Calculations and Discussion	*Code Reference*

1. **Basic allowable stress design** **§12.4.2.3**

The governing load combinations for basic allowable stress design are Basic ASD Combinations 5, 6, and 8, as modified in §12.4.2.3. These are used without the usual one-third stress increase.

§12.4.2 defines the seismic load effect E for use in load combinations as

$E = E_h + E_v$ (Eq 12.4-1)

$= \delta Q_E + 0.2\ S_{DS}D$ (Eq 12.4-3)

$= Q_E + 0.06D$ when D and Q_E are in the same sense (Eq 12.4-4)

and $E = \rho Q_E - 0.2S_{DS}D$

$= Q_E - 0.06D$ when D and Q_E have opposite sense

For ASD Basic Combination 5 the load combination is:

$D + 0.7E$ (Comb. 5)

$= D(1.0) + 0.7\ (0.6D + Q_E)$

$= (1.042)D + 0.7Q_E$ for D and Q_E with the same sense

and $D(1.0) + 0.7\ (-0.6D - Q_E)$

$= 0.958D - 0.7Q_E$ for D and Q_E with opposite sense

For ASD Basic Combination 6 the load combination is:

$D + 0.75(0.7E) + 0.75\ (L + L_r)$ (Comb. 6)

$= D(1.0 + (0.75)(0.7)(0.06)) + (0.75)(0.70)(1.0)Q_E + 0.75\ L_r$

$= 1.032D + 0.75L_r + 0.525\ Q_E$ for D and Q_E with the same sense

$= -\ 0.968D + 0.75\ L_r - 0.525\ Q_E$ for D and Q_E with the opposite sense

For ASD Basic Combination 8 the load combination is:

$0.6D + 0.7E$ (Comb. 8)

Calculations and Discussion	*Code Reference*

1. Maximum considered earthquake spectral response accelerations §11.4.1

For the given position (Near Sonora – NW of Sacramento, California) of 38° North (Latitude = 38.123°) and 121.123° West (Longitude = – 121.123°), USGS provides the values of

$$S_S = 46.2\%g = 0.573g$$

$$S_1 = 20.3\%g = 0.230g$$

2. Site coefficients and adjusted maximum considered earthquake spectral response accelerations §11.4.3

From the USGS for the given site class *D*, and $S_S = 0.573g$, $S_1 = 0.230g$, the site coefficients are as follows

$F_a = 1.43$ T 11.4-1

$F_v = 1.99$ T 11.4-2

The adjusted maximum considered earthquake spectral response accelerations (based on §11.4.3) are also given on the CD ROM as follows

$S_{MS} = F_a S_s = 1.43(0.573g) = 0.819g$ (Eq 11.4-1)

$S_{M1} = F_v S_1 = 1.99(0.230g) = 0.458g$ (Eq 11.4-2)

3. Design spectral response acceleration parameters §11.4.4

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} (0.819g) = 0.546g \qquad \text{(Eq 11.4-3)}$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} (0.458g) = 0.305g \qquad \text{(Eq 11.4-4)}$$

4. General procedure response spectrum §11.4.5

For periods less than or equal to T_o, the design spectral response shall be given by

$$S_a = 0.6 \frac{S_{DS}}{T_o} T + 0.4\, S_{DS} \qquad \text{(Eq 11.4-5)}$$

For periods greater than or equal to T_o and less than or equal to T_s, the design spectral response acceleration S_a shall be taken equal to S_{DS}

For periods greater than T_s, and less than T_L, the design spectral response acceleration S_a shall be given by

$$S_a = (S_{D1}) / T \qquad \text{(Eq 11.4-6)}$$

Where: $T_o = 0.20\ (S_{D1} / S_{DS})$

$= 0.2\ (0.305 / 0.546)$

$= 0.11$ sec

$T_s = S_{D1} / S_{DS}$

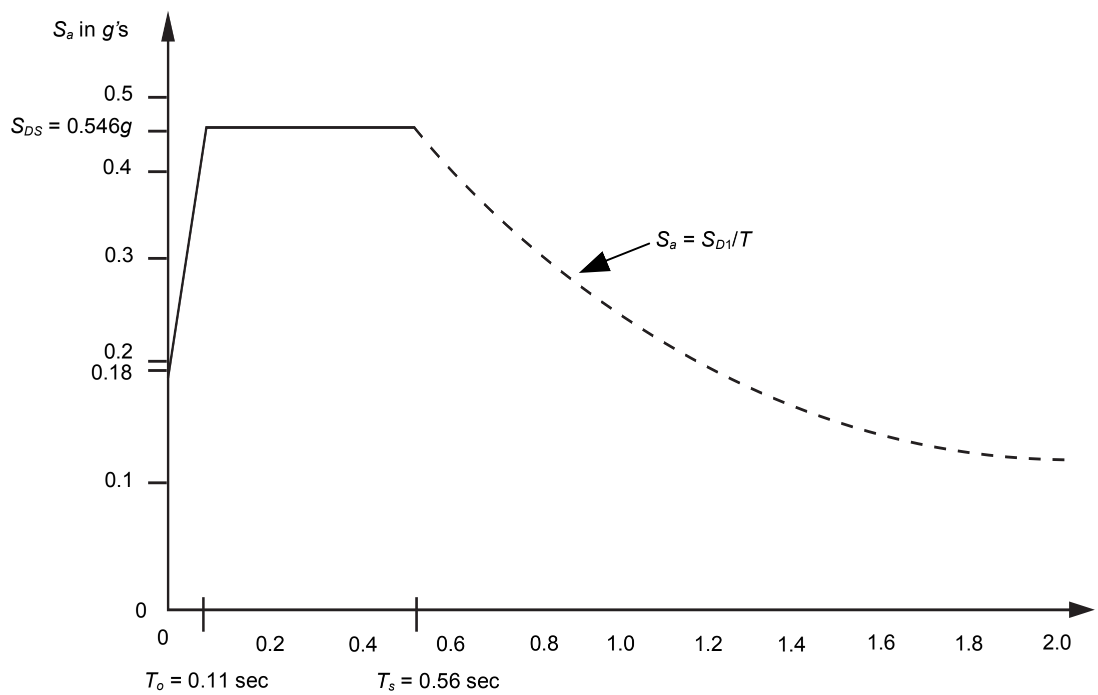
$= 0.305 / 0.546$

$= 0.56$ sec

$T_L = 8$ sec (F 22-15)

Thus:

T = Period	S_a/g	Computation for S_a
0.00	0.18	0.4 (0.44)
0.11	0.546	0.44
0.56	0.546	0.30 / 0.55
0.80	0.38	0.30 / 0.8
1.00	0.30	0.30 / 1.2
1.20	0.25	0.30 / 1.4
1.40	0.22	0.30 / 1.6
1.60	0.19	0.30 / 1.8
2.00	0.15	0.30 / 2.0



General Procedure Response Spectrum

5. Calculation of seismic response coefficient C_s (Recall Soil Site Class D, I = 1.0 and T = 0.60) §12.8.1

The seismic response coefficient shall be determined by

$$C_s = S_{DS} / (R/I) \quad \text{(Eq 12.8-2)}$$

$$= 0.546 / (6.0/1.0)$$

$$= 0.091 \text{ . . . Governs}$$

The value of C_s need not exceed

$$C_s = S_{D1} / (R/I)\, T \quad \text{(Eq 12.8-3)}$$

$$= 0.30 / (6.0/1.0)\,(0.6)$$

$$= 0.085$$

for $T \leq T_L$

But shall not be taken less than

$$C_s = 0.01 \quad \text{(Eq 12.8-5)}$$

where $S_1 \geq 0.6g$ C_s shall not be less than

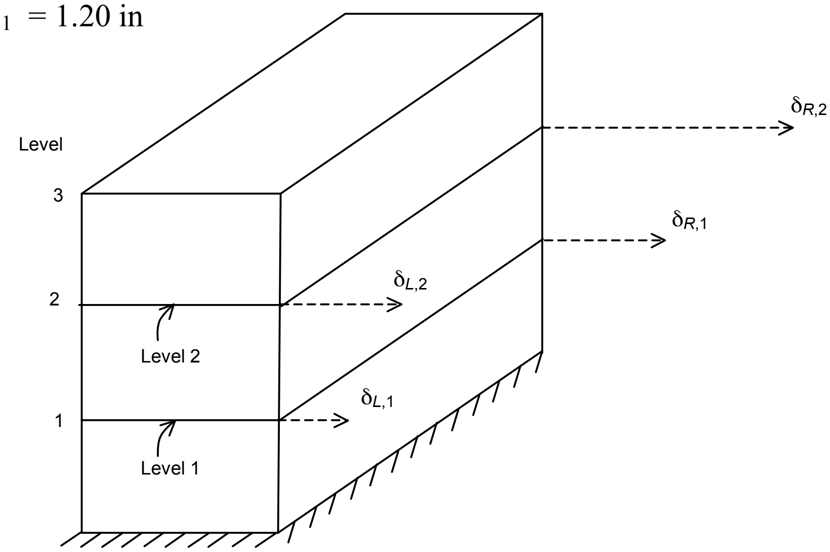
$$C_s = 0.5S_1 / (R/I) \quad \text{(Eq 12.8-6)}$$

Example 10
Horizontal Irregularity Type 1a and Type 1b §12.3.2.1

A three-story special moment-resisting frame building has rigid floor diaphragms. Under code-prescribed seismic forces, including the effects of accidental torsion, it has the following elastic displacements δ_{xe} at Levels 1 and 2.

$\delta_{L,2}$ = 1.20 in $\delta_{R,2}$ = 1.90 in

$\delta_{L,1}$ = 1.00 in $\delta_{R,1}$ = 1.20 in



1. **Determine if a Type 1a or Type 1b torsional irregularity exists at the second story**

If it does:

2. **Compute the torsional amplification factor A_x for Level 2**

Calculations and Discussion ***Code Reference***

A Type 1a torsional irregularity is considered to exist when the maximum story drift, including accidental torsion effects, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure, see §12.8.6 for story drift determination

2. Compute amplification factor A_x for Level 2 §12.8.4.3

When torsional irregularity exists at a Level x, the accidental torsional moment M_{ta} must be increased by an amplification factor A_x. This must be done for each level, and each level may have a different A_x value. In this example, A_x is computed for Level 2.
Note that A_x is a function of the displacements as opposed to/versus the drift.

