $0.6g$, C_s shall not be less than

$$C_s = 0.5 S_1/(R/I_e) \tag{12.8-6}$$

where I_e and R are as defined in this section, and

S_{D1} = the design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 11.4.5 or 11.4.6;
T = the fundamental period of the structure(s) determined in Section 12.8.2;
T_L = long-period transition period(s) determined in Section 11.4.6; and
S_1 = the mapped maximum considered earthquake spectral response acceleration parameter determined in accordance with Section 11.4.2 or 11.4.4.

12.8.1.2 Soil–Structure Interaction Reduction. A soil–structure interaction reduction is permitted where determined using Chapter 19 or other generally accepted procedures approved by the authority having jurisdiction.

12.8.1.3 Maximum S_{DS} Value in Determination of C_s and E_v. The values of C_s and E_v are permitted to be calculated using a value of S_{DS} equal to 1.0, but not less than 70% of S_{DS}, as defined in Section 11.4.5, provided that all of the following criteria are met:

1. The structure does not have irregularities, as defined in Section 12.3.2;
2. The structure does not exceed five stories above the lower of the base or grade plane as defined in Section 11.2. Where present, each mezzanine level shall be considered a story for the purposes of this limit;

3. The structure has a fundamental period, T, that does not exceed 0.5 s, as determined using Section 12.8.2;
4. The structure meets the requirements necessary for the redundancy factor, ρ, to be permitted to be taken as 1.0, in accordance with Section 12.3.4.2;
5. The site soil properties are not classified as Site Class E or F, as defined in Section 11.4.3; and
6. The structure is classified as Risk Category I or II, as defined in Section 1.5.1.

12.8.2 Period Determination. The fundamental period of the structure, T, in the direction under consideration shall be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The fundamental period, T, shall not exceed the product of the coefficient for upper limit on calculated period (C_u) from Table 12.8-1 and the approximate fundamental period, T_a, determined in accordance with Section 12.8.2.1. As an alternative to performing an analysis to determine the fundamental period, T, it is permitted to use the approximate building period, T_a, calculated in accordance with Section 12.8.2.1, directly.

12.8.2.1 Approximate Fundamental Period. The approximate fundamental period (T_a), in seconds, shall be determined from the following equation:

$$T_a = C_t h_n^x \tag{12.8-7}$$

where h_n is the structural height as defined in Section 11.2 and the coefficients C_t and x are determined from Table 12.8-2.

Alternatively, it is permitted to determine the approximate fundamental period (T_a), in seconds, from the following equation for structures not exceeding 12 stories above the base as defined in Section 11.2 where the seismic force-resisting system consists entirely of concrete or steel moment-resisting frames and the average story height is at least 10 ft (3 m):

$$T_a = 0.1N \tag{12.8-8}$$

where N = number of stories above the base.

The approximate fundamental period, T_a, in seconds, for masonry or concrete shear wall structures not exceeding 120 ft (36.6 m) in height is permitted to be determined from Eq. (12.8-9) as follows:

$$T_a = \frac{C_q}{\sqrt{C_w}} h_n \tag{12.8-9}$$

where

C_q = 0.0019 ft (0.00058 m)
C_w is calculated from Eq. (12.8-10) as follows:

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \frac{A_i}{\left[1 + 0.83\left(\frac{h_n}{D_i}\right)^2\right]} \tag{12.8-10}$$

where

A_B = area of base of structure [ft^2 (m^2)];
A_i = web area of shear wall i [ft^2 (m^2)];
D_i = length of shear wall i [ft (m)]; and
x = number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

Table 12.8-1 Coefficient for Upper Limit on Calculated Period

Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤0.1	1.7

Table 12.8-2 Values of Approximate Period Parameters C_t and x

Structure Type	C_t	x
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces:		
Steel moment-resisting frames	0.028 (0.0724)[a]	0.8
Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
Steel eccentrically braced frames in accordance with Table 12.2-1 lines B1 or D1	0.03 (0.0731)[a]	0.75
Steel buckling-restrained braced frames	0.03 (0.0731)[a]	0.75
All other structural systems	0.02 (0.0488)[a]	0.75

[a]Metric equivalents are shown in parentheses.

12.8.3 Vertical Distribution of Seismic Forces. The lateral seismic force (F_x) (kip or kN) induced at any level shall be determined from the following equations:

$$F_x = C_{vx} V \tag{12.8-11}$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{12.8-12}$$

where

C_{vx} = vertical distribution factor;
V = total design lateral force or shear at the base of the structure [kip (kN)];
w_i and w_x = portion of the total effective seismic weight of the structure (W) located or assigned to level i or x;
h_i and h_x = height [ft (m)] from the base to level i or x; and
k = an exponent related to the structure period as follows:

- for structures that have a period of 0.5 s or less, $k = 1$;
- for structures that have a period of 2.5 s or more, $k = 2$; and
- for structures that have a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2.

12.8.4 Horizontal Distribution of Forces. The seismic design story shear in any story (V_x) [kip (kN)] shall be determined from the following equation:

$$V_x = \sum_{i=x}^{n} F_i \tag{12.8-13}$$

where F_i = the portion of the seismic base shear (V) [kip (kN)] induced at level i.

The seismic design story shear (V_x) [kip (kN)] shall be distributed to the various vertical elements of the seismic force-resisting system in the story under consideration based on the relative lateral stiffness of the vertical resisting elements and the diaphragm.

12.8.4.1 Inherent Torsion. For diaphragms that are not flexible, the distribution of lateral forces at each level shall consider the effect of the inherent torsional moment, M_t, resulting from eccentricity between the locations of the center of mass and the center of rigidity. For flexible diaphragms, the distribution of forces to the vertical elements shall account for the position and distribution of the masses supported.

12.8.4.2 Accidental Torsion. Where diaphragms are not flexible, the design shall include the inherent torsional moment (M_t) resulting from the location of the structure masses plus the accidental torsional moments (M_{ta}) caused by assumed displacement of the center of mass each way from its actual location by a distance equal to 5% of the dimension of the structure perpendicular to the direction of the applied forces.

Where earthquake forces are applied concurrently in two orthogonal directions, the required 5% displacement of the center of mass need not be applied in both of the orthogonal directions at the same time but shall be applied in the direction that produces the greater effect.

Accidental torsion shall be applied to all structures for determination if a horizontal irregularity exists as specified in Table 12.3-1. Accidental torsion moments (M_{ta}) need not be included when determining the seismic forces E in the design of the structure and in the determination of the design story drift in Sections 12.8.6, 12.9.1.2, or Chapter 16, or limits of Section 12.12.1, except for the following structures:

1. Structures assigned to Seismic Category B with Type 1b horizontal structural irregularity.
2. Structures assigned to Seismic Category C, D, E, and F with Type 1a or Type 1b horizontal structural irregularity.

12.8.4.3 Amplification of Accidental Torsional Moment. Structures assigned to Seismic Design Category C, D, E, or F, where Type 1a or 1b torsional irregularity exists as defined in Table 12.3-1 shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor (A_x) as illustrated in Fig. 12.8-1 and determined from the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad (12.8\text{-}14)$$

where

δ_{max} = maximum displacement at level x computed assuming $A_x = 1$ [in. (mm)], and

δ_{avg} = average of the displacements at the extreme points of the structure at level x computed assuming $A_x = 1$ [in. (mm)].

The torsional amplification factor (A_x) shall not be less than 1 and is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

12.8.5 Overturning. The structure shall be designed to resist overturning effects caused by the seismic forces determined in Section 12.8.3.

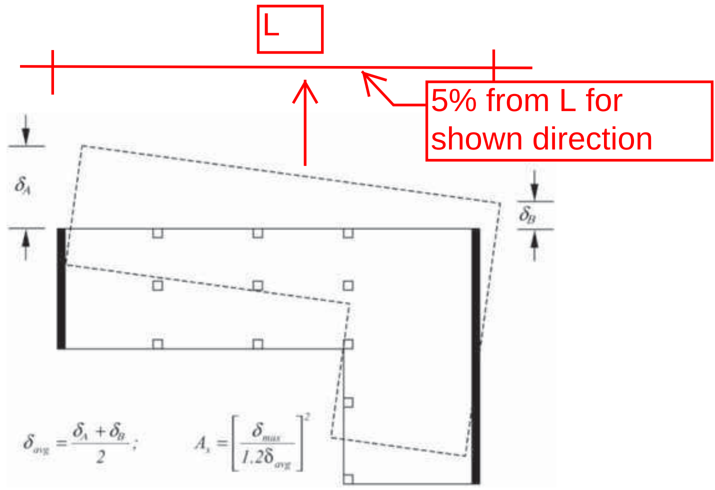
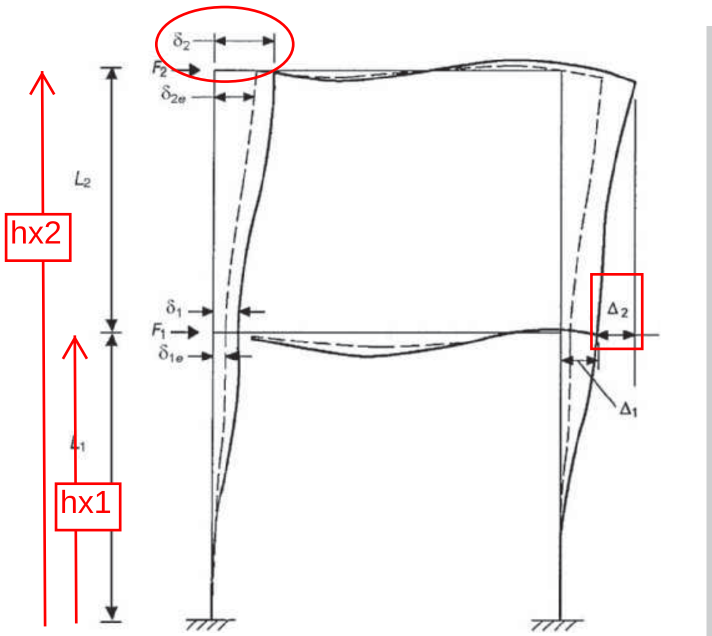


FIGURE 12.8-1 Torsional Amplification Factor, A_x



Note: Δ_i = story drift; Δ_i/L_i = story drift ratio; δ_x = total displacement; i = level under consideration.
Story Level 1: F_1 = strength-level design earthquake force; δ_{1e} = elastic displacement computed under strength-level design earthquake forces; $\delta_1 = C_d\delta_{1e}/I_E$ = amplified displacement; $\Delta_1 = \delta_1 \le \Delta_a$ (Table 12.12-1).
Story Level 2: F_2 = strength-level design earthquake force; δ_{2e} = elastic displacement computed under strength-level design earthquake forces; $\delta_2 = C_d\delta_{2e}/I_E$ = amplified displacement; $\Delta_2 = C_d(\delta_{2e} - \delta_{1e})/I_E \le \Delta_a$ (Table 12.12-1).

FIGURE 12.8-2 Story Drift Determination

12.8.6 Story Drift Determination. The design story drift (Δ) shall be computed as the difference of the deflections at the centers of mass at the top and bottom of the story under consideration (Fig. 12.8-2). Where centers of mass do not align vertically, it is permitted to compute the deflection at the bottom of the story based on the vertical projection of the center of mass at the top of the story. Where allowable stress design is used, Δ shall be computed using the strength level seismic forces specified in Section 12.8 without reduction for allowable stress design.

For structures assigned to Seismic Design Category C, D, E, or F that have horizontal irregularity Type 1a or 1b of Table 12.3-1, the design story drift, Δ, shall be computed as the largest difference of the deflections of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure.

The deflection at level x (δ_x) (in. or mm) used to compute the design story drift, Δ, shall be determined in accordance with the following equation:

$$\boxed{\delta_x = \frac{C_d \delta_{xe}}{I_e}} \qquad (12.8\text{-}15)$$

where

C_d = deflection amplification factor in Table 12.2-1;
δ_{xe} = deflection at the location required by this section determined by an elastic analysis; and
I_e = Importance Factor determined in accordance with Section 11.5.1.

12.8.6.1 Minimum Base Shear for Computing Drift. The elastic analysis of the seismic force-resisting system for computing drift shall be made using the prescribed seismic design forces of Section 12.8.

