DETERMINATION OF THE STRUCTURE FUNDAMENTAL PERIOD

Each structure fundamental period of vibration is influenced by the stiffness and height of the structure. ASCE 7 §12.8.2 provides three methods for determining the fundamental period of the structure, *T*. The general approximate method, the approximate method for moment resisting frames and the rational analysis method

The ***general approximate method*** calculates the fundamental period of the structure from ASCE 7 Eq. 12.8-7 of:

$$T_a = C_t\ h_n^{\ x}$$

Where:

- h_n = structural height, in feet, of the roof above the base, not including any parapet or penthouse.
- C_t and x = empirical data parameters obtained from ASCE 7 Table 12.8-2.

Steel Moment Resisting Frames (Special, Intermediate, Ordinary):	$\boldsymbol{T_a=0.028\ h_n^{0.8}}$
Concrete Moment Resisting Frames (Special, Intermediate, Ordinary):	$\boldsymbol{T_a=0.016\ h_n^{0.9}}$
Steel Eccentrically Braced Frames; Steel Eccentrically Braced Frames with Special Moment Resisting Frames (Dual System); Steel Buckling-Restrained Braced Frames:	$\boldsymbol{T_a=0.03\ h_n^{0.75}}$
All Other Structures:	$\boldsymbol{T_a=0.02\ h_n^{0.75}}$

For ***Moment Resisting Frame (Steel or Concrete) buildings***, with number of stories above the base not greater than 12 and with average story height of at least 10 ft, the approximate fundamental period can be determined from ASCE 7 (Eq. 12.8-8):

$$T_a = 0.1N$$

where *N* is the number of stories above the base. See Fig. 5.3

Rational analysis method: ASCE 7 permits the use of a *properly substantiated analysis* (e.g. *Rayleigh Method, or Dynamic/Modal Analysis*) to determine the fundamental period, provided that the value of the period determined using this method, $T_{rational}$, may not exceed the value of:

$$T_{rational} \leq C_u T_a$$

Where C_u is a function of S_{D1} (ASCE 7, Table 12.8-1) as given in Table 5.5 below, and T_a is the approximate period calculated using the ***general approximate method*** (ASCE 7, Eq. 12.8-7).

$= \delta_x - \delta_{x-1}$

$= C_d (\delta_{xe} - \delta_{xe-1}) / I_e$

V_x = Story shear force, acting between levels x and x-1.

h_{sx} = Story height between levels x and x-1.

C_d = Deflection amplification factor from ASCE 7 Table 12.2-1.