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad \text{(IBC Eq 16-44)}$$

δ_{max} = 1.90 in…($\delta_{R,2}$)

$$\delta_{avg} = \frac{\delta_{L,2} + \delta_{R,2}}{2} = \frac{1.30 + 1.90}{2} = 1.60 \text{ in}$$

$$A_2 = \left(\frac{1.90}{1.2(1.60)}\right)^2 = 0.98 < 1.0$$ … Note A_x shall not be less than 1.0

$\therefore$ use $A_x = 1.0$.

Commentary

In §12.8.4.3, there is the provision that the more severe loading shall be considered. The interpretation of this for the case of the story drift and displacements to be used for the average values $\Delta\delta_{avg}$ and δ_{avg} is as follows. The most severe condition is when both $\delta_{R,X}$ and $\delta_{L,X}$ are computed for the same accidental center-of-mass displacement that causes the maximum displacement δ_{max}. For the condition shown in this example where $\delta_{R,X} = \delta_{max}$, the centers-of-mass at all levels should be displaced by the accidental eccentricity to the right side R, and both $\delta_{R,X}$ and $\delta_{L,X}$ should be evaluated for this load condition.

Table 12.3-1 triggers a number of special design requirements for torsionally irregular structures. In fact, if irregularity Type 1b (Extreme Torsional Irregularity) is present, §12.3.3.1 is triggered, which prohibits such structures for SDC E or F. It is important to recognize that torsional irregularity is defined in terms of story drift Δ_x, while the evaluation of A_x by Equation 12.8-14 is, in terms of displacements δ_{xe}. There can be instances where the story-drift values indicate torsional irregularity and where the related displacement values produce an A_x value less than 1.0. This result is not the intent of the provision, and the value of A_x used to determine accidental torsion should not be less than 1.0.

The displacement and story-drift values should be obtained by the equivalent lateral-force method with the code-prescribed lateral forces. Theoretically, if the dynamic analysis procedure were to be used, the values of Δ_{max} and Δ_{avg} would have to be found for each dynamic mode, then combined by the appropriate SRSS or CQC procedures, and then scaled to the code-prescribed base shear. However, in view of the complexity of this determination and the judgmental nature of the 1.2 factor, it is reasoned that the equivalent static force method is sufficiently accurate to detect torsional irregularity and evaluate the A_x factor.

Example 15
Redundancy Factor ρ *§12.3.4*

The calculation of the redundancy factor ρ has changed considerably between earlier codes (1997 UBC; 2000 and 2003 IBC; ASCE/SEI 7-02) and the ASCE/SEI 7-05. The calculation is in some ways simpler, although it nevertheless requires some effort for conditions that do not comply with prescriptive requirements (unless the full penalty is taken, as described below).

ASCE/SEI 7-05 permits the redundancy factor to be taken as 1.0 in the following circumstances (§12.3.4.1):

1. Structures assigned to Seismic Design Category B or C. (Note that the load combinations that include the redundancy factor are not used for Seismic Design Category A.)
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 load combinations with overstrength factor of §12.4.3.2 are used.
6. Design of members or connections where the load combinations with overstrength of §12.4.3.2 are required for design.
7. Diaphragm loads determined using Eq. 12.10-1 (note that this does not apply to forces transferred through a diaphragm, such as due to an out-of-plane offset in the seismic load resisting system, and the higher ρ factor may apply as otherwise required).
8. Structures with damping systems designed in accordance with 18.

Additionally, §12.3.4.2 identifies two other conditions in which ρ may be taken as 1.0. Note that the criteria for these conditions need only be met at floor levels in which more than 35-percent of the base shear is being resisted; for the top level or levels of taller structures, the conditions need not be met. The factor may be taken as 1.0 when either of the conditions listed below is met. In all other conditions, ρ is taken as 1.3. There is no longer a calculated ρ factor between the minimum and maximum values.

<u>Condition I</u>
12.3.4.2(a) Configurations in which the removal of one element (as described below in the summary of Table 12.3-3) will not result in more than a 33-percent reduction in story shear strength or in an extreme torsional irregularity (as defined in Table 12.3-1).

Summary of Table 12.3-3

Removal of one element is defined as:

1. The removal of a brace (braced frames).
2. Loss of moment resistance at the beam-to-column connections at both ends of a single beam (moment frames).
3. Removal of a shear wall or wall pier with a height-to-length ratio greater than 1.0 (shear wall systems).
4. Loss of moment resistance at the base connections of any single cantilever column (cantilever column systems).
5. For other systems, such as seismically damped structures, no prescriptive requirements are given, allowing ρ to be taken as 1.0.

<u>Condition II</u>
12.3.4.2(b) Configurations with no plan irregularities at any level and with sufficient perimeter braced frames, moment frames, or shearwalls. Sufficient perimeter bracing is defined as at least two bays of seismic force-resisting perimeter framing on each side of the structure in each orthogonal direction. For shear wall systems the number of bays is calculated as the length of shear wall divided by the story height (two times the length of shear wall divided by the story height for light-framed construction).

EXAMPLE
To illustrate the application of the method for establishing the redundancy factor, the structure shown in Figure 15.1 will be analyzed.

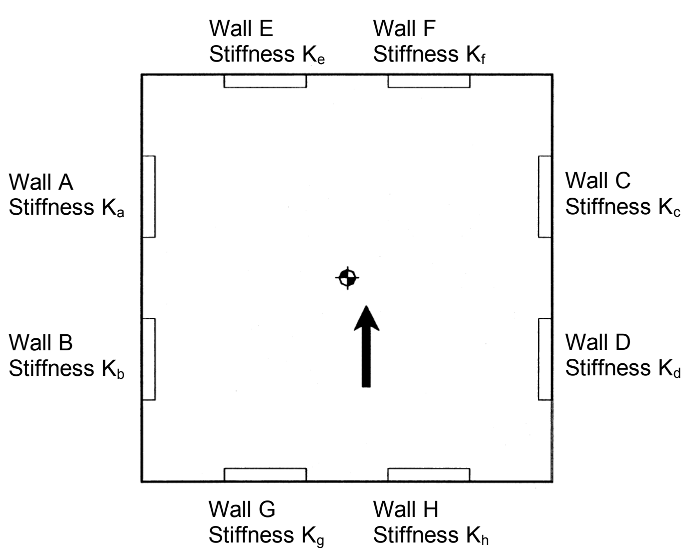


Figure 15-1

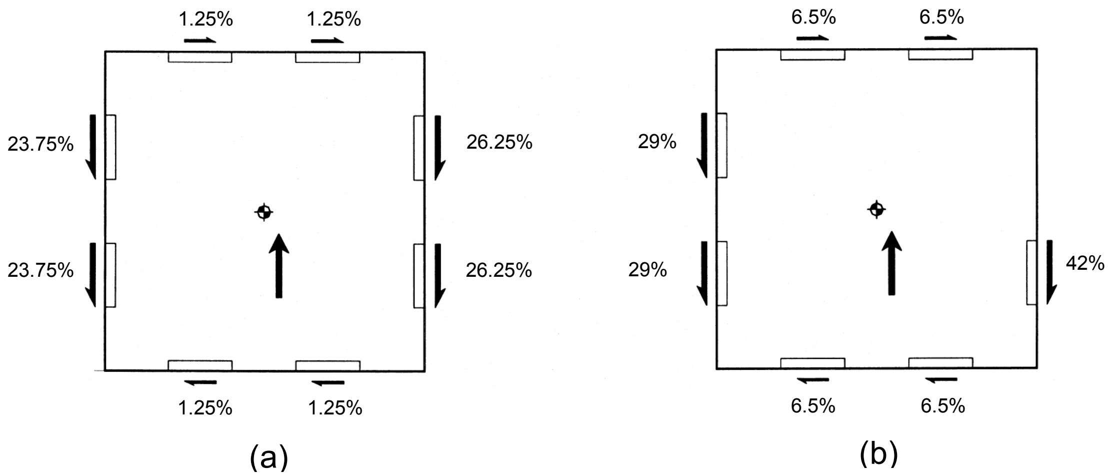


Figure 15-2

While this is an acceptable method of demonstrating compliance with the conditions justifying a factor ρ of 1.0, this method is not required. A more direct method of establishing story shear capacity is to utilize a plastic mechanism analysis. This is the method envisioned by the committee that authored the redundancy provision, and it is more consistent with the principles of seismic design (i.e., considering strength and limit states, rather than elastic design). In this method of analysis, the story shear capacity before removal of a wall is the sum of the capacities of the 4 walls resisting the seismic force in the direction under consideration (provided that the orthogonal walls have sufficient strength to resist the torsion, which in this case is only the accidental torsion). This is shown in Figure 15.3(a), where R_n denotes the capacity of the wall. If one wall is removed, the story shear capacity is the sum of the capacities of the 3 remaining walls resisting the seismic force in the direction under consideration; again, the orthogonal walls must be checked for the forces resulting from building torsion, which in this case is substantial. This is shown in Figure 15.3(b). Thus the reduction in capacity is only 25-percent. The resulting building torsional forces must be resisted by the frames in the orthogonal direction. This interpretation of the story shear capacity has been endorsed by the SEAOC Seismology Committee.