EXCEPTION: Eq. (12.8-5) need not be considered for computing drift.

12.8.6.2 Period for Computing Drift. For determining compliance with the story drift limits of Section 12.12.1, it is permitted to determine the elastic drifts (δ_{xe}) using seismic design forces based on the computed fundamental period of the structure without the upper limit ($C_u T_a$) specified in Section 12.8.2.

12.8.7 P-Delta Effects. 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 where the stability coefficient (θ) as determined by the following equation is equal to or less than 0.10:

$$\boxed{\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d}} \qquad (12.8\text{-}16)$$

where

P_x = total vertical design load at and above level x [kip (kN)]; where computing P_x, no individual load factor need exceed 1.0;
Δ = design story drift as defined in Section 12.8.6 occurring simultaneously with V_x [in. (mm)];
I_e = Importance Factor determined in accordance with Section 11.5.1;
V_x = seismic shear force acting between levels x and $x - 1$ [kip (kN)];
h_{sx} = story height below level x [in. (mm)]; and
C_d = deflection amplification factor in Table 12.2-1.

The stability coefficient (θ) shall not exceed θ_{max}, determined as follows:

$$\boxed{\theta_{max} = \frac{0.5}{\beta C_d} \le 0.25} \qquad (12.8\text{-}17)$$

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

Where the stability coefficient (θ) is greater than 0.10 but less than or equal to θ_{max}, the incremental factor related to P-delta effects on displacements and member forces shall be determined by rational analysis. Alternatively, it is permitted to multiply displacements and member forces by $1.0/(1 - \theta)$.

Where θ is greater than θ_{max}, the structure is potentially unstable and shall be redesigned.

Where the P-delta effect is included in an automated analysis, Eq. (12.8-17) shall still be satisfied; however, the value of θ computed from Eq. (12.8-16) using the results of the P-delta analysis is permitted to be divided by $(1 + \theta)$ before checking Eq. (12.8-17).

12.9 LINEAR DYNAMIC ANALYSIS

12.9.1 Modal Response Spectrum Analysis

12.9.1.1 Number of Modes. An analysis shall be conducted to determine the natural modes of vibration for the structure. The analysis shall include a sufficient number of mo[illegible] combined modal mass participation of 100% o[illegible] mass. For this purpose, it shall be permitted [illegible] modes with periods less than 0.05 s in a single r[illegible] that has a period of 0.05 s.

summation of
effective modal
mass ratios

EXCEPTION: Alternatively, the analysis shall be permitted to include a minimum number of modes to obtain a combined modal mass participation of at least 90% of the actual mass in each orthogonal horizontal direction of response considered in the model.

12.9.1.2 Modal Response Parameters. The value for each force-related design parameter of interest, including story drifts, support forces, and individual member forces for each mode of response, shall be computed using the properties of each mode and the response spectra defined in either Section 11.4.6 or 21.2 divided by the quantity R/I_e. The value for displacement and drift quantities shall be multiplied by the quantity C_d/I_e.

12.9.1.3 Combined Response Parameters. The value for each parameter of interest calculated for the various modes shall be combined using the square root of the sum of the squares (SRSS) method, the complete quadratic combination (CQC) method, the complete quadratic combination method as modified by ASCE 4 (CQC-4), or an approved equivalent approach. The CQC or the CQC-4 method shall be used for each of the modal values where closely spaced modes have significant cross-correlation of translational and torsional response.

12.9.1.4 Scaling Design Values of Combined Response. A base shear (V) shall be calculated in each of the two orthogonal horizontal directions using the calculated fundamental period of the structure T in each direction and the procedures of Section 12.8.

12.9.1.4.1 Scaling of Forces. Where the calculated fundamental period exceeds $C_u T_a$ in a given direction, $C_u T_a$ shall be used in lieu of T in that direction. Where the combined response for the modal base shear (V_t) is less than 100% of the calculated base shear (V) using the equivalent lateral force procedure, the forces shall be multiplied by V/V_t where

V = the equivalent lateral force procedure base shear, calculated in accordance with this section and Section 12.8, and
V_t = the base shear from the required modal combination.

12.9.1.4.2 Scaling of Drifts. Where the combined response for the modal base shear (V_t) is less than $C_s W$, and where C_s is

Cs=0.5*S1/R

determined in accordance with Eq. (12.8-6), drifts shall be multiplied by C_sW/V_t.

12.9.1.5 Horizontal Shear Distribution. The distribution of horizontal shear shall be in accordance with Section 12.8.4, except that amplification of torsion in accordance with Section 12.8.4.3 is not required where accidental torsion effects are included in the dynamic analysis model.

12.9.1.6 P-Delta Effects. The P-delta effects shall be determined in accordance with Section 12.8.7. The base shear used to determine the story shears and the story drifts shall be determined in accordance with Section 12.8.6.

12.9.1.7 Soil–Structure Interaction Reduction. A soil–structure interaction reduction is permitted where determined using Chapter 19 or other generally accepted procedures approved by the authority having jurisdiction.

12.9.1.8 Structural Modeling. A mathematical model of the structure shall be constructed in accordance with Section 12.7.3, except that all structures designed in accordance with this section shall be analyzed using a 3D representation. Where the diaphragms have not been classified as rigid in accordance with Section 12.3.1, the model shall include representation of the diaphragm's stiffness characteristics and additional dynamic degrees of freedom as required to account for the participation of the diaphragm in the structure's dynamic response.

12.9.2 Linear Response History Analysis

12.9.2.1 General Requirements. Linear response history analysis shall consist of an analysis of a linear mathematical model of the structure to determine its response through methods of numerical integration, to suites of spectrally matched acceleration histories compatible with the design response spectrum for the site. The analysis shall be performed in accordance with the requirements of this section.

12.9.2.2 General Modeling Requirements. Three-dimensional (3D) models of the structure shall be required. Modeling the distribution of stiffness and mass throughout the structure's lateral load-resisting system and diaphragms shall be in accordance with Section 12.7.3.

12.9.2.2.1 P-Delta Effects. The mathematical model shall include P-delta effects. Limits on the stability coefficient, θ, shall be satisfied in accordance with Section 12.8.7.

12.9.2.2.2 Accidental Torsion. Accidental torsion, where required by Section 12.8.4.2, shall be included by offsetting the center of mass in each direction (i.e., plus or minus) from its expected location by a distance equal to 5% of the horizontal dimension of the structure at the given floor measured perpendicular to the direction of loading. Amplification of accidental torsion in accordance with Section 12.8.4.3 is not required.

12.9.2.2.3 Foundation Modeling. Where foundation flexibility is included in the analysis, modeling of the foundation shall be in accordance with Section 12.13.3.

12.9.2.2.4 Number of Modes to Include in Modal Response History Analysis. Where the modal response history analysis procedure is used, the number of modes to include in the analysis shall be in accordance with Section 12.9.1.1.

12.9.2.2.5 Damping. Linear viscous damping shall not exceed 5% critical for any mode with a vibration period greater than or equal to T_{lower}.

12.9.2.3 Ground Motion Selection and Modification. Ground acceleration histories used for analysis shall consist of a suite of no fewer than three pairs of spectrally matched orthogonal components derived from artificial or recorded ground motion events. The target response spectrum for each spectrally matched set shall be developed in accordance with Sections 11.4.6 or 21.3, as applicable.

12.9.2.3.1 Procedure for Spectrum Matching. Each component of ground motion shall be spectrally matched over the period range $0.8T_{\text{lower}}$ to $1.2T_{\text{upper}}$. Over the same period range and in each direction of response, the average of the 5% damped pseudoacceleration ordinates computed using the spectrum-matched records shall not fall above or below the target spectrum by more than 10% in each direction of response.

12.9.2.4 Application of Ground Acceleration Histories. Two orthogonal directions of response, designated as X and Y, shall be selected and used for all response history analysis. Ground motions shall be applied independently in the X and Y directions.

12.9.2.5 Modification of Response for Design

12.9.2.5.1 Determination of Maximum Elastic and Inelastic Base Shear. For each ground motion analyzed, a maximum elastic base shear, designated as V_{EX} and V_{EY} in the X and Y directions, respectively, shall be determined. The mathematical model used for computing the maximum elastic base shear shall not include accidental torsion.

For each ground motion analyzed, a maximum inelastic base shear, designated as V_{IX} and V_{IY} in the X and Y directions, respectively, shall be determined as follows:

$$V_{IX} = \frac{V_{EX}I_e}{R_X} \tag{12.9-1}$$

$$V_{IY} = \frac{V_{EY}I_e}{R_Y} \tag{12.9-2}$$

where I_e is the Importance Factor and R_X and R_Y are the response modifications coefficients for the X and Y directions, respectively.

12.9.2.5.2 Determination of Base Shear Scale Factor. Design base shears, V_X, and V_Y, shall be computed in the X and Y directions, respectively, in accordance with Section 12.8.1. For each ground motion analyzed, base shear scale factors in each direction of response shall be determined as follows:

$$\eta_X = \frac{V_X}{V_{IX}} \geq 1.0 \tag{12.9-3}$$

$$\eta_Y = \frac{V_Y}{V_{IY}} \geq 1.0 \tag{12.9-4}$$

12.9.2.5.3 Determination of Combined Force Response. For each direction of response and for each ground motion analyzed, the combined force response shall be determined as follows:

a. The combined force response in the X direction shall be determined as $I_e\eta_X/R_X$ times the computed elastic response in the X direction using the mathematical model with accidental torsion (where required) plus $I_e\eta_Y/R_Y$ times the computed elastic response in the Y direction using the mathematical model without accidental torsion.
b. The combined force response in the Y direction shall be determined as $I_e\eta_Y/R_Y$ times the computed elastic response in the Y direction using the mathematical model with accidental torsion (where required), plus $I_e\eta_X/R_X$ times

the computed elastic response in the X direction using the mathematical model without accidental torsion.

12.9.2.5.4 Determination of Combined Displacement Response. Response modification factors C_{dX} and C_{dY} shall be assigned in the X and Y directions, respectively. For each direction of response and for each ground motion analyzed, the combined displacement responses shall be determined as follows:

a. The combined displacement response in the X direction shall be determined as $\eta_X C_{dX}/R_X$ times the computed elastic response in the X direction using the mathematical model with accidental torsion (where required), plus $\eta_Y C_{dY}/R_Y$ times the computed elastic response in the Y direction using the mathematical model without accidental torsion.
b. The combined displacement response in the Y direction shall be determined as $\eta_Y C_{dY}/R_Y$ times the computed elastic response in the Y direction using the mathematical model with accidental torsion (where required), plus $\eta_X C_{dX}/R_X$ times the computed elastic response in the X direction using the mathematical model without accidental torsion.

EXCEPTION: Where the design base shear in the given direction is not controlled by Eq. (12.8-6), the factors η_X or η_Y, as applicable, are permitted to be taken as 1.0 for the purpose of determining combined displacements.

12.9.2.6 Enveloping of Force Response Quantities. Design force response quantities shall be taken as the envelope of the combined force response quantities computed in both orthogonal directions and for all ground motions considered. Where force interaction effects are considered, demand to capacity ratios are permitted to be enveloped in lieu of individual force quantities.

12.9.2.7 Enveloping of Displacement Response Quantities. Story drift quantities shall be determined for each ground motion analyzed and in each direction of response using the combined displacement responses defined in Section 12.9.2.5.4. For the purpose of complying with the drift limits specified in Section 12.12, the envelope of story drifts computed in both orthogonal directions and for all ground motions analyzed shall be used.

12.10 DIAPHRAGMS, CHORDS, AND COLLECTORS

Diaphragms, chords, and collectors shall be designed in accordance with Sections 12.10.1 and 12.10.2.

EXCEPTIONS:

1. Precast concrete diaphragms, including chords and collectors in structures assigned to Seismic Design Categories C, D, E, or F, shall be designed in accordance with Section 12.10.3.
2. Precast concrete diaphragms in Seismic Design Category B, cast-in-place concrete diaphragms, and wood-sheathed diaphragms supported by wood diaphragm framing are permitted to be designed in accordance with Section 12.10.3.