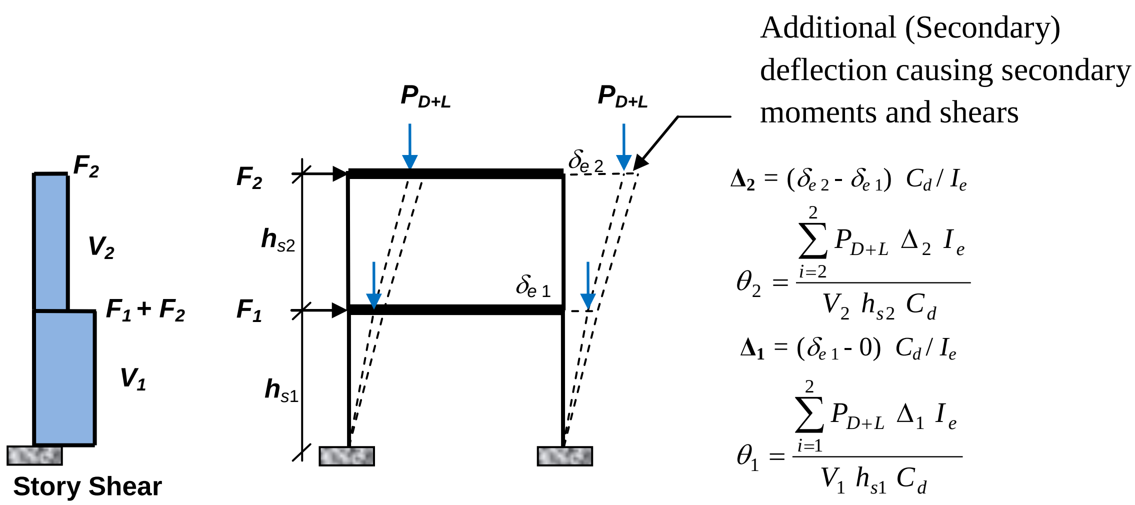


Fig. 5.17 P-Δ Effects

The stability coefficient, θ, shall not exceed, θ_{max}, value given by ASCE 7 Eq. 12.8-17

$$\theta_{\max} = \frac{0.5}{\beta . C_d} \leq 0.25$$

Where β is the ratio of shear demand to shear capacity of the story between levels x and level x-1. Conservatively, it can be assumed that β =1.0.

- If $\theta \leq 0.10$ the P-Delta effect can be ignored.
- If $0.10 < \theta \leq \theta_{max}$, rational analysis is required (i.e., second order analysis considering the deformed geometry of the building), or alternatively, the computed forces and story drift shall be amplified by multiplying times the factor $\dfrac{1}{1-\theta}$. The amplified drift shall be used to check against the drift limits, and the members design shall be checked against amplified forces.
- If $\theta > \theta_{max}$, the structure must be redesigned (i.e., too flexible).
- If rational analysis is used to include effects of P-Delta, ASCE 7 Eq. 12.8-17 must still be satisfied, however it is permitted to revise the check to: $\dfrac{\theta}{1+\theta} \leq \theta_{\max}$
- In determining drift for the P-Delta effects, the upper limit for the fundamental period, T, specified in ASCE 7 §12.8.2 is waived. That is: $\boldsymbol{T \leq C_u . T_a}$ does not apply to drift calculations.

Table 5.14 Conditions for Redundancy Factor, ρ = 1.0, for SDC = D, E, or F

Seismic Force Resisting System	Conditions for Each Story Resisting More than 35% of the Base Shear*
Braced Frames	Removal of individual brace or its connection would not result in more than 33% reduction in the story strength
Moment Frames	Loss of moment resistance at beam-column connections at both ends of a single beam would not result in more than 33% reduction in the story strength
Shear Walls & Wall Piers (height /length) > 1.0	Removal of a shear wall or wall pier with height-to-length ratio > 1.0, within any story, or collector connection would not result in more than 33% reduction in the story strength
Cantilever Columns	Loss of moment resistance at the base connections of any single cantilever column would not result in more than 33% reduction in the story strength

* *Any removal or loss change in any of the described SFRS would not result in a system having an extreme torsional irregularity (horizontal irregularity Type 1b)*