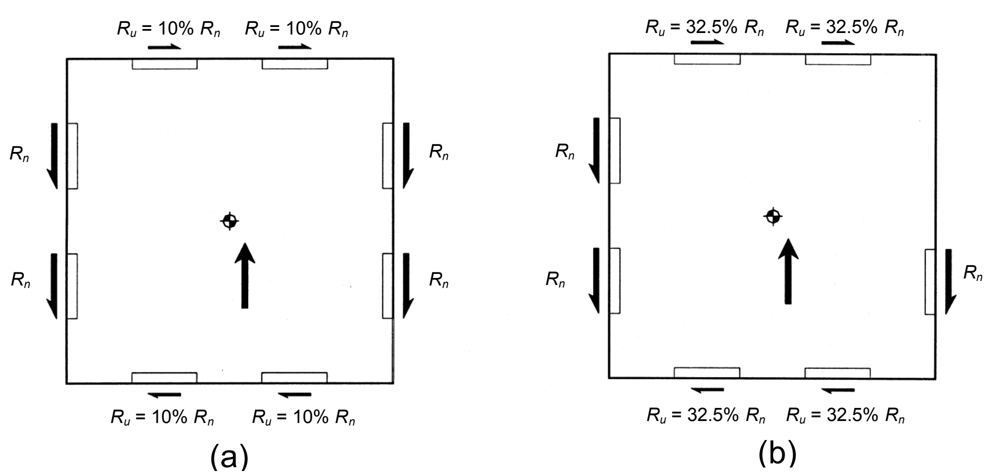


Figure 15-3

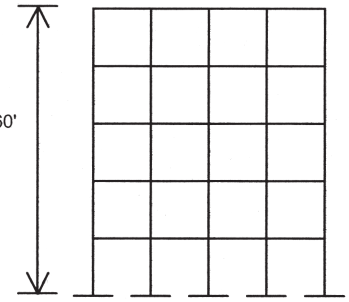
Example 17
Seismic Base Shear §12.8.1

Find the design base shear for a 5-story steel special moment-resisting frame building shown below.

The following information is given.

Seismic Design Category D

S_{DS} = 0.45g
S_{D1} = 0.28g
I = 1.0
R = 8
W = 1626 kips
h_n = 60 feet



To solve this example, follow these steps.

1. Determine the structure period

2. Determine the seismic response coefficient C_s

3. Determine seismic base shear

Calculations and Discussion **Code Reference**

1. Determine the structure period §12.8.2.1

The appropriate fundamental period T_a is to be used. C_T for steel moment-resisting frames is 0.035.

$$T_a = C_T(h_n)^{3/4} = 0.035(60)^{3/4} = 0.75 \text{ sec}$$ (Eq 12.8-7)

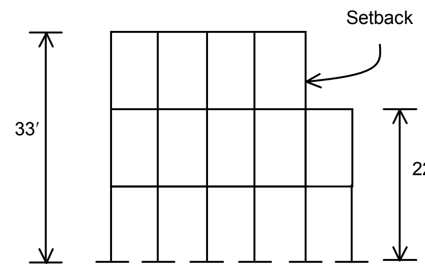
2. Determine the seismic response coefficient C_s §12.8.1

The design value of C_s is the smaller value of

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{(0.45)}{\left(\frac{8}{1.0}\right)} = 0.0562$$ (Eq 12.8-2)

2. Concrete special moment frame (SMF) structure

Height of the tallest part of the building is 33 feet, and this is used to determine period. Roof penthouses are generally not considered in determining h_n, but heights of setbacks are included. However, if the setback represents more than a 130-percent change in the lateral force system dimension, there is a vertical geometric irregularity (Table 12.3-2) and dynamic analysis is required for this type of irregularity per Table 12.6-1.



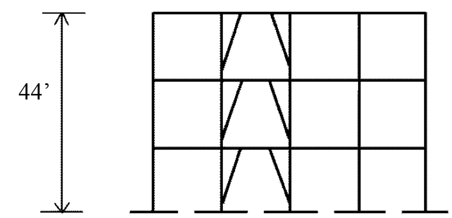
$$h_n = 33 \text{ feet}$$

$$C_T = 0.016;\ x = 0.9$$

$$T_a = C_T(h_n)^x = 0.016(33)^{0.9} = 0.37 \text{ sec}$$

3. Steel eccentric braced frame (EBF)

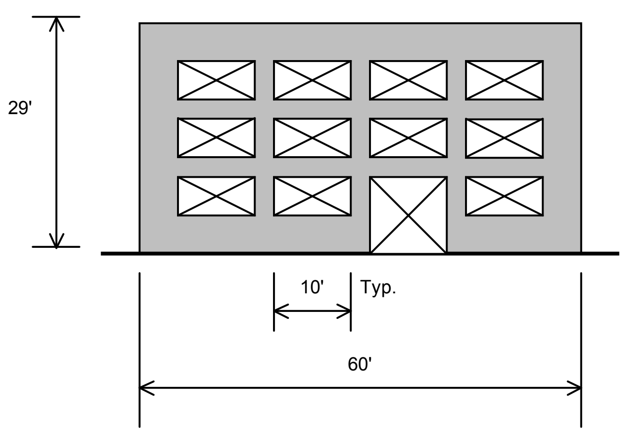
EBF structures use the $C_t = 0.03$ and $x = 0.75$.



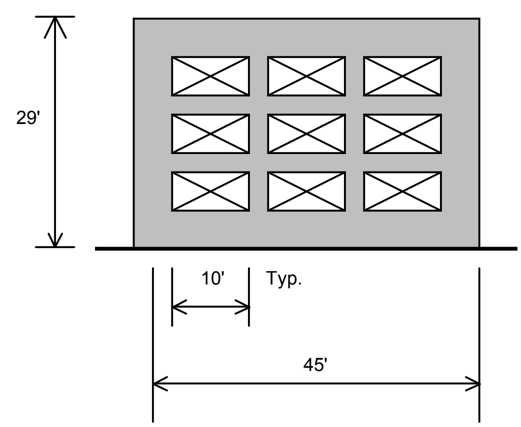
$$C_T = 0.030;\ x = 0.75$$

$$T = C_T(h_n)^x = 0.030(44)^{0.75} = 0.51 \text{ sec}$$

4. Masonry shear wall building



Front wall elevation



Back wall elevation

For this structure, C_T may be taken as 0.020 and x may be taken as 0.75, the values for “all other buildings”

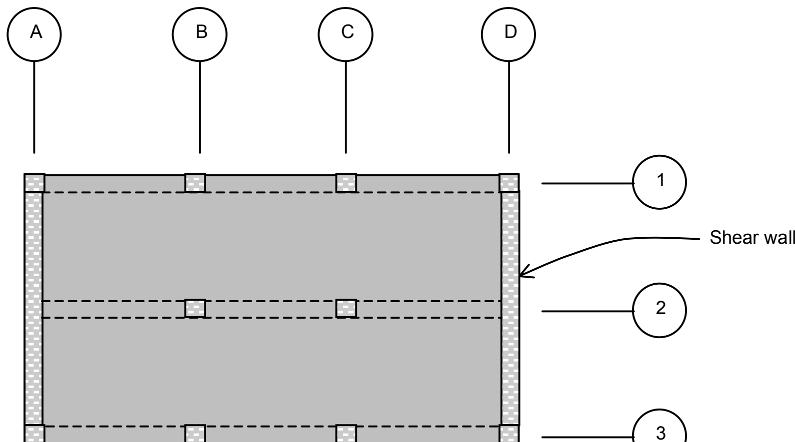
$$T_a = C_T(h_n)^x = 0.020(29)^{0.75} = 0.25 \text{ sec}$$

Example 21
Combination of Framing Systems in Different Directions §12.2.2

This example illustrates the determination of response modification coefficient R, system over strength factor Ω_o , and deflection amplification factor C_d values for a building that has different seismic framing systems along different axes (i.e., directions) of the building.

In this example, a three-story building has concrete shear walls in one direction and concrete moment frames in the other. Floors are concrete slab, and the building is SDC D and Occupancy Category II.

Determine the R, C_d, and Ω_o values for each direction.



Typical Floor Plan

Lines A and D are special reinforced concrete shear walls (bearing wall system)
$R = 5$, $\Omega_o = 2.5$, $C_d = 5$, Table 12.2-1 (A1)

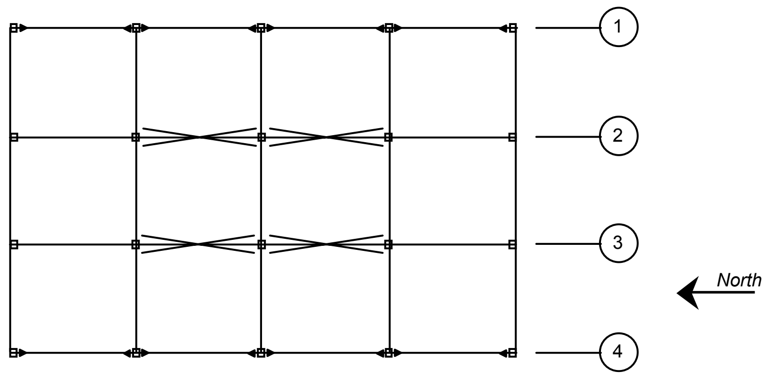
Lines 1, 2, and 3 are special reinforced concrete moment frames
$R = 8.0$, $\Omega_o = 3.0$, $C_d = 5.5$ Table 12.2-1 (C5)

1. Determine the *R* value for each direction

Example 22
Combination of Structural Systems: Along the Same Axis §12.2.3.2

Occasionally, it is necessary or convenient to have different structural systems in the same direction. This example shows how the response modification coefficient *R* value is determined in such a situation.

A one-story steel frame structure has the roof plan shown below. The structure is assigned to Occupancy Category II.



Roof Plan

Lines 1 and 4 are ordinary steel moment frames: $R = 3.5$

Lines 2 and 3 are special steel concentrically braced frames: $R = 6.0$

1. Determine the *R* value for the N/S direction

Calculations and Discussion — *Code Reference*

When a combination of structural systems is used in the same direction, §12.2.3.2 requires that (except for dual systems and shear wall-frame interactive systems) the value of *R* used shall not be greater than the least value of any system utilized in that direction.

∴ Use $R = 3.5$ for entire structure.

Commentary

An exception is given for light frame, <u>flexible</u> diaphragm buildings of Occupancy Category I or II two stories or less in height. However, to qualify as a flexible diaphragm, the lateral deformation of the diaphragm must be more than two times the average story drift of the associated story; see definition in §12.3.1.3.