12.10.1 Diaphragm Design. Diaphragms shall be designed for both the shear and bending stresses resulting from design forces. At diaphragm discontinuities, such as openings and reentrant corners, the design shall ensure that the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm is within shear and tension capacity of the diaphragm.

12.10.1.1 Diaphragm Design Forces. Floor and roof diaphragms shall be designed to resist design seismic forces from the structural analysis but shall not be less than that determined in accordance with Eq. (12.10-1) as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (12.10\text{-}1)$$

where

F_{px} = the diaphragm design force at level x;
F_i = the design force applied to level i;
w_i = the weight tributary to level i; and
w_{px} = the weight tributary to the diaphragm at level x.

The force determined from Eq. (12.10-1) shall not be less than

$$F_{px} = 0.2 S_{DS} I_e w_{px} \qquad (12.10\text{-}2)$$

The force determined from Eq. (12.10-1) need not exceed

$$F_{px} = 0.4 S_{DS} I_e w_{px} \qquad (12.10\text{-}3)$$

All diaphragms shall be designed for the inertial forces determined from Eqs. (12.10-1) through (12.10-3) and for all applicable transfer forces. For structures that have a horizontal structural irregularity of Type 4 in Table 12.3-1, the transfer forces from the vertical seismic force-resisting elements above the diaphragm to other vertical seismic force-resisting elements below the diaphragm shall be increased by the overstrength factor of Section 12.4.3 before being added to the diaphragm inertial forces. For structures that have horizontal or vertical structural irregularities of the types indicated in Section 12.3.3.4, the requirements of that section shall also apply.

EXCEPTION: One- and two-family dwellings of light-frame construction shall be permitted to use $\Omega_0 = 1.0$.

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

12.10.2.1 Collector Elements Requiring Load Combinations Including Overstrength for Seismic Design Categories C through F. In structures assigned to Seismic Design Category C, D, E, or F, collector elements (Fig. 12.10-1) and their

chord element

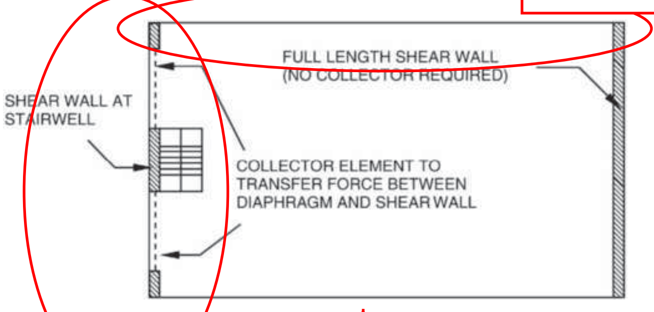


FIGURE 12.10-1 Collectors

connections, including connections to vertical elements, shall be designed to resist the maximum of the following:

1. Forces calculated using the seismic load effects including overstrength of Section 12.4.3 with seismic forces determined by the equivalent lateral force procedure of Section 12.8 or the modal response spectrum analysis procedure of Section 12.9.1;
2. Forces calculated using the seismic load effects including overstrength of Section 12.4.3 with seismic forces determined by Eq. (12.10-1); and
3. Forces calculated using the load combinations of Section 2.3.6 with seismic forces determined by Eq. (12.10-2).

Transfer forces as described in Section 12.10.1.1 shall be considered.

EXCEPTION:

1. In structures or portions thereof braced entirely by wood light-frame shear walls, collector elements and their connections, including connections to vertical elements, need only be designed to resist forces using the load combinations of Section 2.3.6 with seismic forces determined in accordance with Section 12.10.1.1.

12.10.3 Alternative Design Provisions for Diaphragms, Including Chords and Collectors. Where required or permitted in Section 12.10, diaphragms, including chords and collectors, shall be designed using the provisions in Section 12.10.3.1 through 12.10.3.5 and the following:

1. Footnote *b* to Table 12.2-1 shall not apply.
2. Section 12.3.3.4 shall not apply.
3. Section 12.3.4.1, Item 5, shall be replaced with the following: "Design of diaphragms, including chords, collectors, and their connections to the vertical elements" are used.
4. Section 12.3.4.1, Item 7, shall not apply.

12.10.3.1 Design. Diaphragms, including chords, collectors, and their connections to the vertical elements, shall be designed in two orthogonal directions to resist the in-plane design seismic forces determined in Section 12.10.3.2. Collectors shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the vertical elements providing the resistance to those forces. Design shall provide for transfer of forces at diaphragm discontinuities, such as openings and reentrant corners.

12.10.3.2 Seismic Design Forces for Diaphragms, Including Chords and Collectors. Diaphragms, including chords, collectors, and their connections to the vertical elements, shall be designed to resist in-plane seismic design forces given by Eq. (12.10-4):

$$F_{px} = \frac{C_{px}}{R_s} w_{px} \tag{12.10-4}$$

The force F_{px} determined from Eq. (12.10-4) shall not be less than:

$$F_{px} = 0.2 S_{DS} I_e w_{px} \tag{12.10-5}$$

C_{px} shall be determined as illustrated in Fig. 12.10-2.

12.10.3.2.1 Design Acceleration Coefficients C_{p0}, C_{pi}, and C_{pn}. Design acceleration coefficients C_{p0} and C_{pn} shall be calculated by Eqs. (12.10-6) and (12.10-7):

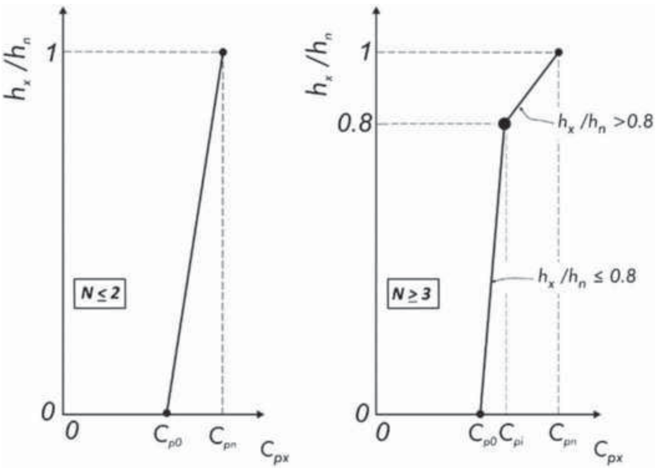


FIGURE 12.10-2 Calculating the Design Acceleration Coefficient C_{px} in Buildings with $N \le 2$ and in Buildings with $N \ge 3$

$$C_{p0} = 0.4 S_{DS} I_e \tag{12.10-6}$$

and

$$C_{pn} = \sqrt{(\Gamma_{m1}\Omega_0 C_s)^2 + (\Gamma_{m2} C_{s2})^2} \ge C_{pi} \tag{12.10-7}$$

Design acceleration coefficient C_{pi} shall be the greater of values given by Eqs. (12.10-8) and (12.10-9):

$$C_{pi} = 0.8 C_{p0} \tag{12.10-8}$$

$$C_{pi} = 0.9 \Gamma_{m1} \Omega_0 C_s \tag{12.10-9}$$

where Ω_0 is the overstrength factor given in Table 12.2-1, C_s is determined in accordance with Section 12.8 or 12.9, and C_{s2} shall be the smallest of values calculated from Eqs. (12.10-10), (12.10-11), and (12.10-12):

$$C_{s2} = (0.15N + 0.25) I_e S_{DS} \tag{12.10-10}$$

$$C_{s2} = I_e S_{DS} \tag{12.10-11}$$

$$\text{For } N \ge 2 \qquad C_{s2} = \frac{I_e S_{D1}}{0.03(N-1)} \tag{12.10-12a}$$

$$\text{For } N = 1 \qquad C_{s2} = 0 \tag{12.10-12b}$$

The modal contribution factors Γ_{m1} and Γ_{m2} in Eq. (12.10-7) shall be calculated from Eqs. (12.10-13) and (12.10-14):

$$\Gamma_{m1} = 1 + \frac{z_s}{2}\left(1 - \frac{1}{N}\right) \tag{12.10-13}$$

and

$$\Gamma_{m2} = 0.9 z_s \left(1 - \frac{1}{N}\right)^2 \tag{12.10-14}$$

where the mode shape factor z_s is to be taken as

- 0.3 for buildings designed with buckling restrained braced frame systems defined in Table 12.2-1, or
- 0.7 for buildings designed with moment-resisting frame systems defined in Table 12.2-1, or
- 0.85 for buildings designed with dual systems defined in Table 12.2-1 with special or intermediate moment frames capable of resisting at least 25% of the prescribed seismic forces, or
- 1.0 for buildings designed with all other seismic force-resisting systems.

12.10.3.3 Transfer Forces in Diaphragms. All diaphragms shall be designed for the inertial forces determined from Eqs. (12.10-4) and (12.10-5) and for all applicable transfer forces. For structures that have a horizontal structural irregularity of Type 4 in Table 12.3-1, the transfer forces from the vertical seismic force-resisting elements above the diaphragm to other vertical seismic force-resisting elements below the diaphragm shall be increased by the overstrength factor of Section 12.4.3 before being added to the diaphragm inertial forces. For structures that have other horizontal or vertical structural irregularities of the types indicated in Section 12.3.3.4, the requirements of that section shall apply.

EXCEPTION: One- and two-family dwellings of light-frame construction shall be permitted to use $\Omega_0 = 1.0$.

12.10.3.4 Collectors—Seismic Design Categories C through F. In structures assigned to Seismic Design Category C, D, E, or F, collectors and their connections, including connections to vertical elements, shall be designed to resist 1.5 times the diaphragm inertial forces from Section 12.10.3.2 plus 1.5 times the design transfer forces.

EXCEPTIONS:

1. Any transfer force increased by the overstrength factor of Section 12.4.3 need not be further amplified by 1.5.
2. For moment frame and braced frame systems, collector forces need not exceed the lateral strength of the corresponding frame line below the collector, considering only the moment frames or braced frames. In addition, diaphragm design forces need not exceed the forces corresponding to the collector forces so determined.
3. In structures or portions thereof braced entirely by light-frame shear walls, collector elements and their connections, including connections to vertical elements, need only be designed to resist the diaphragm seismic design forces without the 1.5 multiplier.

12.10.3.5 Diaphragm Design Force Reduction Factor. The diaphragm design force reduction factor, R_s, shall be determined in accordance with Table 12.10-1.

Table 12.10-1 Diaphragm Design Force Reduction Factor, R_s

Diaphragm System		Shear-Controlled	Flexure-Controlled
Cast-in-place concrete designed in accordance with Section 14.2 and ACI 318	—	1.5	2
Precast concrete designed in accordance with Section 14.2.4 and ACI 318	EDO[a]	0.7	0.7
	BDO[b]	1.0	1.0
	RDO[c]	1.4	1.4
Wood sheathed designed in accordance with Section 14.5 and AWC SDPWS-15	—	3.0	NA

[a]EDO is precast concrete diaphragm elastic design option.
[b]BDO is precast concrete diaphragm basic design option.
[c]RDO is precast concrete diaphragm reduced design option.

12.11 STRUCTURAL WALLS AND THEIR ANCHORAGE

12.11.1 Design for Out-of-Plane Forces. Structural walls shall be designed for a force normal to the surface equal to $F_p = 0.4S_{DS}I_e$ times the weight of the structural wall with a minimum force of 10% of the weight of the structural wall.

12.11.2 Anchorage of Structural Walls and Transfer of Design Forces into Diaphragms or Other Supporting Structural Elements

12.11.2.1 Wall Anchorage Forces. The anchorage of structural walls to supporting construction shall provide a direct connection capable of resisting the following:

$$F_p = 0.4S_{DS}k_aI_eW_p \qquad (12.11\text{-}1)$$

F_p shall not be taken as less than $0.2k_aI_eW_p$.

$$k_a = 1.0 + \frac{L_f}{100} \qquad (12.11\text{-}2)$$

k_a need not be taken as larger than 2.0.
k_a need not be taken as larger than 1.0 when the connection is not at a flexible diaphragm.

where

F_p = the design force in the individual anchors;
S_{DS} = the design spectral response acceleration parameter at short periods per Section 11.4.5;
I_e = the Importance Factor determined in accordance with Section 11.5.1;
k_a = amplification factor for diaphragm flexibility;
L_f = the span, in feet, of a flexible diaphragm that provides the lateral support for the wall; the span is measured between vertical elements that provide lateral support to the diaphragm in the direction considered; use zero for rigid diaphragms; and
W_p = the weight of the wall tributary to the anchor.