2. Find F_x at each level

The vertical distribution of seismic forces is determined as

$$F_x = C_{vx} V \qquad \text{(Eq 12.8-11)}$$

where

$$C_{vx} = \frac{W_x h_x^k}{\sum_{i=1}^{n} W_i h_i^k} \qquad \text{(Eq 12.8-12)}$$

Since there are nine levels above the ground, $n = 9$

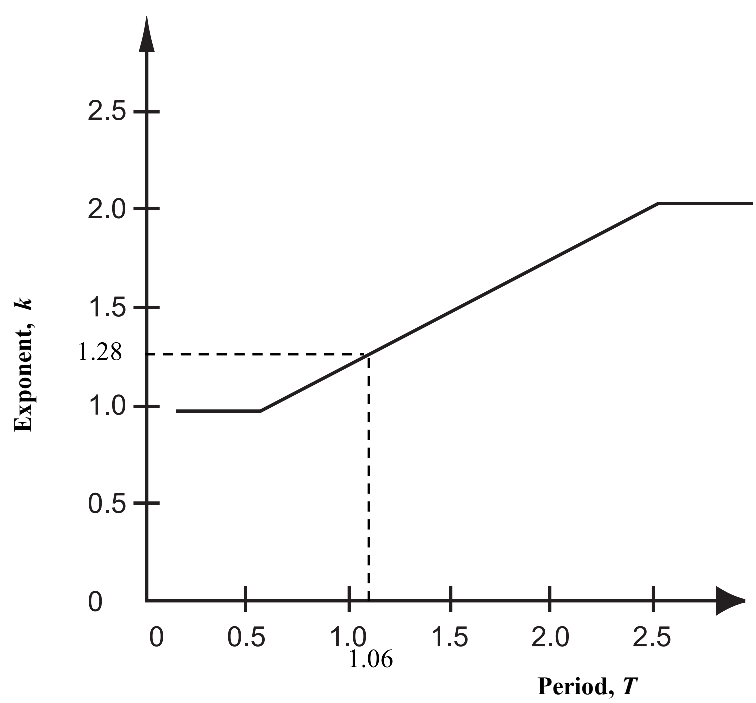
Thus:

$$F_x = \frac{233.2 w_x h_x^k}{\sum_{i=1}^{9} w_i h_i^k}$$

3. Find the distribution exponent k §12.8.3

The distribution exponent k is equal to 1.0 for buildings having a period of $T \leq 0.5$ seconds, and is equal to 2.0 for buildings having a period of $T \geq 2.5$. For intermediate value of the building period, k is determined by linear interpolation.

Thus:



Now:

$$\begin{aligned} \text{for} \quad T &= 1.06 \text{ sec} \\ k &= 1.0 + (1.06 - 0.5)\left(\frac{1}{2.5 - 0.5}\right) \\ &= 1.28 \end{aligned}$$

Use: $k = 1.28$

4. Equation 12.8-12 is solved in the table below given *V* = 233.8 kips and *k* = 1.28

Level x	h_x	h_x^k (ft)	w_x (kips)	$w_x h_x^k$ kip-ft	$C_{vx} = \dfrac{w_x h_x^k}{\Sigma w_i h_i^k}$	$F_x = C_{vx}V$ (kips)	$F_x/w_x = \overline{S_a}$
9	116 ft	439	214	93,946	0.116	27.3	0.127
8	104	382	405	154,710	0.192	44.8	0.111
7	92	326	405	132,030	0.169	38.3	0.094
6	80	273	405	110,565	0.137	32.1	0.079
5	68	222	584	129,648	0.161	37.6	0.064
4	56	173	422	73,006	0.091	21.2	0.050
3	44	127	422	53,594	0.067	15.5	0.037
2	32	84	440	36,960	0.046	10.8	0.024
1	20	46	465	21,390	0.027	6.2	0.013
			Σ = 3762	Σ = 806,289	1.004	233.2	

Commentary

Note that certain types of vertical irregularity can result in a dynamic response having a load distribution significantly different from that given in this section. Table 12.6-1 lists the minimum allowable analysis procedures for seismic design. Redundancy requirements must also be evaluated once the type of lateral-force-resisting system to be used is specified, because this may require modification of the building framing system and vertical distribution of horizontal forces as a result of changes in building period T.

Often, the horizontal forces at each floor level are increased when ρ is greater than 1.0. This is done to simplify the analysis of the framing members. The horizontal forces need not be increased at each floor level when ρ is greater than 1.0, provided that, when designing the individual members of the lateral-force-resisting system, the seismic forces are factored by ρ. When checking building drift, ρ = 1.0 (§12.3.4.1) shall be used.

Structures that have a vertical irregularity of Type 1a, 1b, 2, or 3 in Table 12.3-2, or horizontal irregularities of Type 1a or 1b in Table 12.3-1, may have significantly different force distributions. Structures with long periods, e.g., T > $3.5T_s$, require a dynamic analysis per Table 12.6-1 in Seismic Design Categories D, E, or F. In addition, some Irregular Structures require a dynamic analysis per Table 12.6-1. The configuration and final design of this structure must be checked for irregularities. Most structural analysis programs used today perform this calculation, and it is rarely necessary to manually perform the calculations shown above. However, it is recommended that these calculations be performed to confirm the computer analysis and to gain insight to structural behavior. Note that $(\overline{S_a})_{max}$ is approximately twice C_s, and $\overline{S_a} = \Gamma \phi S_a$ from a modal analysis.

Example 24
Horizontal Distribution of Shear §12.8.4

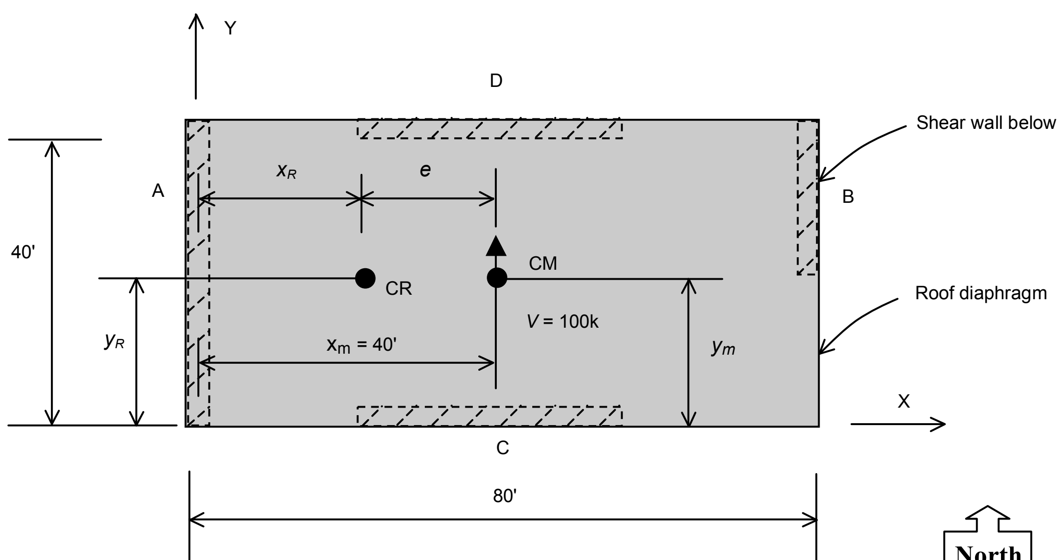
A single-story building has a rigid roof diaphragm. See appendix to this example for a procedure for the distribution of lateral forces in structures with rigid diaphragms and cross walls and/or frames of any orientation. Lateral forces in both directions are resisted by shear walls. The mass of the roof can be considered to be uniformly distributed, and in this example, the weight of the walls is neglected. In actual practice, particularly with concrete shear walls, the weight of the walls should be included in the determination of the center-of-mass (CM).

The following information is given.

Design base shear: V = 100 kips in north-south direction

Wall rigidities: R_A = 300 kip/in
R_B = 100 kip/in
$R_C = R_D$ = 200 kip/in
Center-of-mass: x_m = 40 ft, y_m = 20 ft

Analyze for seismic forces in north-south direction.



Roof plan

Determine the following.

1. **Eccentricity and rigidity properties**

2. **Direct shear in walls A and B**

3. **Plan irregularity requirements**

4. **Torsional shear in walls A and B**

5. **Total shear in walls A and B**

Calculations and Discussion	***Code Reference***

1. Eccentricity and rigidity properties §12.8.4.1

The rigidity of the structure in the direction of applied force is the sum of the rigidities of walls parallel to this force.

$$R = R_A + R_B = 300 + 100 = 400 \text{ kip/in}$$

The centers of rigidity (CR) along the x and y axes are

$$x_R = \frac{R_B(80 \text{ ft})}{R_A + R_B} = 20 \text{ ft}$$

$$y_R = \frac{R_C(40 \text{ ft})}{R_C + R_D} = 20 \text{ ft}$$

$$\text{eccentricity } e = x_m - x_R = 40 - 20 = 20 \text{ ft}$$

Torsional rigidity about the center of rigidity is determined as

$$J = R_A(20)^2 + R_B(60)^2 + R_C(20)^2 + R_D(20)^2$$

$$= 300(20)^2 + 100(60)^2 + 200(20)^2 + 200(20)^2 = 64 \times 10^4 \text{ (kip/in) ft}^2$$

The seismic force V applied at the CM is equivalent to having V applied at the CR together with a counter-clockwise torsion T. With the requirements for accidental eccentricity e_{acc}, the total shear on walls A and B can be found by the addition of the direct and torsional load cases.

The initial total shears are

$$V'_A = V_{D,A} - V'_{T,A} = 75.0 - 22.5 = 52.5 \text{ kips}$$

$$V'_B = V_{D,B} + V'_{T,B} = 25.0 + 22.5 = 47.5 \text{ kips}$$

(NOTE: This is not the design force for Wall A, as accidental eccentricity here is used to reduce the force).

The resulting displacements δ′, which for this single-story building are also the story drift values, are

$$\delta' = \frac{V'_A}{R_A} = \frac{52.5}{300} = 0.18 \text{ in}$$

$$\delta_B = \frac{V'_B}{R_B} = \frac{47.5}{100} = 0.48 \text{ in}$$

$$\delta_{avg} = \frac{0.18 + 0.48}{2} = 0.33 \text{ in}$$

$$\delta_{max} = \delta'_B = 0.48 \text{ in}$$

$$\frac{\delta_{max}}{\delta_{avg}} = \frac{0.49}{0.33} = 1.45 > 1.4$$

∴ Extreme Torsional Irregularity Type 1b exists. (See Example 26) Assuming SDC D, structural modeling must include 3 dimensions per §12.7.3, and diaphragm shear transfer forces to collectors must be increased 25 percent per §12.3.3.4.