Where the anchorage is not located at the roof and all diaphragms are not flexible, the value from Eq. (12.11-1) is permitted to be multiplied by the factor $(1 + 2z/h)/3$, where z is the height of the anchor above the base of the structure and h is the height of the roof above the base; however, F_p shall not be less than required by Section 12.11.2 with a minimum anchorage force of $F_p = 0.2W_p$.

Structural walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 ft (1,219 mm). Interconnection of structural wall elements and connections to supporting framing systems shall have sufficient strength, rotational capacity, and ductility to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces.

12.11.2.2 Additional Requirements for Anchorage of Concrete or Masonry Structural Walls to Diaphragms in Structures Assigned to Seismic Design Categories C through F

12.11.2.2.1 Transfer of Anchorage Forces into Diaphragm. Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. Diaphragm connections shall be positive, mechanical, or welded. Added chords are permitted to be used to form subdiaphragms to transmit the anchorage forces to the main continuous crossties. The maximum length-to-width ratio of structural subdiaphragms that serve as part of the continuous tie system shall be 2.5 to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

12.11.2.2.2 Steel Elements of Structural Wall Anchorage System. The strength design forces for steel elements of the structural wall anchorage system, with the exception of anchor bolts and reinforcing steel, shall be increased by 1.4 times the forces otherwise required by this section.

12.11.2.2.3 Wood Diaphragms. The anchorage of concrete or masonry structural walls to wood diaphragms shall be in accordance with AWC SDPWS 4.1.5.1 and this section. Continuous ties required by this section shall be in addition to the diaphragm sheathing. Anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension. The diaphragm sheathing shall not be considered effective for providing the ties or struts required by this section.

12.11.2.2.4 Metal Deck Diaphragms. In metal deck diaphragms, the metal deck shall not be used as the continuous ties required by this section in the direction perpendicular to the deck span.

12.11.2.2.5 Embedded Straps. Diaphragm to structural wall anchorage using embedded straps shall be attached to, or hooked around, the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

12.11.2.2.6 Eccentrically Loaded Anchorage System. Where elements of the wall anchorage system are loaded eccentrically or are not perpendicular to the wall, the system shall be designed to resist all components of the forces induced by the eccentricity.

12.11.2.2.7 Walls with Pilasters. Where pilasters are present in the wall, the anchorage force at the pilasters shall be calculated considering the additional load transferred from the wall panels to the pilasters. However, the minimum anchorage force at a floor or roof shall not be reduced.

12.12 DRIFT AND DEFORMATION

12.12.1 Story Drift Limit. The design story drift (Δ) as determined in Sections 12.8.6, 12.9.1, or 12.9.2 shall not exceed the allowable story drift (Δ_a) as obtained from Table 12.12-1 for any story.

12.12.1.1 Moment Frames in Structures Assigned to Seismic Design Categories D through F. For seismic force-resisting systems solely comprising moment frames in structures assigned to Seismic Design Categories D, E, or F, the design story drift (Δ) shall not exceed Δ_a/ρ for any story. ρ shall be determined in accordance with Section 12.3.4.2.

12.12.2 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.

12.12.3 Structural Separation. All portions of the structure shall be designed and constructed to act as an integral unit in resisting seismic forces unless separated structurally by a distance sufficient to avoid damaging contact as set forth in this section.

Separations shall allow for the maximum inelastic response displacement (δ_M). δ_M shall be determined at critical locations with consideration for translational and torsional displacements of the structure including torsional amplifications, where applicable, using the following equation:

$$\delta_M = \frac{C_d \delta_{max}}{I_e} \qquad (12.12\text{-}1)$$

where δ_{max} = maximum elastic displacement at the critical location.

Table 12.12-1 Allowable Story Drift, Δ_a[a,b]

Structure	Risk Category I or II	Risk Category III	Risk Category IV
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below level x.
[b] For seismic force-resisting systems solely comprising moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] 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 story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are so constructed that moment transfer between shear walls (coupling) is negligible.

Adjacent structures on the same property shall be separated by at least δ_{MT}, determined as follows:

$$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2} \quad (12.12\text{-}2)$$

where δ_{M1} and δ_{M2} are the maximum inelastic response displacements of the adjacent structures at their adjacent edges.

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

EXCEPTION: Smaller separations or property line setbacks are permitted where justified by rational analysis based on inelastic response to design ground motions.

12.12.4 Members Spanning between Structures. Gravity connections or supports for members spanning between structures or seismically separate portions of structures shall be designed for the maximum anticipated relative displacements. These displacements shall be calculated as follows:

1. Using the deflection calculated at the locations of support, per Eq. (12.8-15) multiplied by $1.5R/C_d$,
2. Considering additional deflection caused by diaphragm rotation including the torsional amplification factor calculated per Section 12.8.4.3 where either structure is torsionally irregular,
3. Considering diaphragm deformations, and
4. Assuming that the two structures are moving in opposite directions and using the absolute sum of the displacements.

12.12.5 Deformation Compatibility for Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E, or F, 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 caused by the design story drift (Δ) as determined in accordance with Section 12.8.6 (see also Section 12.12.1).

EXCEPTION: Reinforced concrete frame members not designed as part of the seismic force-resisting system shall comply with Section 18.14 of ACI 318.

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.

12.13 FOUNDATION DESIGN

12.13.1 Design Basis. The design basis for foundations shall be as set forth in Section 12.1.5.

12.13.2 Materials of Construction. Materials used for the design and construction of foundations shall comply with the requirements of Chapter 14 and the additional requirements of Section 12.13.9 for foundations on liquefiable sites. Design and detailing of steel piles shall comply with Section 14.1.8 and the additional requirements for Section 12.13.9 where applicable. Design and detailing of concrete piles shall comply with Section 14.2.3 and the additional requirements for Section 12.13.9 where applicable.

12.13.3 Foundation Load-Deformation Characteristics. Where foundation flexibility is included for the linear analysis procedures in Chapter 12, the load-deformation characteristics of the foundation–soil system shall be modeled in accordance with the requirements of this section. The linear load-deformation behavior of foundations shall be represented by an equivalent linear stiffness using soil properties that are compatible with the soil strain levels associated with the design earthquake motion. The strain-compatible shear modulus, G, and the associated strain-compatible shear wave velocity, v_S, needed for the evaluation of equivalent linear stiffness shall be determined using the criteria in Chapter 19 or based on a site-specific study. A 50% increase and decrease in stiffness shall be incorporated in dynamic analyses unless smaller variations can be justified based on field measurements of dynamic soil properties or direct measurements of dynamic foundation stiffness. The largest values of response shall be used in design.

12.13.4 Reduction of Foundation Overturning. Overturning effects at the soil–foundation interface are permitted to be reduced by 25% for foundations of structures that satisfy both of the following conditions:

a. The structure is designed in accordance with the equivalent lateral force analysis as set forth in Section 12.8, and
b. The structure is not an inverted pendulum or cantilevered column type structure.

Overturning effects at the soil–foundation interface are permitted to be reduced by 10% for foundations of structures designed in accordance with the modal analysis requirements of Section 12.9.

12.13.5 Strength Design for Foundation Geotechnical Capacity. Where basic combinations for strength design listed in Chapter 2 are used, combinations that include earthquake loads, E, are permitted to include reduction of foundation overturning effects defined in Section 12.13.4. The following sections shall apply for determination of the applicable nominal strengths and resistance factors at the soil–foundation interface.

12.13.5.1 Nominal Strength. The nominal foundation geotechnical capacity, Q_{ns}, shall be determined using any of the following methods:

1. presumptive load-bearing values,
2. by a registered design professional based on geotechnical site investigations that include field and laboratory testing to determine soil classification and as-required active, passive, and at-rest soil strength parameters, or
3. by in situ testing of prototype foundations.

For structures that are supported on more than one foundation, the method used to determine the nominal strength of all foundations shall be the same. Nominal strength values are permitted to be based on either a limitation of maximum expected foundation deformation, or by the nominal strength that is associated with an anticipated failure mechanism.

12.13.5.1.1 Soil Strength Parameters. For competent soils that do not undergo strength degradation under seismic loading, strength parameters for static loading conditions shall be used to compute nominal foundation geotechnical capacities for seismic design unless increased seismic strength values based on site conditions are provided by a registered design professional. For sensitive cohesive soils or saturated cohesionless soils, the potential for earthquake-induced strength degradation shall be considered. Nominal foundation geotechnical capacities for vertical, lateral, and rocking loading shall be determined using accepted foundation design procedures and principles of plastic analysis, and shall be best-estimate values using soil properties that are representative average values.

Total resistance to lateral loads is permitted to be determined by taking the sum of the values derived from lateral bearing pressure plus horizontal sliding resistance (from some combination of friction and cohesion).

1. Lateral sliding resistance from friction shall be limited to sand, silty sand, clayey sand, silty gravel, and clayey gravel soils (SW, SP, SM, SC, GM and GC), and rock. Lateral sliding resistance from friction shall be calculated as the most unfavorable dead load factor multiplied by dead load, D, and multiplied by a coefficient of friction.
2. Lateral sliding resistance from cohesion shall be limited to clay, sandy clay, clayey silt, silt, and sandy silt (CL, ML, SC, and SM). Lateral sliding resistance from cohesion shall be calculated as the contact area multiplied by the cohesion.
3. Horizontal friction sliding resistance and cohesion sliding resistance shall be taken as zero for areas of foundations supported by piles.

Where presumptive load bearing values for supporting soils are used to determine nominal soil strengths, organic silt, organic clays, peat, or nonengineered fill shall not be assumed to have a presumptive load capacity.

12.13.5.2 Resistance Factors. The resistance factors prescribed in this section shall be used for vertical, lateral, and rocking resistance of all foundation types. Nominal foundation geotechnical capacities, Q_{ns}, shall be multiplied by the resistance factors (ϕ) shown in Table 12.13-1. Alternatively, a vertical resistance factor, $\phi = 0.80$ is permitted to be used when the nominal strength (upward or downward) is determined by in - situ testing of prototype foundations, based on a test program that is approved by the authority having jurisdiction.

12.13.5.3 Acceptance Criteria. For linear seismic analysis procedures in accordance with Sections 12.8 and 12.9, factored loads, including reductions permitted in Section 12.13.4, shall not exceed foundation design strengths, $\phi\ Q_{ns}$.

12.13.6 Allowable Stress Design for Foundation Geotechnical Capacity. Where basic combinations for allowable stress design listed in Section 12.4 are used for design, combinations that include earthquake loads, E, are permitted to include reduction of foundation overturning effects defined in Section 12.13.4. Allowable foundation load capacities, Q_{as}, shall be determined using allowable stresses in geotechnical materials that have been determined by geotechnical investigations required by the Authority Having Jurisdiction (AHJ).

Table 12.13-1 Resistance Factors for Strength Design of Soil–Foundation Interface

Direction and Type of Resistance	Resistance Factors, ϕ
Vertical Resistance	
Compression (bearing) strength	0.45
Pile friction (either upward or downward)	0.45
Lateral Resistance	
Lateral bearing pressure	0.5
Sliding (by either friction or cohesion)	0.85

12.13.7 Requirements for Structures Assigned to Seismic Design Category C. In addition to the requirements of Section 11.8.2, the following foundation design requirements shall apply to structures assigned to Seismic Design Category C.

12.13.7.1 Pole-Type Structures. Where construction using posts or poles as columns embedded in earth or embedded in concrete footings in the earth is used to resist lateral loads, the depth of embedment required for posts or poles to resist seismic forces shall be determined by means of the design criteria established in the foundation investigation report.

12.13.7.2 Foundation Ties. Individual pile caps, drilled piers, or caissons shall be interconnected by ties. All ties shall have a design strength in tension or compression at least equal to a force equal to 10% of S_{DS} times the larger pile cap or column factored dead plus factored live load unless it is demonstrated that equivalent restraint will be provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils, very dense granular soils, or other approved means.

12.13.7.3 Pile Anchorage Requirements. In addition to the requirements of Section 14.2.3.1, anchorage of piles shall comply with this section. Where required for resistance to uplift forces, anchorage of steel pipe [round hollow structure steel (HSS) sections], concrete-filled steel pipe, or H piles to the pile cap shall be made by means other than concrete bond to the bare steel section.

EXCEPTION: Anchorage of concrete-filled steel pipe piles is permitted to be accomplished using deformed bars developed into the concrete portion of the pile.

12.13.8 Requirements for Structures Assigned to Seismic Design Categories D through F. In addition to the requirements of Sections 11.8.2, 11.8.3, 14.1.8, and 14.2.3.2, the following foundation design requirements shall apply to structures assigned to Seismic Design Category D, E, or F. Design and construction of concrete foundation elements shall conform to the requirements of ACI 318, Section 18.9, except as modified by the requirements of this section.

EXCEPTION: Detached one- and two-family dwellings of light-frame construction not exceeding two stories above grade plane need only comply with the requirements for Sections 11.8.2, 11.8.3 (items 2 through 4), 12.13.2, and 12.13.7.

12.13.8.1 Pole-Type Structures. Where construction using posts or poles as columns embedded in earth or embedded in concrete footings in the earth is used to resist lateral loads, the depth of embedment required for posts or poles to resist seismic forces shall be determined by means of the design criteria established in the foundation investigation report.

12.13.8.2 Foundation Ties. Individual pile caps, drilled piers, or caissons shall be interconnected by ties. In addition, individual spread footings founded on soil defined in Chapter 20 as Site Class E or F shall be interconnected by ties. All ties shall have a design strength in tension or compression at least equal to a force equal to 10% of S_{DS} times the larger pile cap or column factored dead plus factored live load unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils, very dense granular soils, or other approved means.

12.13.8.3 General Pile Design Requirement. Piling shall be designed and constructed to withstand deformations from earthquake ground motions and structure response. Deformations shall include both free-field soil strains (without the structure) and deformations induced by lateral pile resistance to structure seismic forces, all as modified by soil–pile interaction.

12.13.8.4 Batter Piles. Batter piles and their connections shall be capable of resisting forces and moments from the load combinations including overstrength from Chapter 2 or Section 12.14.3.2.3. 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.

12.13.8.5 Pile Anchorage Requirements. In addition to the requirements of Section 12.13.7.3, anchorage of piles shall comply with this section. Design of anchorage of piles into the pile cap shall consider the combined effect of axial forces because of uplift and bending moments caused by fixity to the pile cap. For piles required to resist uplift forces or provide rotational restraint, anchorage into the pile cap shall comply with the following:

1. In the case of uplift, the anchorage shall be capable of developing the least of the nominal tensile strength of the longitudinal reinforcement in a concrete pile, the nominal tensile strength of a steel pile, and 1.3 times the pile pullout resistance, or shall be designed to resist the axial tension force resulting from the seismic load effects including overstrength of Section 12.4.3 or 12.14.3.2. The pile pullout resistance shall be taken as the ultimate frictional or adhesive force that can be developed between the soil and the pile plus the pile weight.
2. In the case of rotational restraint, the anchorage shall be designed to resist the axial and shear forces and moments resulting from the seismic load effects including overstrength of Section 12.4.3 or 12.14.3.2 or shall be capable of developing the full axial, bending, and shear nominal strength of the pile.

12.13.8.6 Splices of Pile Segments. Splices of pile segments shall develop the nominal strength of the pile section.

EXCEPTION: Splices designed to resist the axial and shear forces and moments from the seismic load effects including overstrength of Section 12.4.3 or 12.14.3.2.

12.13.8.7 Pile–Soil Interaction. Pile moments, shears, and lateral deflections used for design shall be established considering the interaction of the shaft and soil. Where the ratio of the depth of embedment of the pile to the pile diameter or width is less than or equal to 6, the pile is permitted to be assumed to be flexurally rigid with respect to the soil.

12.13.8.8 Pile Group Effects. Pile group effects from soil on lateral pile nominal strength shall be included where pile center-to-center spacing in the direction of lateral force is less than eight pile diameters or widths. Pile group effects on vertical nominal strength shall be included where pile center-to-center spacing is less than three pile diameters or widths.

12.13.9 Requirements for Foundations on Liquefiable Sites. Where the geotechnical investigation report required in Section 11.8 identifies the potential for soil strength loss caused by liquefaction in MCE_G earthquake motions, structures shall be designed to accommodate the effects of liquefaction in accordance with the requirements of Sections 12.13.9.1 through 12.13.9.3. Such structures shall also be designed to resist the seismic load effects of Section 12.4, presuming liquefaction does not occur.

Table 12.13-2 Upper Limit on Lateral Spreading Horizontal Ground Displacement for Shallow Foundations Beyond Which Deep Foundations Are Required

Risk Category	I or II	III	IV
Limit (in. (mm))	18 (455)	12 (305)	4 (100)

EXCEPTION: Structures on shallow foundations need not be designed for the requirements of this section where the geotechnical investigation report indicates that there is negligible risk of lateral spreading, no bearing capacity loss, and differential settlements of site soils or improved site soils do not exceed one-fourth of the differential settlement threshold specified in Table 12.3-3.

Where the geotechnical investigation report indicates the potential for flow failure, the provisions of Section 12.13.9 are not applicable and the condition shall be mitigated.

12.13.9.1 Foundation Design. Foundations shall be designed to support gravity and design earthquake loads, as indicated in the basic load combinations of Section 12.4, using the reduced soil bearing capacity, as indicated in the geotechnical investigation report, considering the effects of liquefaction caused by MCE_G earthquake motions. The anticipated lateral spreading, differential settlement values, and foundation design shall be permitted to include the mitigating effects of any planned ground improvements for the site.

12.13.9.2 Shallow Foundations. Building structures shall be permitted to be supported on shallow foundations provided that the foundations are designed and detailed in accordance with Section 12.13.9.2.1 and the conditions provided in items (a) and (b) of Section 12.13.9.2 are met.

a. The geotechnical investigation report indicates that permanent horizontal ground displacement induced by lateral spreading associated with MCE_G earthquake motions does not exceed the value in Table 12.13-2.
b. The foundation and superstructure are designed to accommodate differential settlements caused by liquefaction without loss of the ability to support gravity loads. For structures assigned to Risk Category II or III, residual strength of members and connections shall not be less than 67% of the undamaged nominal strength, considering the nonlinear behavior of the structure or, alternatively, demands on all members and connections shall not exceed the element's nominal strength when subjected to differential settlements. For structures assigned to Risk Category IV, demands on all members and connections shall not exceed the element's nominal strength when subjected to differential settlements.

EXCEPTION: Where the geotechnical investigation report indicates that the differential settlement over a defined length, *L*, does not exceed the differential settlement threshold specified in Table 12.13-3, explicit design beyond the requirements of Section 12.13.9.2.1 to accommodate differential settlements is not required.

Table 12.13-3 Differential Settlement Threshold

Structure Type	Risk Category I or II	III	IV
Single-story structures with concrete or masonry wall systems	$0.0075L$	$0.005L$	$0.002L$
Other single-story structures	$0.015L$	$0.010L$	$0.002L$
Multistory structures with concrete or masonry wall systems	$0.005L$	$0.003L$	$0.002L$
Other multistory structures	$0.010L$	$0.006L$	$0.002L$

12.13.9.2.1 Shallow Foundation Design Shallow foundations shall satisfy the design and detailing requirements of Sections 12.13.9.2.1.1 or 12.13.9.2.1.2 as required.

12.13.9.2.1.1 Foundation Ties. Individual footings shall be interconnected by ties in accordance with Section 12.13.8.2 and the additional requirements of this section. The ties shall be designed to accommodate the differential settlements between adjacent footings per Section 12.13.9.2, item b. Reinforced concrete sections shall be detailed in accordance with Sections 18.6.2.1 and 18.6.4 of ACI 318. Where the geotechnical investigation report indicates that permanent ground displacement induced by lateral spreading exceeding 3 in. (76.2 mm) will occur in MCE_G earthquake motions, both of the following requirements shall be met:

1. Ties between individual footings on the same column or wall line shall, in lieu of the force requirements of Section 12.13.8.2, have a design strength in tension and compression at least equal to F_{tie}, as indicated in Eq. (12.13-1). These effects shall be combined with the load effects from design earthquake lateral loads.

$$F_{tie} = 0.5\mu P_u \tag{12.13-1}$$

where

F_{tie} = the design tie force;

μ = the coefficient of friction between the bottom of the footing and the soil, as indicated in the geotechnical report, or is taken as 0.5 in the absence of other information; and

P_u = the total of the supported gravity loads of all footings along the same column or wall line, determined in accordance with load combination 5 in Section 2.3.2.

2. Individual footings shall be integral with or connected to a reinforced concrete slab-on-ground, at least 5 in. (127.0 cm) thick and reinforced in each horizontal direction with a minimum reinforcing ratio of 0.0025. Alternately, individual footings shall be integral with or connected to a post-tensioned concrete slab-on-ground designed according to PTI DC10.5 with a minimum effective compression after losses of 100 psi (690 kPa). For sites with expansive soils, movements from both expansive soils and liquefied soils need not be considered concurrently. For purposes of this section, concrete slab-on-ground need not satisfy Section 18.6.4 of ACI 318.

EXCEPTION: A system of diagonal reinforced concrete ties is permitted to be used, if the system of ties provides equivalent lateral shear strength and stiffness to a slab-on-ground as defined above.

12.13.9.2.1.2 Mat Foundations. Mat foundations shall be designed to accommodate the expected vertical differential settlements indicated in the geotechnical investigation report per Section 12.13.9.2, item b, considering any increased loads induced by differential settlements of adjacent columns. The flexural demands caused by liquefaction need not be considered if the mat is detailed in accordance with the requirements of Section 18.6.3.1 of ACI 318. Mat foundations shall have longitudinal reinforcement in both directions at the top and bottom.

12.13.9.3 Deep Foundations. Deep foundations shall be designed to support vertical loads as indicated in the basic load combinations of Section 12.4, in combination with the moments and shears caused by lateral deformation of deep foundation elements in response to lateral inertial loads. Axial capacity of the deep foundation and lateral resistance of the soil shall be reduced to account for the effects of liquefaction. Deep foundations shall satisfy the design and detailing requirements of Sections 12.13.9.3.1 through 12.13.9.3.5.

12.13.9.3.1 Downdrag Design of piles shall incorporate the effects of downdrag caused by liquefaction. For geotechnical design, the liquefaction-induced downdrag shall be determined as the downward skin friction on the pile within and above the liquefied zone(s). The net geotechnical ultimate capacity of the pile shall be the ultimate geotechnical capacity of the pile below the liquefiable layer(s) reduced by the downdrag load. For structural design, downdrag load induced by liquefaction shall be treated as a seismic load and factored accordingly.

12.13.9.3.2 Lateral Resistance Passive pressure and friction mobilized against walls, pile caps, and grade beams, when reduced for the effects of liquefaction, shall be permitted to resist lateral inertial loads in combination with piles. Resistance provided by the combination of piles, passive pressure, and friction shall be determined based on compatible lateral deformations.

12.13.9.3.3 Concrete Deep Foundation Detailing Concrete piles including cast-in-place and precast piles shall be detailed to comply with Sections 18.7.5.2 through 18.7.5.4 of ACI 318 from the top of the pile to a depth exceeding that of the deepest liquefiable soil by at least 7 times the member cross-sectional dimension.

12.13.9.3.4 Lateral Spreading Where the geotechnical investigation report indicates that permanent ground displacement induced by lateral spreading will occur in the event of MCE_G earthquake motions, pile design shall be based on a detailed analysis incorporating the expected lateral deformation, the depths over which the deformation is expected to occur, and the nonlinear behavior of the piles. Where nonlinear behavior of piles occurs caused by permanent ground displacement induced by lateral spreading, the pile deformations shall not result in loss of the pile's ability to carry gravity loads, nor shall the deteriorated pile's lateral strength be less than 67% of the undamaged nominal strength. In addition, the following requirements shall be satisfied:

1. Structural steel H-piles shall satisfy the width-thickness limits for highly ductile H-pile members in ANSI/AISC 341.
2. Unfilled structural steel pipe piles shall satisfy the width-thickness limits for highly ductile round HSS elements in ANSI/AISC 341.
3. Concrete piles shall be detailed to comply with Sections 18.7.5.2 through 18.7.5.4 of ACI 318 from the top

of the pile to a depth exceeding that of the deepest layer of soil prone to lateral spreading by at least 7 times the pile diameter. Nominal shear strength shall exceed the maximum forces that can be generated because of pile deformations determined in the detailed analysis.