Section 12.8.4.3 requires the evaluation and application of the torsional amplification factor

$$A_x = \left(\frac{\delta_{max}}{1.2_{avg}}\right)^2 = \left(\frac{0.48}{1.2(0.33)}\right)^2 = 1.47 < 3.0 \qquad (12.8\text{-}14)$$

Note: the factor A_x is not calculated iteratively (i.e., it is not recalculated with amplified torsion).

4. Torsional shears in walls A and B

To account for the effects of torsional irregularity, §12.8.4.2 requires that the accidental torsional moment, Ve_{acc}, be multiplied by the torsional amplification factor A_x.

	Force F_x Position		
	$\overline{X}_{c2}$	$\overline{X}_{c2} - e_{acc}$	$\overline{X}_{c2} + e_{acc}$
Wall shear V_A	185.0 kips	196.0 kips	174.0 kips
Wall shear V_B	115.0 kips	104.0 kips	126.0 kips
Story drift Δ_A	0.35 in	0.37 in	0.33 in
Story drift Δ_B	0.62	0.56	0.68
Level 2 displacement δ_A	0.80	0.85	0.75
Level 2 displacement δ_B	1.31	1.18	1.44

For the second story, find the following.

1. Maximum force in shear walls A and B

2. Check if torsional irregularity exists

3. Determine the amplification factor A_x

4. New accidental torsion eccentricity

Calculations and Discussion — *Code Reference*

1. Maximum force in shear walls A and B

The maximum force in each shear wall is a result of direct shear, inherent torsion (center of mass not being congruent with center of rigidity) and the contribution due to accidental torsion. As mentioned above, in this example it is assumed that accidental eccentricity is the only source of torsional moment at this floor level. From the above table, it is determined that

$$V_A = 196.0 \text{ kips}$$

$$V_B = 126.0 \text{ kips}$$

2. Check if torsional irregularity exists

The building may have a torsional irregularity Type 1 (Table 12.3-1). The following is a check of the story drifts.

$$\Delta_{max} = 0.68 \text{ in}$$

$$\Delta_{avg} = \frac{0.68 + 0.33}{2} = 0.51 \text{ in}$$

$$\frac{\Delta_{max}}{\Delta_{avg}} = \frac{0.68}{0.51} = 1.33 > 1.2$$

∴ Torsional irregularity Type 1a exists – Note: if $\Delta_{max}/\Delta_{avg}$ is larger than 1.4, then torsional irregularity Type 1b exists.

3. Determine the amplification factor A_x

Because a torsional irregularity exists, §12.8.4.3 requires that the second story torsional moment be amplified by the following factor. In this example, because the only source of torsion is the accidental eccentricity, the amplification factor will be used to calculate a new and increased accidental eccentricity, as shown below.

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

Where:

$$\delta_{max} = \delta_B = 1.44 \text{ in}$$

the average story displacement is computed as

$$\delta_{avg} = \frac{1.44 + 0.75}{2} = 1.10 \text{ in}$$

$$A_2 = \left(\frac{1.44}{(1.2)(1.10)}\right)^2 = 1.19$$

4. New accidental torsion eccentricity

Since A_2 (i.e., A_x for the second story) is greater than unity, a second analysis for torsion must be performed using the new accidental eccentricity.

$$e_{acc} = (1.19)(4.0 \text{ ft}) = 4.76 \text{ ft}$$

Calculations and Discussion ***Code Reference***

This example demonstrates the loading criteria and detailing required for elements supporting discontinued or offset elements of a seismic-force-resisting system.

Required strength

Because of the discontinuous configuration of the shear wall at the first story, the first story columns on lines A and D must support the wall elements above this level. Column C on line D is treated in this example. Because of symmetry, the column on line A would have identical requirements.

Section 12.3.3.3 requires that the column shall have a design strength to resist special seismic load combination of §12.4.3.2

$P_u = 1.2D + 0.5L + 1.0E_m$ §12.4.2.3 (Comb. 5)

$P_u = 0.9D + 1.0E_m$ §12.4.2.3 (Comb. 7)

where

$E_m = \Omega_o\, Q_E + 0.2\, S_{DS}\, D = 2.5(100) + 0.2(1.10)(40) = 259$ kips §12.4.3.2 (Comb. 5)

or $E_m = \Omega_o\, Q_E - 0.2\, S_{DS}\, D = 2.5(100) - 0.2(1.10)(40) = 241$ kips §12.4.3.2 (Comb. 7)

Substituting the values of dead, live, and seismic loads

$P_u = 1.2\,(40) + 0.5\,(20) + 259 = 317$ kips compression

and

$P_u = 0.9\,(40) - 0.5\,(241) = -84.5$ kips tension

2. Determine footing size

$P = D + 0.75(0.7E) = 50 + 0.75(0.7)(40) = 71$ kips (Comb. 5)

$P = D + 0.75[0.7(0.75)E + L]$ (Comb. 6)

$= 50 + 0.75[0.7(0.75)40 + 30] = 88$ kips

$P = 0.6D + 0.7(0.75)E$ (Comb. 7)

$= 0.6(50) + 0.7(0.75)(–40) = 9$ kips

Equation 6 governs. The required footing size is 88 kips/3.20 ksf = 27.5 sf
Use 5 ft, 3-in-square footing. A = 27.6 sf

3. Determine soil pressure reactions for strength design of footing

For the design of the concrete elements, strength design is used. The reduction in overturning does not apply, and the vertical seismic load effect is included

$P = 1.2D + 0.5L + E$ §2.3.2 (Comb. 5)

$= 1.2(50) + 0.5(30) + 40 + 0.2(1.0)(50) = 125\text{k}$

A uniform pressure of 125k/27.6 sf = 4.53 ksf should be used to determine the internal forces of the footing. (Note that if the footing also resisted moments, the pressure would not be uniform.)

The other seismic load combination is

$P = 0.9D – E$ §2.3.2 (Comb. 7)

$= 0.9(50) – 40 – 0.2(1.0)50 = –5\text{k}$

Note that this indicates uplift will occur. ASCE/SEI 7-05 does not require that foundation stability be maintained using strength-level seismic forces. This combination is only used here to determine internal forces of concrete elements of the foundation. As it results in no internal forces, it may be neglected.

2. Compare story drifts with the limit value §12.12

For this four-story building in Occupancy Category I, §12.12, Table 12.12-1 requires that the calculated design story drift shall not exceed 0.025 times the story height.

For SMF in SDC D, E, and F, this limit is reduced by ρ per §12.12.1.1:

$$\Delta a/\rho = 0.025h/1.3 = 0.0192h$$

Determine drift limit at each level

Levels 4, 3, and 2

$$\Delta \leq 0.0192h = 0.0192\ (12\ \text{ft} \times 12\ \text{in/ft}) = 2.76\ \text{in}$$

Level 1

$$\Delta \leq 0.0192h = 0.0192\ (16\ \text{ft} \times 12\ \text{in/ft}) = 3.68\ \text{in}$$

For $\Delta = \delta_x - \delta_{x\text{-}1}$, check actual design story drifts against limits

Level x	δ_{xe}	δ_x	Δ	Limit	Status
4	2.08 in	11.43 in	2.51 in	2.76	*o.k.*
3	1.62	8.92	2.68	2.76	*o.k.*
2	1.13	6.24	2.65	2.76	*o.k.*
1	0.65	3.59	3.59	3.68	*o.k.*

Therefore: The story drift limits of §12.12 are satisfied.

Note that use of the drift limit of 0.025*h* requires interior and exterior wall systems to be detail to accommodate this drift per Table 12.12-1

Commentary

Whenever the dynamic analysis procedure of §12.9 is used, story drift should be determined as the modal combination of the story-drift value for each mode.
Determination of story drift from the difference of the combined mode deflections may produce erroneous results because differences in the combined modal displacements can be less than the corresponding combined modal story drift.

Note that if the diaphragm is flexible, §12.11.2.1 requires the anchorage force (but not the wall force) to be increased.

The force F_p is considered to be applied at the mid-height (centroid) of the panel, but this must be uniformly distributed between the base and the top of parapet.

For the given $S_{DS} = 1.0$ and $I = 1.0$, the wall panel seismic force is

$$F_p = 0.40(1.0)(1.0)w = 0.40w$$

The weight of the panel between the base and the top of the parapet is

$$w_w = \left(\frac{8}{12}\right)(150)(24) = 2400 \text{ lb per foot of width}$$

$$F_p = 0.40\,(2400) = 960 \text{ lb/ft}$$

$$F_p > 400 \text{ lb/ft } S_{DS}I = 400(1)(1) = 400 \text{ lb/ft}$$

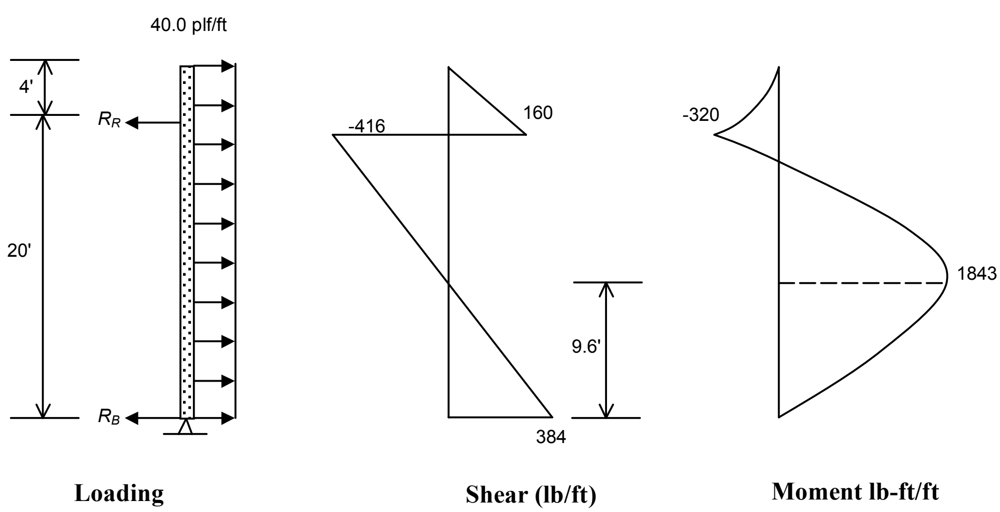
$$F_p > 280 \text{ lb/ft}$$

The force F_p is the total force on the panel. It acts at the centroid. For design of the panel for out-of-plane forces, F_p must be expressed as a distributed load f_p

$$f_p = \frac{960 \text{ lb/ft}}{24 \text{ ft}} = 40.0 \text{ plf/ft}$$

2. Shear and moment diagrams for wall panel design

Using the uniformly distributed load f_p, the loading, shear, and moment diagrams are determined for a unit width of panel. The 40.0 plf/ft uniform loading is also applied to the parapet. See step 3, below, for the parapet design load.