12.13.9.3.5 Foundation Ties Individual pile caps shall be interconnected by ties in accordance with Section 12.13.8.2. Where the geotechnical investigation report indicates permanent ground displacement induced by lateral spreading, the design forces for ties shall include the additional pressures applied to foundation elements because of the lateral displacement in accordance with the recommendations of the geotechnical investigation report. These effects shall be combined with the load effects from design earthquake lateral loads.

12.14 SIMPLIFIED ALTERNATIVE STRUCTURAL DESIGN CRITERIA FOR SIMPLE BEARING WALL OR BUILDING FRAME SYSTEMS

12.14.1 General

12.14.1.1 Simplified Design Procedure. The procedures of this section are permitted to be used in lieu of other analytical procedures in Chapter 12 for the analysis and design of simple buildings with bearing wall or building frame systems, subject to all of the limitations listed in this section. Where these procedures are used, the Seismic Design Category shall be determined from Table 11.6-1 using the value of S_{DS} from Section 12.14.8.1, except that where S_1 is greater than or equal to 0.75, the Seismic Design Category shall be E. The simplified design procedure is permitted to be used if the following limitations are met:

1. The structure shall qualify for Risk Category I or II in accordance with Table 1.5-1.
2. The site class, defined in Chapter 20, shall not be Site Class E or F.
3. The structure shall not exceed three stories above grade plane.
4. The seismic force-resisting system shall be either a bearing wall system or a building frame system, as indicated in Table 12.14-1.
5. The structure shall have at least two lines of lateral resistance in each of two major axis directions. At least one line of resistance shall be provided on each side of the center of weight in each direction.
6. The center of weight in each story shall be located not further from the geometric centroid of the diaphragm than 10% of the length of the diaphragm parallel to the eccentricity.
7. For structures with cast-in-place concrete diaphragms, overhangs beyond the outside line of shear walls or braced frames shall satisfy the following:

$$a \le d/3 \tag{12.14-1}$$

where

a = the distance perpendicular to the forces being considered from the extreme edge of the diaphragm to the line of vertical resistance closest to that edge, and
d = the depth of the diaphragm parallel to the forces being considered at the line of vertical resistance closest to the edge.

All other diaphragm overhangs beyond the outside line of shear walls or braced frames shall satisfy the following:

$$a \le d/5 \tag{12.14-2}$$

8. For buildings with a diaphragm that is not flexible, the forces shall be apportioned to the vertical elements as if the diaphragm were flexible. The following additional requirements shall be satisfied:
   a. For structures with two lines of resistance in a given direction, the distance between the two lines is at least 50% of the length of the diaphragm perpendicular to the lines;
   b. For structures with more than two lines of resistance in a given direction, the distance between the two most extreme lines of resistance in that direction is at least 60% of the length of the diaphragm perpendicular to the lines;

   Where two or more lines of resistance are closer together than one-half the horizontal length of the longer of the walls or braced frames, it shall be permitted to replace those lines by a single line at the centroid of the group for the initial distribution of forces, and the resultant force to the group shall then be distributed to the members of the group based on their relative stiffnesses.
9. Lines of resistance of the seismic force-resisting system shall be oriented at angles of no more than 15 deg from alignment with the major orthogonal horizontal axes of the building.
10. The simplified design procedure shall be used for each major orthogonal horizontal axis direction of the building.
11. System irregularities caused by in-plane or out-of-plane offsets of lateral force-resisting elements shall not be permitted.

    EXCEPTION: Out-of-plane and in-plane offsets of shear walls are permitted in two-story buildings of light-frame construction provided that the framing supporting the upper wall is designed for seismic force effects from overturning of the wall amplified by a factor of 2.5.
12. The lateral load resistance of any story shall not be less than 80% of the story above.

12.14.1.2 Reference Documents. The reference documents listed in Chapter 23 shall be used as indicated in Section 12.14.

12.14.1.3 Definitions. The definitions listed in Section 11.2 shall be used in addition to the following:

PRINCIPAL ORTHOGONAL HORIZONTAL DIRECTIONS: The orthogonal directions that overlay the majority of lateral force-resisting elements.

12.14.1.4 Notation.

D = the effect of dead load
E = the effect of horizontal and vertical earthquake-induced forces
F_a = acceleration-based site coefficient, see Section 12.14.8.1
F_i = the portion of the seismic base shear, V, induced at level i
F_p = the seismic design force applicable to a particular structural component
F_x = see Section 12.14.8.2
h_i = the height above the base to level i
h_x = the height above the base to level x

Table 12.14-1 Design Coefficients and Factors for Seismic Force-Resisting Systems for Simplified Design Procedure

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Limitations[b] Seismic Design Category B	C	D, E
A. BEARING WALL SYSTEMS					
1. Special reinforced concrete shear walls	14.2	5	P	P	P
2. Ordinary reinforced concrete shear walls	14.2	4	P	P	NP
3. Detailed plain concrete shear walls	14.2	2	P	NP	NP
4. Ordinary plain concrete shear walls	14.2	1½	P	NP	NP
5. Intermediate precast shear walls	14.2	4	P	P	40[c]
6. Ordinary precast shear walls	14.2	3	P	NP	NP
7. Special reinforced masonry shear walls	14.4	5	P	P	P
8. Intermediate reinforced masonry shear walls	14.4	3½	P	P	NP
9. Ordinary reinforced masonry shear walls	14.4	2	P	NP	NP
10. Detailed plain masonry shear walls	14.4	2	P	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	P	NP	NP
12. Prestressed masonry shear walls	14.4	1½	P	NP	NP
13. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	P	P	P
14. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	P	P	P
15. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	P	P	NP[d]
16. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	P	P	P
B. BUILDING FRAME SYSTEMS					
1. Steel eccentrically braced frames	14.1	8	P	P	P
2. Steel special concentrically braced frames	14.1	6	P	P	P
3. Steel ordinary concentrically braced frames	14.1	3¼	P	P	P
4. Special reinforced concrete shear walls	14.2	6	P	P	P
5. Ordinary reinforced concrete shear walls	14.2	5	P	P	NP
6. Detailed plain concrete shear walls	14.2 and 14.2.2.7	2	P	NP	NP
7. Ordinary plain concrete shear walls	14.2	1½	P	NP	NP
8. Intermediate precast shear walls	14.2	5	P	P	40[c]
9. Ordinary precast shear walls	14.2	4	P	NP	NP
10. Steel and concrete composite eccentrically braced frames	14.3	8	P	P	P
11. Steel and concrete composite special concentrically braced frames	14.3	5	P	P	P
12. Steel and concrete composite ordinary braced frames	14.3	3	P	P	NP
13. Steel and concrete composite plate shear walls	14.3	6½	P	P	P
14. Steel and concrete composite special shear walls	14.3	6	P	P	P
15. Steel and concrete composite ordinary shear walls	14.3	5	P	P	NP
16. Special reinforced masonry shear walls	14.4	5½	P	P	P
17. Intermediate reinforced masonry shear walls	14.4	4	P	P	NP
18. Ordinary reinforced masonry shear walls	14.4	2	P	NP	NP
19. Detailed plain masonry shear walls	14.4	2	P	NP	NP
20. Ordinary plain masonry shear walls	14.4	1½	P	NP	NP
21. Prestressed masonry shear walls	14.4	1½	P	NP	NP
22. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	7	P	P	P
23. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	7	P	P	P
24. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2½	P	P	NP[d]
25. Steel buckling-restrained braced frames	14.1	8	P	P	P
26. Steel special plate shear walls	14.1	7	P	P	P

[a]Response modification coefficient, R, for use throughout the standard.
[b]P = permitted; NP = not permitted.
[c]Light-frame walls with shear panels of all other materials are not permitted in Seismic Design Category E.
[d]Light-frame walls with shear panels of all other materials are permitted up to 35 ft (10.6 m) in structural height, h_n, in Seismic Design Category D and are not permitted in Seismic Design Category E.

Level i = the building level referred to by the subscript i; $i = 1$ designates the first level above the base

Level n = the level that is uppermost in the main portion of the building

Level x = see "Level i"

Q_E = the effect of horizontal seismic forces

R = the response modification coefficient as given in Table 12.14-1

S_{DS} = see Section 12.14.8.1
S_S = see Section 11.4.1
S_1 = see Section 11.4.1
V = the total design shear at the base of the structure in the direction of interest, as determined using the procedure of Section 12.14.8.1
V_x = the seismic design shear in story x. See Section 12.14.8.3
W = see Section 12.14.8.1
W_c = weight of wall
w_i = the portion of the effective seismic weight, W, located at or assigned to level i
W_p = weight of structural component
w_x = see Section 12.14.8.2

12.14.2 Design Basis. The structure shall include complete lateral and vertical force-resisting systems with adequate strength to resist the design seismic forces, specified in this section, in combination with other loads. Design seismic forces shall be distributed to the various elements of the structure and their connections using a linear elastic analysis in accordance with the procedures of Section 12.14.8. The members of the seismic force-resisting system and their connections shall be detailed to conform with the applicable requirements for the selected structural system as indicated in Section 12.14.4.1. A continuous load path, or paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the final point of resistance. The foundation shall be designed to accommodate the forces developed.

12.14.3 Seismic Load Effects. All members of the structure, including those not part of the seismic force-resisting system, shall be designed using the seismic load effects of Section 12.14.3 unless otherwise exempted by this standard. Seismic load effects are the axial, shear, and flexural member forces resulting from application of horizontal and vertical seismic forces as set forth in Section 12.14.3.1. Where required, seismic load effects shall include overstrength, as set forth in Section 12.14.3.2.

12.14.3.1 Seismic Load Effect. The seismic load effect, E, shall be determined in accordance with the following:

1. For use in load combination 6 in Section 2.3.6 or load combinations 8 and 9 in Section 2.4.5, E shall be determined in accordance with Eq. (12.14-3) as follows:

$$E = E_h + E_v \qquad (12.14\text{-}3)$$

2. For use in load combination 7 in Section 2.3.6 or load combination 10 in Section 2.4.5, E shall be determined in accordance with Eq. (12.14-4) as follows:

$$E = E_h - E_v \qquad (12.14\text{-}4)$$

where

E = seismic load effect;
E_h = effect of horizontal seismic forces as defined in Section 12.14.3.1.1; and
E_v = effect of vertical seismic forces as defined in Section 12.14.3.1.2.

12.14.3.1.1 Horizontal Seismic Load Effect The horizontal seismic load effect, E_h, shall be determined in accordance with Eq. (12.14-5) as follows:

$$E_h = Q_E \qquad (12.14\text{-}5)$$

where Q_E = effects of horizontal seismic forces from V or F_p as specified in Sections 12.14.7.5, 12.14.8.1, and 13.3.1.

12.14.3.1.2 Vertical Seismic Load Effect The vertical seismic load effect, E_v, shall be determined in accordance with Eq. (12.14-6) as follows:

$$E_v = 0.2S_{DS}D \qquad (12.14\text{-}6)$$

where

S_{DS} = design spectral response acceleration parameter at short periods obtained from Section 11.4.5, and
D = effect of dead load.

EXCEPTION: The vertical seismic load effect, E_v, is permitted to be taken as zero for either of the following conditions:

1. In Eqs. (12.14-3), (12.14-4), (12.14-7), and (12.14-8) where S_{DS} is equal to or less than 0.125.
2. In Eq. (12.14-4) where determining demands on the soil–structure interface of foundations.

12.14.3.2 Seismic Load Effect Including Overstrength. Where required, the seismic load effects, including overstrength, shall be determined in accordance with the following:

1. For use in load combination 6 in Section 2.3.6 or load combinations 8 and 9 in Section 2.4.5, E shall be taken as equal to E_m as determined in accordance with Eq. (12.14-7) as follows:

$$E_m = E_{mh} + E_v \qquad (12.14\text{-}7)$$

2. For use in load combination 7 in Section 2.3.6 or load combination 10 in Section 2.4.5, E shall be taken as equal to E_m as determined in accordance with Eq. (12.14-8) as follows:

$$E_m = E_{mh} - E_v \qquad (12.14\text{-}8)$$

where

E_m = seismic load effect including overstrength;
E_{mh} = effect of horizontal seismic forces, including overstrength, as defined in Section 12.14.3.2.1 or 12.14.3.2.2; and
E_v = vertical seismic load effect as defined in Section 12.14.3.1.2.

12.14.3.2.1 Horizontal Seismic Load Effect with a 2.5 Overstrength The effect of horizontal seismic forces, including overstrength, E_{mh}, shall be determined in accordance with Eq. (12.14-9) as follows:

$$E_{mh} = 2.5Q_E \qquad (12.14\text{-}9)$$

where

Q_E = effects of horizontal seismic forces from V or F_p as specified in Sections 12.14.7.5, 12.14.8.1, and 13.3.1.

E_{mh} need not be taken as larger than E_{cl} where E_{cl} = the capacity-limited horizontal seismic load effect as defined in Section 11.3.

12.14.3.2.2 Capacity-Limited Horizontal Seismic Load Effect Where capacity-limited design is required by the material reference document, the seismic load effect including overstrength shall be calculated with the capacity-limited horizontal seismic load effect, E_{cl}, substituted for E_{mh} in the load combinations of Section 12.14.3.2.3.

12.14.3.2.3 Load Combinations Including Overstrength. Where the seismic load effect including overstrength, E_m, defined in Section 12.14.3.2, is combined with the effects of other loads as set forth in Chapter 2, the following seismic load combinations for structures not subject to flood or atmospheric ice loads shall be used in lieu of the seismic load combinations in Section 2.3.2 or 2.4.1.

Basic Combinations for Strength Design Including Overstrength (see Sections 2.2 and 2.3.2 for notation)

5. $(1.2 + 0.2S_{DS})D + E_{mh} + L + 0.2S$
7. $(0.9 - 0.2S_{DS})D + E_{mh}$

NOTES:

1. The load factor on L in combination 5 is permitted to equal 0.5 for all occupancies in which L_o in Table 4.3-1 is less than or equal to 100 psf (4.79 kN/m^2), with the exception of garages or areas occupied as places of public assembly.
2. Where fluid loads F are present, they shall be included with the same load factor as dead load D in combinations 1 through 5 and 7. Where load H is present, it shall be included as follows:
   a. where the effect of H adds to the primary variable load effect, include H with a load factor of 1.6;
   b. where the effect of H resists the primary variable load effect, include H with a load factor of 0.9 where the load is permanent or a load factor of 0 for all other conditions.
3. Where lateral earth pressure provides resistance to structural actions from other forces, it shall not be included in H but shall be included in the design resistance.

Basic Combinations for Allowable Stress Design Including Overstrength (see Sections 2.2 and 2.4.1 for notation)

5. $(1.0 + 0.14S_{DS})D + 0.7E_{mh}$

6b. $(1.0 + 0.105S_{DS})D + 0.525E_{mh} + 0.75L + 0.75S$

8. $(0.6 - 0.14S_{DS})D + 0.7E_{mh}$

NOTES:

Where fluid loads F are present, they shall be included in combinations 1 through 6 and 8 with the same factor as that used for dead load D.

Where load H is present, it shall be included as follows:

1. where the effect of H adds to the primary variable load effect, include H with a load factor of 1.0;
2. where the effect of H resists the primary variable load effect, include H with a load factor of 0.6 where the load is permanent or a load factor of 0 for all other conditions.

12.14.4 Seismic Force-Resisting System

12.14.4.1 Selection and Limitations. The basic lateral and vertical seismic force-resisting system shall conform to one of the types indicated in Table 12.14-1 and shall conform to all of the detailing requirements referenced in the table. The appropriate response modification coefficient, R, indicated in Table 12.14-1 shall be used in determining the base shear and element design forces as set forth in the seismic requirements of this standard.

Special framing and detailing requirements are indicated in Section 12.14.7 and in Sections 14.1, 14.2, 14.3, 14.4, and 14.5 for structures assigned to the various Seismic Design Categories.

12.14.4.2 Combinations of Framing Systems

12.14.4.2.1 Horizontal Combinations Different seismic force-resisting systems are permitted to be used in each of the two principal orthogonal building directions. Where a combination of different structural systems is used to resist lateral forces in the same direction, the value of R used for design in that direction shall not be greater than the least value of R for any of the systems used in that direction.

EXCEPTION: For buildings of light-frame construction or buildings that have flexible diaphragms and that are two stories or fewer above grade plane, resisting elements are permitted to be designed using the least value of R of the different seismic force-resisting systems found in each independent line of framing. The value of R used for design of diaphragms in such structures shall not be greater than the least value for any of the systems used in that same direction.

12.14.4.2.2 Vertical Combinations Different seismic force-resisting systems are permitted to be used in different stories. The value of R used in a given direction shall not be greater than the least value of any of the systems used in that direction.

12.14.4.2.3 Combination Framing Detailing Requirements The detailing requirements of Section 12.14.7 required by the higher response modification coefficient, R, shall be used for structural members common to systems that have different response modification coefficients.

12.14.5 Diaphragm Flexibility. Diaphragms constructed of steel decking (untopped), wood structural panels, or similar panelized construction techniques are permitted to be considered flexible.

12.14.6 Application of Loading. The effects of the combination of loads shall be considered as prescribed in Section 12.14.3. The design seismic forces are permitted to be applied separately in each orthogonal direction, and the combination of effects from the two directions need not be considered. Reversal of load shall be considered.

12.14.7 Design and Detailing Requirements. The design and detailing of the members of the seismic force-resisting system shall comply with the requirements of this section. The foundation shall be designed to resist the forces developed and accommodate the movements imparted to the structure by the design ground motions. The dynamic nature of the forces, the expected ground motion, the design basis for strength and energy dissipation capacity of the structure, and the dynamic properties of the soil shall be included in the determination of the foundation design criteria. The design and construction of foundations shall comply with Section 12.13. Structural elements including foundation elements shall conform to the material design and detailing requirements set forth in Chapter 14.

12.14.7.1 Connections. All parts of the structure between separation joints shall be interconnected, and the connection shall be capable of transmitting the seismic force, F_p, induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements that have a strength of 0.20 times the short-period design spectral response acceleration coefficient, S_{DS}, times the weight of the smaller portion or 5% of the portion's weight, whichever is greater.

A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder, or truss, either directly to its supporting elements or to slabs designed to act as diaphragms. Where the connection is through a diaphragm, then the member's supporting element must also be connected to the diaphragm. The connection shall have minimum design strength of 5% of the dead plus live load reaction.

12.14.7.2 Openings or Reentrant Building Corners. Except where otherwise specifically provided for in this standard, openings in shear walls, diaphragms, or other plate-type elements shall be provided with reinforcement at the edges of the openings or reentrant corners designed to transfer the stresses into the structure. The edge reinforcement shall extend into the body of the wall or diaphragm a distance sufficient to develop the force in the reinforcement.

EXCEPTION: Shear walls of wood structural panels are permitted where designed in accordance with AWC SDPWS-15 for perforated shear walls or ANSI/AISI S400 for Type II shear walls.

12.14.7.3 Collector Elements. Collector elements shall be provided with adequate strength to transfer the seismic forces originating in other portions of the structure to the element providing the resistance to those forces (Fig. 12.10-1). Collector elements, splices, and their connections to resisting elements shall be designed to resist the forces defined in Section 12.14.3.2.

EXCEPTION: In structures, or portions thereof, braced entirely by light-frame shear walls, collector elements, splices, and connections to resisting elements are permitted to be designed to resist forces in accordance with Section 12.14.7.4.

12.14.7.4 Diaphragms. Floor and roof diaphragms shall be designed to resist the design seismic forces at each level, F_x, calculated in accordance with Section 12.14.8.2. Where the diaphragm is required to transfer design seismic forces from the vertical-resisting elements above the diaphragm to other vertical-resisting elements below the diaphragm because of changes in relative lateral stiffness in the vertical elements, the transferred portion of the seismic shear force at that level, V_x, shall be added to the diaphragm design force. Diaphragms shall provide for both the shear and bending stresses resulting from these forces. Diaphragms shall have ties or struts to distribute the wall anchorage forces into the diaphragm. Diaphragm connections shall be positive, mechanical, or welded type connections.

12.14.7.5 Anchorage of Structural Walls. Structural walls shall be anchored to all floors, roofs, and members that provide out-of-plane lateral support for the wall or that are supported by the wall. The anchorage shall provide a positive direct connection between the wall and floor, roof, or supporting member with the strength to resist the out-of-plane force given by Eq. (12.14-10):

$$F_p = 0.4k_a S_{DS} W_p \qquad (12.14\text{-}10)$$

F_p shall not be taken as less than $0.2k_a W_p$.

$$k_a = 1.0 + \frac{L_f}{100} \qquad (12.14\text{-}11)$$

k_a need not be taken as larger than 2.0 where

F_p = the design force in the individual anchors;
k_a = amplification factor for diaphragm flexibility;
L_f = the span, in feet, of a flexible diaphragm that provides the lateral support for the wall; the span is measured between vertical elements that provide lateral support to the diaphragm in the direction considered; use zero for rigid diaphragms;
S_{DS} = the design spectral response acceleration at short periods per Section 12.14.8.1; and
W_p = the weight of the wall tributary to the anchor.

12.14.7.5.1 Transfer of Anchorage Forces into Diaphragms Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. Added chords are permitted to be used to form subdiaphragms to transmit the anchorage forces to the main continuous crossties. The maximum length-to-width ratio of the structural subdiaphragm shall be 2.5 to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

12.14.7.5.2 Wood Diaphragms The anchorage of concrete or masonry structural walls to wood diaphragms shall be in accordance with AWC SDPWS 4.1.5.1 and this section. Continuous ties required by this section shall be in addition to the diaphragm sheathing. Anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension. The diaphragm sheathing shall not be considered effective as providing the ties or struts required by this section.

12.14.7.5.3 Metal Deck Diaphragms In metal deck diaphragms, the metal deck shall not be used as the continuous ties required by this section in the direction perpendicular to the deck span.

12.14.7.5.4 Embedded Straps Diaphragm to wall anchorage using embedded straps shall be attached to or hooked around the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

12.14.7.6 Bearing Walls and Shear Walls. Exterior and interior bearing walls and shear walls and their anchorage shall be designed for a force equal to 40% of the short-period design spectral response acceleration, S_{DS}, times the weight of wall, W_c, normal to the surface, with a minimum force of 10% of the weight of the wall. Interconnection of wall elements and connections to supporting framing systems shall have sufficient ductility, rotational capacity, or strength to resist shrinkage, thermal changes, and differential foundation settlement where combined with seismic forces.

12.14.7.7 Anchorage of Nonstructural Systems. Where required by Chapter 13, all portions or components of the structure shall be anchored for the seismic force, F_p, prescribed therein.

12.14.8 Simplified Lateral Force Analysis Procedure. An equivalent lateral force analysis shall consist of the application of equivalent static lateral forces to a linear mathematical model of the structure. The lateral forces applied in each direction shall sum to a total seismic base shear given by Section 12.14.8.1 and shall be distributed vertically in accordance with Section 12.14.8.2. For purposes of analysis, the structure shall be considered fixed at the base.

12.14.8.1 Seismic Base Shear. The seismic base shear, V, in a given direction shall be determined in accordance with Eq. (12.14-12):

$$V = \frac{F S_{DS}}{R} W \qquad (12.14\text{-}12)$$

where

$$S_{DS} = \frac{2}{3} F_a S_s$$

where F_a is permitted to be taken as 1.0 for rock sites, 1.4 for soil sites, or determined in accordance with Section 11.4.4. For the purpose of this section, sites are permitted to be considered to be rock if there is no more than 10 ft (3 m) of soil between the rock surface and the bottom of spread footing or mat foundation. In calculating S_{DS}, S_s shall be in accordance with Section 11.4.4 but need not be taken as larger than 1.5.