When the uniform load is also applied to the parapet, the total force on the panel is

$$40.0 \text{ plf/ft } (24 \text{ ft}) = 960 \ \frac{\text{lb}}{\text{ft}}$$

The reaction at the roof and base are calculated as

$$R_R = \frac{960(12)}{20} = 576 \text{ lb/ft}$$

$$R_B = 960 - 576 = 385 \text{ lb/ft}$$

The shears and moments are the Q_E load actions for strength design. Note that the reaction at the roof R_R is not necessarily the force used for wall-to-roof anchorage design, see 12.11.

3. Loading, shear and moment diagrams for parapet design §13.3.1

This section requires that the design force for parapets (note that parapets are classified as architectural components) be determined by Equation 13.3-1 with the Table 13.5-1 values of

$a_p = 2.5$ and $R_p = 2.5$ T 13.5-1

for the unbraced cantilever parapet portion of the wall panel.

The parapet is considered an element with an attachment elevation at the roof level

$$z = h$$

The weight of the parapet is

$$W_p = \left(\frac{8}{12}\right)(150)(4) = 400 \text{ lb per foot of width}$$

The concentrated force applied at the mid-height (centroid) of the parapet is

$$F_p = \frac{0.4 a_p S_{DS} I_p}{R_p}\left(1 + 2\frac{z}{h}\right) W_p \qquad \text{(Eq 13.3-1)}$$

$$F_p = \frac{0.4(2.5)(1.0)(1.0)}{2.5}\left(1 + 2\frac{20}{20}\right) W_p$$

$F_p = 1.2W_p = 1.2\ (400) = 480$ lb/ft $< 1.6\ S_{DS} I_p W_p = 640$ lb/ft . . . *o.k.* (Eq 13.3-2)

and $> 0.3\ S_{DS} I_p W_p$ . . . *o.k.* (Eq 13.3-3)

Example 35 Out-of-Plane Seismic Forces for Two-Story Wall Panel §12.11.1 and 12.11.2

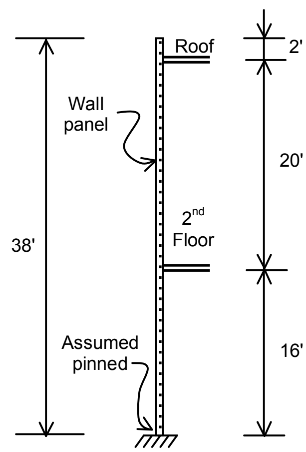
This example illustrates determination of out-of-plane seismic forces for the design of the two-story tilt-up wall panel shown below. A typical solid panel (no door or window openings) is assumed. Walls span from floor to floor to roof. The typical wall panel in this building has no pilasters and the tilt-up walls are bearing walls. The roof consists of 1-1/2-inch, 20-gage metal decking on open web steel joists and has been determined to be a flexible diaphragm. The second floor consists of 1-inch, 18-gage composite decking with a 2-1/2-inch lightweight concrete topping. This is considered a rigid diaphragm.

The following information is given.

Seismic Design Category D

S_{DS} = 1.0
I = 1.0

Wall weight = W_W = 113 psf



Wall section

Determine the following.

1. **Out-of-plane forces for wall panel design**

2. **Out-of-plane forces for wall anchorage design**

Calculations and Discussion *Code Reference*

1. Out-of-plane forces for wall panel design §12.11.1

Requirements for out-of-plane seismic forces are specified in §12.11.1

$$F_p = 0.40\, S_{DS} I w_w \geq 0.1 w_w$$

$$= 0.40(1.0)(1.0)w_w = 0.40w_w = 0.40(113)$$

$$= 45.2 \text{ psf}$$

b. Anchorage force for the rigid second floor diaphragm §12.11.2

For the case of rigid diaphragms the anchorage force is given by the greater of the following:

a. The force set forth in §12.11.1. $F_p = 0.4\ S_{DS}I$

b. A force of 400 $S_{DS}I$ (plf).

c. 280 (plf) of wall.

z = 16 ft = the height of the anchorage of the rigid diaphragm attachment, and W_p is the weight of the wall tributary to the anchor

$$W_p = \left[\left(\frac{20\text{ ft}}{2}\right) + \left(\frac{16\text{ ft}}{2}\right)\right](113\text{ psf}) = 2034\text{ plf}$$

$$F_p = 0.4(1.0)(1.0) = W_p = 0.4(2034) = 814\text{ plf}$$

$$F_p = 400\ S_{DS}I = 400(1.0)(1.0) = 400\text{ plf}$$

$$F_p = 280\text{ plf}$$

$\therefore F_p = 814$ plf controls

Calculations and Discussion ***Code Reference***

1. Collector unfactored force at tie to wall §12.10.2

The seismic force in the collector is made up of two parts: 1) the tributary out-of-plane wall forces, and 2) the tributary roof diaphragm force. The panelized wood roof has been determined to be flexible; thus the tributary roof area is taken as the 100-foot by 50-foot area shown on the roof plan above. Seismic forces for collector design are determined from Equation 12.10-1 used for diaphragm design. This equation reduces to the following for a single story structure.

$$F_{p1} = \frac{F_1}{W_1} w_{p1}$$

$$F_{p1} \max = 0.4\, S_{DS} I W_{p1} = 0.40 W_{p1}$$

$$F_{p1} \min = 0.2\, S_{DS} I W_{p1} = 0.2 W_{p1}$$

F_1 = design force at roof
W_1 = structure weight above one half $h_1 = W$
w_{p1} = weight tributary to the collector element

giving:

$$F_{p1} = \frac{V}{W} w_{p1} = 0.218 W_{p1}$$

w_{p1} = tributary roof and out-of-plane wall weight

$$w_{p1} = 15 \text{ psf}(100)(50) + 113 \text{ psf}\left(\frac{30}{2}\right)(100) = 75{,}000 + 169{,}500 = 244.5 \text{ kips}$$

$$\therefore F_{p1} = 0.218(244.5) = 53.3 \text{ kips.}$$

Note: This force corresponds to the diaphragm design forces calculated using §12.10.1. These forces are compared to the diaphragm shear strength; including the shear strength of connection between the diaphragm and collector. The design of the collector and its connections requires that the axial forces be amplified as shown below.

2. Special seismic load of §12.4.3.2 at tie to wall §12.10.2

Given the force F_{p1} specified by Equation 12.10-1, the collector elements, splices, and their connections to resisting elements shall have the design strength to resist the earthquake loads as defined in the Special Load Combinations of §12.4.3.2.

The governing load combination is

$$1.2\,D + 0.5L + E_m \qquad \text{§2.3.2 (Comb. 5)}$$

where

$$E_m = \Omega_o Q_E + 0.2\,S_{DS}D \qquad \text{(Eq 12.4-5)}$$

Here, Q_E is the horizontal collector design force F_{p1} = 53.3 kips, and

$\Omega_o Q_E = 2.5(53.3) = 133.25$ kips axial tension and compression load

$0.2\,S_{DS}D = 0.2(1.2)D = 0.24D$ vertical load

The strength design of the collector and its connections must resist the following load components.

$\Omega_o Q_E = 2.5(53.3) = 133.25$ kips axial tension and compression load
and vertical downward load equal to

$1.2D + 0.5L + 0.24\,D = 1.44\,D + 0.5\,L$

Assume tributary width for D and L is 16'.

with $D = (50 \text{ ft} + 50 \text{ ft})(16 \text{ ft})(15 \text{ psf}) = 24{,}000$ lb

$L = (50 \text{ ft} + 50 \text{ ft})(16 \text{ ft})(20 \text{ psf}) = 32{,}000$ lb

The resulting total factored vertical load is

$1.44(24{,}000) + 0.5(32{,}000) = 50{,}560$ lb

which is applied as a uniform distributed load w = 50,560 lb/50 ft = 1011 plf on the 50-foot length of the collector element.

Commentary

Note that §12.4.3.1 specifies that the term $\Omega_o Q_E$ in Equation 12.4-7 need not exceed the maximum force that can be delivered by the lateral-force-resisting system as determined by rational analysis. For example, the overturning moment capacity of the shear wall can limit the required strength of the collector and its connection to the shear wall.

2. Wall anchorage force

The tributary wall weight is one-half of the weight between the roof and base *plus* all the weight above the roof.

$$w_w = 150\left(\frac{8}{12}\right)(4 \text{ ft} + 10 \text{ ft})(1 \text{ ft}) = 1400 \text{ lb/ft}$$

For the given values of $S_{DS} = 1.0$ and $I = 1.0$, Equation 12.11-1 gives

$$F_p = 0.8(1.0)(1.0)w_p = 0.8w_p = 0.8(1400)$$

$$= 1120 \text{ lb/ft} > 400(1.0)(1.0) = 400 \text{ lb/ft} \ldots o.k.$$

$$> 280 \text{ lb/ft} \ldots o.k.$$

$$\therefore F_{anch} = F_p = 1120 \text{ lb/ft}$$

This is the Q_E load in the seismic load combinations.