F = 1.0 for buildings that are one story above grade plane;
F = 1.1 for buildings that are two stories above grade plane;
F = 1.2 for buildings that are three stories above grade plane;
R = the response modification factor from Table 12.14-1; and
W = effective seismic weight of the structure that includes the dead load, as defined in Section 3.1, above grade plane and other loads above grade plane as listed in the following text:

1. In areas used for storage, a minimum of 25% of the floor live load shall be included.
   EXCEPTIONS:
   a. Where the inclusion of storage loads adds no more than 5% to the effective seismic weight at that level, it need not be included in the effective seismic weight.
   b. Floor live load in public garages and open parking structures need not be included.
2. Where provision for partitions is required by Section 4.3.2 in the floor load design, the actual partition weight, or a minimum weight of 10 psf (0.48 kN/m^2) of floor area, whichever is greater.
3. Total operating weight of permanent equipment.
4. Where the flat roof snow load, P_f, exceeds 30 psf (1.44 kN/m^2), 20% of the uniform design snow load, regardless of actual roof slope.
5. Weight of landscaping and other materials at roof gardens and similar areas.

12.14.8.2 Vertical Distribution. The forces at each level shall be calculated using the following equation:

$$F_x = \frac{w_x}{W} V \qquad (12.14\text{-}13)$$

where w_x = the portion of the effective seismic weight of the structure, W, at level x.

12.14.8.3 Horizontal Shear Distribution. The seismic design story shear in any story, V_x [kip (kN)], shall be determined from the following equation:

$$V_x = \sum_{1=x}^{n} F_i \qquad (12.14\text{-}14)$$

where F_i = the portion of the seismic base shear, V [kip (kN)] induced at level i.

12.14.8.3.1 Flexible Diaphragm Structures The seismic design story shear in stories of structures with flexible diaphragms, as defined in Section 12.14.5, shall be distributed to the vertical elements of the seismic force-resisting system using tributary area rules. Two-dimensional analysis is permitted where diaphragms are flexible.

12.14.8.3.2 Structures with Diaphragms That Are Not Flexible For structures with diaphragms that are not flexible, as defined in Section 12.14.5, the seismic design story shear, V_x [kip (kN)], shall be distributed to the various vertical elements of the seismic force-resisting system in the story under consideration based on the relative lateral stiffnesses of the vertical elements and the diaphragm.

12.14.8.3.2.1 Torsion. The design of structures with diaphragms that are not flexible shall include the torsional moment, M_t [kip-ft (kN-m)] resulting from eccentricity between the locations of center of mass and the center of rigidity.

12.14.8.4 Overturning. The structure shall be designed to resist overturning effects caused by the seismic forces determined in Section 12.14.8.2. The foundations of structures shall be designed for not less than 75% of the foundation overturning design moment, M_f [kip-ft (kN-m)] at the foundation–soil interface.

12.14.8.5 Drift Limits and Building Separation. Structural drift need not be calculated. Where a drift value is needed for use in material standards, to determine structural separations between buildings or from property lines, for design of cladding, or for other design requirements, it shall be taken as 1% of structural height, h_n, unless computed to be less. Each portion of the structure shall be designed to act as an integral unit in resisting seismic forces unless it is separated structurally by a distance sufficient to avoid damaging contact under the total deflection.

12.15 CONSENSUS STANDARDS AND OTHER REFERENCED DOCUMENTS

See Chapter 23 for the list of consensus standards and other documents that shall be considered part of this standard to the extent referenced in this chapter.

This page intentionally left blank

CHAPTER 13
SEISMIC DESIGN REQUIREMENTS FOR NONSTRUCTURAL COMPONENTS

13.1 GENERAL

13.1.1 Scope. This chapter establishes minimum design criteria for nonstructural components that are permanently attached to structures and for their supports and attachments. Where the weight of a nonstructural component is greater than or equal to 25% of the effective seismic weight, W, of the structure as defined in Section 12.7.2, the component shall be classified as a nonbuilding structure and shall be designed in accordance with Section 15.3.2.

13.1.2 Seismic Design Category. For the purposes of this chapter, nonstructural components shall be assigned to the same Seismic Design Category as the structure that they occupy or to which they are attached.

13.1.3 Component Importance Factor. All components shall be assigned a component Importance Factor as indicated in this section. The component Importance Factor, I_p, shall be taken as 1.5 if any of the following conditions apply:

1. The component is required to function for life-safety purposes after an earthquake, including fire protection sprinkler systems and egress stairways.
2. The component conveys, supports, or otherwise contains toxic, highly toxic, or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released.
3. The component is in or attached to a Risk Category IV structure, and it is needed for continued operation of the facility or its failure could impair the continued operation of the facility.
4. The component conveys, supports, or otherwise contains hazardous substances and is attached to a structure or portion thereof classified by the Authority Having Jurisdiction as a hazardous occupancy.

All other components shall be assigned a component Importance Factor, I_p, equal to 1.0.

13.1.4 Exemptions. The following nonstructural components are exempt from the requirements of this chapter:

1. Furniture except storage cabinets, as noted in Table 13.5-1;
2. Temporary or movable equipment;
3. Architectural components in Seismic Design Category B, other than parapets, provided that the component Importance Factor, I_p, is equal to 1.0;
4. Mechanical and electrical components in Seismic Design Category B;
5. Mechanical and electrical components in Seismic Design Category C provided that either
   a. The component Importance Factor, I_p, is equal to 1.0 and the component is positively attached to the structure; or
   b. The component weighs 20 lb (89 N) or less or, in the case of a distributed system, 5 lb/ft (73 N/m) or less.
6. Discrete mechanical and electrical components in Seismic Design Categories D, E, or F that are positively attached to the structure, provided that either
   a. The component weighs 400 lb (1,779 N) or less, the center of mass is located 4 ft (1.22 m) or less above the adjacent floor level, flexible connections are provided between the component and associated ductwork, piping, and conduit, and the component Importance Factor, I_p, is equal to 1.0; or
   b. The component weighs 20 lb (89 N) or less or, in the case of a distributed system, 5 lb/ft (73 N/m) or less; and.
7. Distribution systems in Seismic Design Categories D, E, or F included in the exceptions for conduit, cable tray, and raceways in Section 13.6.5, duct systems in 13.6.6 and piping and tubing systems in 13.6.7.3. Where in-line components, such as valves, in-line suspended pumps, and mixing boxes require independent support, they shall be addressed as discrete components and shall be braced considering the tributary contribution of the attached distribution system.

13.1.5 Premanufactured Modular Mechanical and Electrical Systems. Premanufactured mechanical and electrical modules 6 ft (1.8 m) high and taller that are not otherwise prequalified in accordance with Chapter 13 and that contain or support mechanical and electrical components shall be designed in accordance with the provisions for nonbuilding structures similar to buildings in Chapter 15. Nonstructural components contained or supported within modular systems shall be designed in accordance with Chapter 13.

13.1.6 Application of Nonstructural Component Requirements to Nonbuilding Structures. Nonbuilding structures (including storage racks and tanks) that are supported by other structures shall be designed in accordance with Chapter 15. Where Section 15.3 requires that seismic forces be determined in accordance with Chapter 13 and values for R_p are not provided in Table 13.5-1 or 13.6-1, R_p shall be taken as equal to the value of R listed in Chapter 15. The value of a_p shall be determined in accordance with footnote *a* of Table 13.5-1 or 13.6-1.

13.1.7 Reference Documents. Where a reference document provides a basis for the earthquake-resistant design of a particular type of nonstructural component, that document is permitted to be used, subject to the approval of the Authority Having Jurisdiction and the following conditions:

1. The design earthquake forces shall not be less than those determined in accordance with Section 13.3.1.
2. Each nonstructural component's seismic interactions with all other connected components and with the supporting structure shall be accounted for in the design. The component shall accommodate drifts, deflections, and relative displacements determined in accordance with the applicable seismic requirements of this standard.
3. Nonstructural component anchorage requirements shall not be less than those specified in Section 13.4.

13.1.8 Reference Documents Using Allowable Stress Design. Where a reference document provides a basis for the earthquake-resistant design of a particular type of component, and the same reference document defines acceptance criteria in terms of allowable stresses rather than strengths, that reference document is permitted to be used. The allowable stress load combination shall consider dead, live, operating, and earthquake loads in addition to those in the reference document. The earthquake loads determined in accordance with Section 13.3.1 shall be multiplied by a factor of 0.7. The allowable stress design load combinations of Section 2.4 need not be used. The component shall also accommodate the relative displacements specified in Section 13.3.2.

13.2 GENERAL DESIGN REQUIREMENTS

13.2.1 Applicable Requirements for Architectural, Mechanical, and Electrical Components, Supports, and Attachments. Architectural, mechanical, and electrical components, supports, and attachments shall comply with the sections referenced in Table 13.2-1. These requirements shall be satisfied by one of the following methods:

1. Project-specific design and documentation submitted for approval to the Authority Having Jurisdiction after review and acceptance by a registered design professional.
2. Submittal of the manufacturer's certification that the component is seismically qualified by at least one of the following:
   a. Analysis, or
   b. Testing in accordance with the alternative set forth in Section 13.2.5, or
   c. Experience data in accordance with the alternative set forth in Section 13.2.6.

13.2.2 Special Certification Requirements for Designated Seismic Systems. Certifications shall be provided for designated seismic systems assigned to Seismic Design Categories C through F as follows:

1. Active mechanical and electrical equipment that must remain operable following the design earthquake ground motion shall be certified by the manufacturer as operable whereby active parts or energized components shall be certified exclusively on the basis of approved shake table testing in accordance with Section 13.2.5 or experience data in accordance with Section 13.2.6 unless it can be shown that the component is inherently rugged by comparison with similar seismically qualified components. Evidence demonstrating compliance with this requirement shall be submitted for approval to the Authority Having Jurisdiction after review and acceptance by a registered design professional.
2. Components with hazardous substances and assigned a component Importance Factor, I_p, of 1.5 in accordance with Section 13.1.3 shall be certified by the manufacturer as maintaining containment following the design earthquake ground motion by (1) analysis, (2) approved shake table testing in accordance with Section 13.2.5, or (3) experience data in accordance with Section 13.2.6. Evidence demonstrating compliance with this requirement shall be submitted for approval to the Authority Having Jurisdiction after review and acceptance by a registered design professional.
3. Certification of components through analysis shall be limited to nonactive components and shall be based on seismic demand considering R_p/I_p equal to 1.0.

13.2.3 Consequential Damage. The functional and physical interrelationship of components, their supports, and their effect on each other shall be considered so that the failure of an essential or nonessential architectural, mechanical, or electrical component shall not cause the failure of an essential architectural, mechanical, or electrical component. Where not otherwise established by analysis or test, required clearances for sprinkler system drops and sprigs shall not be less than those specified in Section 13.2.3.1.

13.2.3.1 Clearances between Equipment, Distribution Systems, Supports, and Sprinkler System Drops and Sprigs. The installed clearance between any sprinkler drop or sprig and the following items shall be at least 3 in. (75 mm) in all directions:

1. permanently attached equipment including their structural supports and bracing; and
2. other distribution systems, including their structural supports and bracing.

EXCEPTION: Sprinklers installed using flexible sprinkler hose need not meet the installed clearance requirement of this section.

13.2.4 Flexibility. The design and evaluation of components, their supports, and their attachments shall consider their flexibility and their strength.

13.2.5 Testing Alternative for Seismic Capacity Determination. As an alternative to the analytical requirements of Sections 13.2 through 13.6, testing shall be deemed as an

Table 13.2-1 Applicable Requirements for Architectural, Mechanical, and Electrical Components: Supports and Attachments

Nonstructural Element (i.e., Component, Support, Attachment)	General Design Requirements (Section 13.2)	Force and Displacement Requirements (Section 13.3)	Attachment Requirements (Section 13.4)	Architectural Component Requirements (Section 13.5)	Mechanical and Electrical Component Requirements (Section 13.6)
Architectural components and supports and attachments for architectural components	X	X	X	X	
Mechanical and electrical components	X	X	X		X
Supports and attachments for mechanical and electrical components	X	X	X		X