Example 48
Determination of Diaphragm Force F_{px}: Highrise §12.10.1

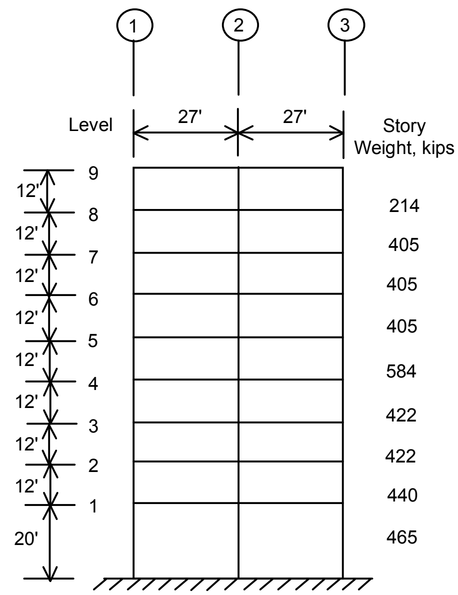
This example illustrates determination of the diaphragm design force F_{px} of Equation 12.10-1 for a representative floor of a multi-story building.

The nine-story moment frame building shown below has the tabulated design seismic forces F_x. These were determined from Equations 12.8-11 and 12.8-12, the design base shear.

The following information is given.

Seismic Design Category D

W = 3,762 kips
C_s = 0.06215
S_{DS} = 1.0
ρ = 1.3
I = 1.0
T = 1.06 sec
$V = C_sW$ = 233.8 kips
k = 2 for Eq 12.8-12



Level x	h_x (ft)	h_x^k	w_x kips	$w_x h_x^k$	$C_{vx} = \dfrac{w_x h_x^k}{\sum w_i h_i^k}$	$F_x = C_{vx}V$	$\dfrac{F_x}{w}$
9	116	13456	214	2879584	0.153	35.8	0.167
8	104	10816	405	4380480	0.233	54.4	0.134
7	92	8464	405	3427920	0.183	42.8	0.106
6	80	6400	405	2592000	0.138	32.3	0.079
5	68	4624	584	2700416	0.144	33.7	0.058
4	56	3136	422	1323392	0.071	16.6	0.039
3	44	1936	422	816992	0.044	10.3	0.024
2	32	1024	440	450560	0.024	5.6	0.013
1	20	400	465	186000	0.010	2.3	0.005
Totals:			3,762	18,757,344		233.8	

3. Separation from an adjacent building on *another* property §12.12.3

If Structure 1 is a building under design and Structure 2 is an existing building on an adjoining property, we would generally not have information about the seismic displacements of Structure 2. Often even basic information about the structural system of Structure 2 may not be known. In this case, separation must be based only on information about Structure 1. The largest elastic displacement of Structure 1 is δ_{3e} = 1.38 inches and occurs at the roof (Level 3). The maximum inelastic displacement is calculated as

$$\delta_M = \frac{C_d \delta_{3e}}{I} = \frac{5.5(1.38)}{1.0} = 7.59 \text{ in} \qquad \text{(Eq 12.8-15)}$$

Structure 1 must be set back 7.59 inches from the property line, unless a smaller separation is justified by a rational analysis based on maximum ground motions. Such an analysis is difficult to perform, and is generally not required except in very special cases.

4. Seismic separation between adjacent buildings

SEAOC recommends the following seismic separation between adjacent buildings.

$$\delta = \sqrt{(\delta_{M1})^2 + (\delta^2{}_{M2})}$$

where

$S_{DS} = 1.33$
$R = 8$
$I = 1.0$

The value of C_s computed in accordance with Equation 12.8-2 need not exceed

$$C_s = \frac{S_{DI}}{(R/I)T} = \frac{(1.0)}{(8/1.0)2.0} \text{ for } T \leq T_L = 0.063 \qquad \text{(Eq 12.8-3)}$$

where

$S_{DI} = 1.0$
$R = 8.0$
$I = 1.0$
$T = 2.0$

Check $T \leq T_L => T_L = 12.0$ sec (Region 1, F 22-16)

The value of C_s shall not be taken less than

$$C_s = \frac{0.5S_1}{(R/I)} = \frac{(0.5)(1.0)}{\left(\frac{8}{1.0}\right)} = 0.063 \qquad \text{(Eq 12.8-6)}$$

where

$S_1 = 1.0$ Note $S_1 \geq 0.6g$
$R = 8$
$I = 1.0$
$T = 2.0$

Thus: $C_s = 0.063$ Equations 12.8-3 and 12.8-6 govern.

$V = C_sW = (0.063)(300) = 18.9$ kips (Eq 12.8-1)

2. Vertical distribution of seismic forces §12.8-2

The design base shear must be distributed over the height of the structure in the same manner as that for a building structure.

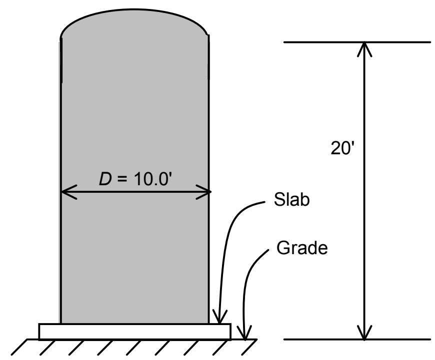
$F_x = C_{vx}\, V = C_{vx}$ (18.9 kips) (Eq 12.8-11)

Example 53
Tank With Supported Bottom §15.7.6

A small liquid storage tank is supported on a concrete slab. The tank does not contain toxic or explosive substances.

The following information is given.

S_{DS} = 1.20
I = 1.0
W = Weight of tank and maximum normal operating contents
= 120 kips
t = 0.50 inch



1. Find the design base shear §15.7.6

Calculations and Discussion **Code Reference**

1. The tank is a nonbuilding structure, and seismic requirements for tanks with supported bottoms are given in §15.7.6. This section requires that seismic forces be determined using the procedures of §15.4.2.

The period may be computed by other rational methods, similar to Example 51

$$T = 7.65 \times 10^{-6}\left(\frac{L}{D}\right)^2\left(\frac{w \times D}{t}\right)^{1/2}$$

where

L = 20 ft
D = 10 ft
L/D = 20/10 = 2.0
w = W/L = 120,000 lb/20 = 6000 plf
t = 0.50 in

$$\frac{wd}{t} = \frac{6000(10)}{(0.50/12)} = 1{,}440{,}000$$

Example i
Classification/Importance Factors §11.5-1
Seismic Design Category §11.6

Determine the importance factors and the seismic design category for a facility given the following information.

Type of occupancy – Elementary School with capacity greater than 250.
Per Table 1-1, Occupancy Category III

S_{DS} = 1.17
S_{D1} = 0.70
S_1 = 0.75

Determine the following.

1. Building Occupancy Category and Importance Factors for Occupancy Category III and for a building to be used for an emergency shelter

2. Seismic Design Category (SDC)

Calculations and Discussion — *Code Reference*

1. Building category and importance factors.

From Table 1-1, "Occupancy Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake and Ice Loads," the Occupancy Category for an Elementary School with an occupancy capacity greater than 250 is an Occupancy Category III. The Occupancy Category is used to determine the "Seismic Design Category," per Section 11.6-1. If the elementary school is to be used for an emergency shelter, the Occupancy Category is IV.

The importance factors for seismic loads are from Table 11.5-1. Importance factors for snow loads are from Table 7-4. Importance factors for wind loads are from Table 6-1.

Category	Seismic Factor I	Snow Factor I	Wind Factor I
III	1.25	1.1	1.15
IV	1.5	1.20	1.15

2. Seismic Design Category

All structures are assigned to a Seismic Design Category (SDC) based on their Occupancy Category and the spectral response acceleration coefficients S_{DS} and S_{D1}, irrespective of the fundamental period of vibration of the structure T. Each building and structure shall be assigned to the most severe SDC in accordance with Table 11.6-1 or 11.6-2 as follows.

Table 2.1 Occupancy Category vs Seismic Design Category

Nature of Occupancy	Occupancy Category	Table 11.6-1		Table 11.6-2		SDC USE*
		S_{DS}	SDC	S_{D1}	SDC	
School	III	1.17	D*	0.70	D*	E
Emergency Shelter	IV	1.17	D*	0.70	D*	F

Recall: $S_1 = 0.75\%$ for this table

*Note that for Occupancy Categories I, II, & III having S_1 equal to or greater than 0.75 (recall $S_1 = 0.75$), the building shall be assigned to SDC E. Also for Occupancy Category IV having $S_1 \geq 0.75$, the building shall be assigned to SDC F.

Calculations and Discussion	*Code Reference*

1. Maximum considered earthquake spectral response accelerations §11.4.1

For the given position (Near Sonora – NW of Sacramento, California) of 38° North (Latitude = 38.123°) and 121.123° West (Longitude = – 121.123°), USGS provides the values of

$$S_S = 57.3\%g = 0.573g$$

$$S_1 = 23.0\%g = 0.230g$$

2. Site coefficients and adjusted maximum considered earthquake spectral response accelerations §11.4.3

From the USGS for the given site class *D*, and $S_S = 0.573g$, $S_1 = 0.230g$, the site coefficients are as follows

$F_a = 1.34$ T 11.4-1

$F_v = 1.94$ T 11.4-2

The adjusted maximum considered earthquake spectral response accelerations (based on §11.4.3) are also given on the CD ROM as follows

$S_{MS} = F_a S_s = 1.34(0.573g) = 0.768g$ (Eq 11.4-1)

$S_{M1} = F_v S_1 = 1.94(0.230g) = 0.446g$ (Eq 11.4-2)

3. Design spectral response acceleration parameters §11.4.4

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.768g) = 0.512g \qquad \text{(Eq 11.4-3)}$$

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.446g) = 0.297g \qquad \text{(Eq 11.4-4)}$$

4. General procedure response spectrum §11.4.5

For periods less than or equal to T_o, the design spectral response shall be given by

$$S_a = 0.6\frac{S_{DS}}{T_o}T + 0.4\,S_{DS} \qquad \text{(Eq 11.4-5)}$$

For periods greater than or equal to T_o and less than or equal to T_s, the design spectral response acceleration S_a shall be taken equal to S_{DS}

For periods greater than T_s, and less than T_L, the design spectral response acceleration S_a shall be given by

$$S_a = (S_{D1}) / T \qquad \text{(Eq 11.4-6)}$$

Where: $T_o = 0.20\,(S_{D1} / S_{DS})$

$= 0.2\,(0.297 / 0.512)$

$= 0.12$ sec

$T_s = S_{D1} / S_{DS}$

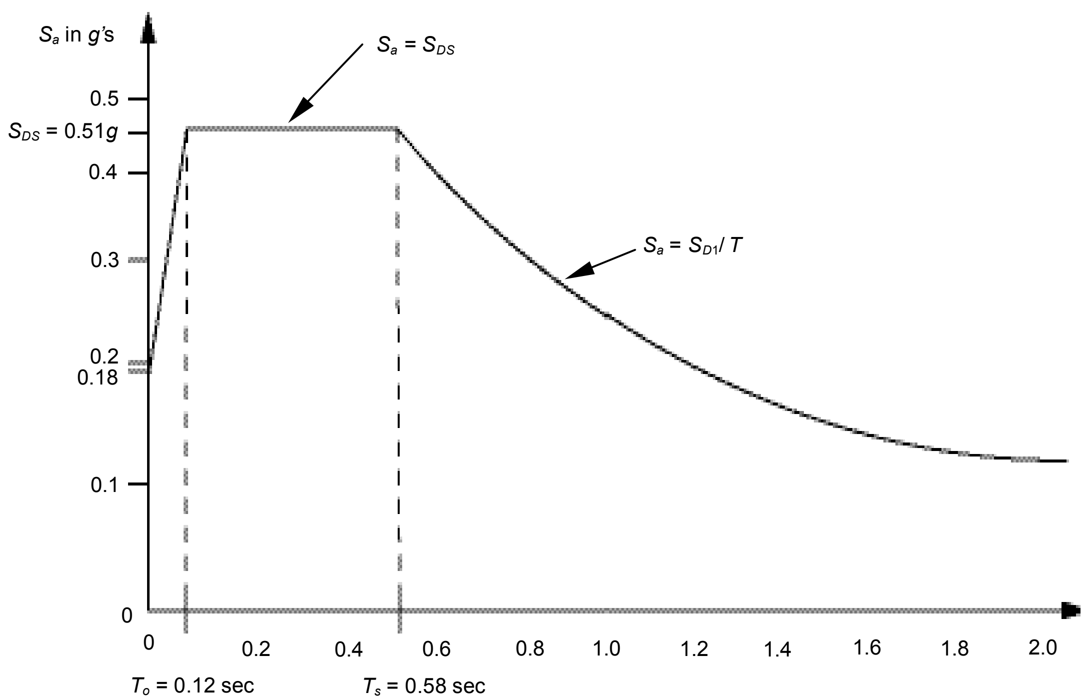
$= 0.297 / 0.512$

$= 0.58$ sec

$T_L = 8$ sec (F 22-15)

Thus:

T = Period	S_a/g	Computation for S_a
0.00	0.20	0.4 (0.51)
0.12	0.51	0.51
0.58	0.51	0.51
0.80	0.41	0.30 / 0.8
1.00	0.25	0.30 / 1.2
1.20	0.21	0.30 / 1.4
1.40	0.19	0.30 / 1.6
1.60	0.17	0.30 / 1.8
2.00	0.15	0.30 / 2.0



General Procedure Response Spectrum

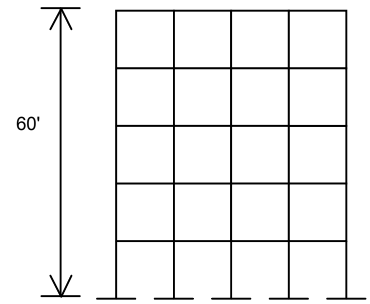
Example 17
Seismic Base Shear §12.8.1

Find the design base shear for a 5-story steel special moment-resisting frame building shown below.

The following information is given.

Seismic Design Category D

S_{DS} = 0.45g
S_{D1} = 0.28g
I = 1.0
R = 8
W = 1626 kips
h_n = 60 feet



To solve this example, follow these steps.

1. **Determine the structure period**

2. **Determine the seismic response coefficient C_s**

3. **Determine seismic base shear**

Calculations and Discussion — *Code Reference*

1. Determine the structure period §12.8.2.1

The appropriate fundamental period T_a is to be used. C_T for steel moment-resisting frames is 0.028.

$$T_a = C_T(h_n)^{3/4} = 0.028(60)^{0.8} = 0.74 \text{ sec} \qquad \text{(Eq 12.8-7)}$$

2. Determine the seismic response coefficient C_s §12.8.1

The design value of C_s is the smaller value of

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{(0.45)}{\left(\frac{8}{1.0}\right)} = 0.0562 \qquad \text{(Eq 12.8-2)}$$

and

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{(0.28)}{\left(\frac{8}{1.0}\right)(0.74)} = 0.047 \text{ for } T \le T_L \qquad \text{(Eq 12.8-3)}$$

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I}\right)} = \text{ for } T > T_L \qquad \text{(Eq 12.8-4)}$$

but shall not be less than

$$C_s = 0.01 \qquad \text{(Eq 12.8-5)}$$

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

$$C_s = \frac{0.5S_1}{\left(\frac{R}{I}\right)} \qquad \text{(Eq 12.8-6)}$$

$\therefore$ Design value of $C_s = 0.0467$

3. Determine seismic base shear §12.8.1

The seismic base shear is given by

$$V = C_sW \qquad \text{(Eq 12.8-1)}$$

$$= 0.047(1626 \text{ kips})$$

$$= 76.4 \text{ kips}$$

Commentary

The S_{D1} value of 0.28g given in this example is based on an S_1 value of 0.21g. If the S_1 value were to have been equal or greater than 0.6g, then the lower bound on C_s is

$$C_s \ge \frac{0.5IS_1}{R} \qquad \text{(Eq 12.8-6)}$$

Calculations and Discussion **Code Reference**

1. Diaphragm force at Level 7 §12.10-1

Seismic forces on the floor and roof diaphragm are specified in §12.10-1. The following equation is used to determine the diaphragm force F_{px} at Level x

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

Section 12.10.1.1 also has the following limits on F_{px}

$$0.2\, S_{DS} I w_{px} \leq F_{px} \leq 0.4 S_{DS} I w_{px}$$

For Level 7, $x = 7$

$$F_{p7} = \frac{(42.8 + 54.4 + 35.8)(405)}{(405 + 405 + 214)} = (0.130)(405) = 52.6 \text{ kips}$$

Check limits:

$0.2\, S_{DS} I w_{px} \quad = 0.2 w_{px}$

$= 0.2(405) = 81.1$ kips > 52.6 kips . . . **not** *o.k.*

$0.4\, S_{DS} I w_{px} \quad = 0.4 w_{px}$

$= 0.4(405) = 162.0$ kips > 52.6 kips . . . *o.k.*

$\therefore\ F_{p7} = 81.1$ kips…minimum value ($0.2\, S_{DS} I w_{px}$) governs.

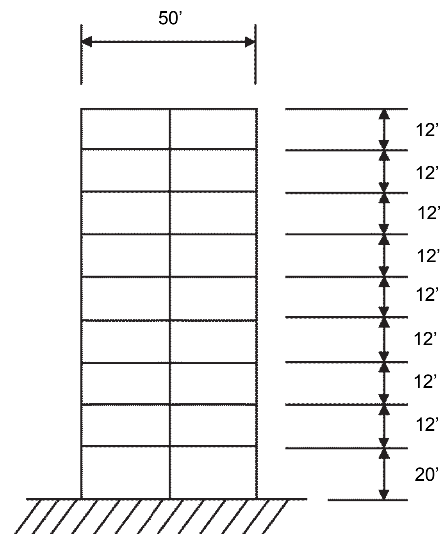
Note that the redundancy factor, in this example $\rho = 1.3$, is to be applied to the load Q_E due to F_{px} (such as chord forces and floor-to-frame shear connections). Also note that Equation 12.10-1 will always govern for the design of the diaphragm versus Equation 12.8-12.

Example 57
Wind Loads – Analytical Procedure §6.5

A 9-story building has a moment-resisting frame for a lateral force-resisting system.

Find the lateral forces on the frame due to wind.

Office building 50 ft by 50 ft in plan with MWFRS at exterior. Located in an urban/suburban area of N.W. Texas



Elevation

Determine:

1. **Wind loads on MWFRS**

Calculations and Discussion

1. **Wind loads on MWFRS** **Chapter 6**

1a. **Determine basic wind speed**

Utilize ASCE/SEI 7-05 §6

Use method 2 analytical procedure §6.5

Confirm building is regular shaped and not subject to across wind loading, vortex shedding, instability due to galloping or flutter; or does not have a site location for which channeling effects or buffeting in wake of upwind obstructions warrant special conditions §6.5.1

Design procedure §6.5.3

Basic wind speed V = 90 mph §6.5.4, F 6-1

1b. Determine velocity pressure

Wind directionality factor K_d = 0.85 §6.5.4.4, T 6-4
(applies when using load combinations in ASCE/SEI 7-05 §2.3 and §2.4)

Importance factor I = 1.00 §6.5.5, T 6-1
(Structural Category II, Table 1-1)

Exposure Category B §6.5.6

Velocity pressure coeff K_z (Case 2) §6.5.6.6, T 6-3

h	Exposure B Case 2	
0-15 ft	0.57	
20	0.62	
25	0.66	
30	0.70	
40	0.76	
50	0.81	
60	0.85	
70	0.89	
80	0.93	
90	0.96	
100	0.99	
116	1.03	← By Interpolation
120	1.04	

Topographic factor K_{Zt} = 1 §6.5.7
(example building on flat land, no nearby hills)

Gust effect factor G §6.5.8
9-story building
Natural period = 0.1(9) = 0.9 sec §12.8.2.1

Natural frequency $= \dfrac{1}{0.9} = 1.1$ Hz >1.0 (Eq 12.8-7)

Therefore: Rigid structure §6.2
G = 0.85 §6.5.8.1