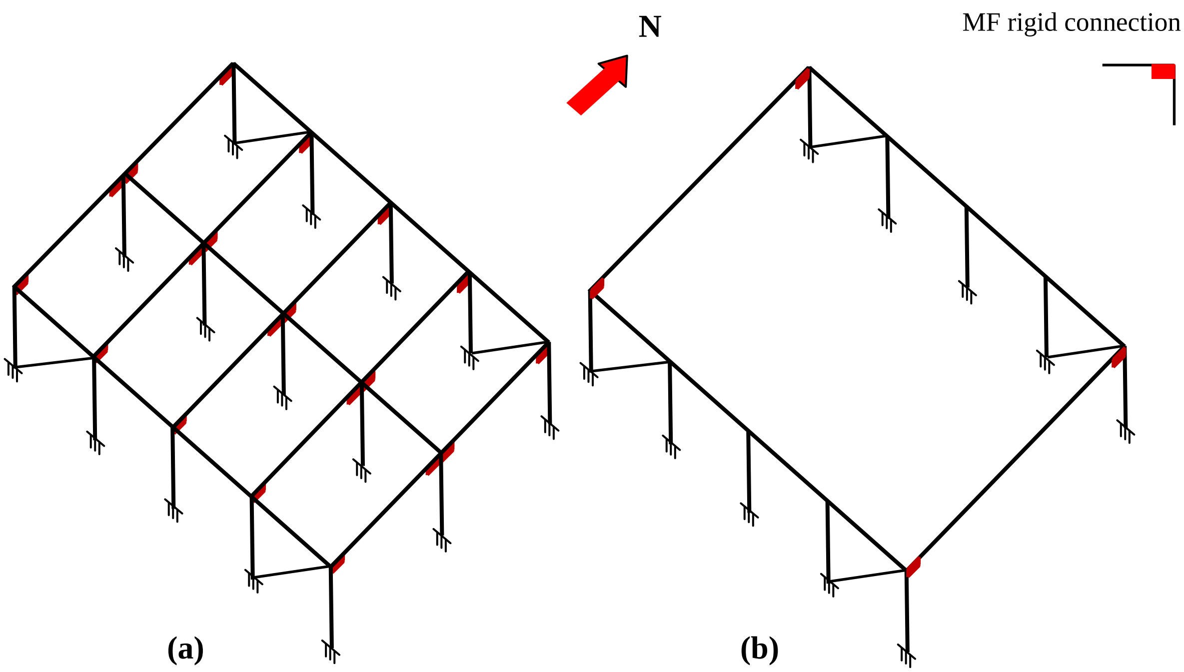


Fig. 5.22 Redundancy in Structural Systems (a) Redundancy in Both Directions, $\rho_{N\text{-}S}$ =1.0, $\rho_{E\text{-}W}$ =1.0, (b) Redundancy in E-W Direction only, $\rho_{N\text{-}S}$ =1.3, $\rho_{E\text{-}W}$ =1.0

Basic Load Combinations

Buildings and other structures and components shall be designed to resist load combinations specified in IBC and ASCE 7. Structures may be designed based on:

- Basic Strength Design (SD) or Load and Resistance Factor Design (LRFD).
- Basic Allowable Stress Design (ASD).
- Alternative Basic Allowable Stress Design.

In addition, special structural elements may require the use of special seismic load combinations of ASCE 7 §12.4.3.2, Load Combinations with Overstrength Factor.

Each load combination uses superposition to consider simultaneous effects of gravity loads (dead load, live load, snow load, etc.) and lateral loads (earthquake, wind, flood, etc.). The following loads are defined herein and used in the following sections:

D : Dead load.
E : Seismic load effect.
E_m : Maximum seismic load effect with overstrength factor per ASCE 7 §12.4.3.
F : Fluid loads.
H : Earth and ground water pressure.
L : Live load (except roof) including any reductions.
L_r : Roof live load.
R : Rain load.
S : Snow load.

1) *Basic SD/LRFD Combinations*

There are seven Basic SD/LRFD load combinations in IBC, however only two combinations include seismic load effects, E. The first one represents the maximum gravity effect where the seismic effect is in the same direction as gravity (that is, the same sign for dead load, D and seismic, E). The second combination represents the minimum gravity effect where the seismic effect is in the opposite direction to gravity (that is, a different sign for dead load, D and seismic, E).

- **Maximum Effect*** (IBC Eq. 16-5):
  $1.2(D+F)+1.0E+f_1L+1.6H+f_2S$ OR $(1.2+0.2S_{DS})D+1.2F+\rho Q_E+f_1L+1.6H+f_2S$

- **Minimum Effect**** (IBC Eq. 16-7):
  $0.9(D+F)+1.0E+1.6H$ OR $(0.9-0.2S_{DS})D+0.9F+\rho Q_E+1.6H$

* Maximum effect: same sign for D and Q_E
** Minimum effect: opposite sign for D and Q_E

where:

f_1 = 1.0 for places of public assembly live loads > 100 psf, and parking garages;
= 0.5 for other live loads.

f_2 = 0.7 for roof configurations that do not shed snow off the structure (e.g., saw tooth configuration);
= 0.2 for other roof configurations

Exception:
Where other factored load combinations are specifically required by IBC provisions (i.e. Chapter 19, Concrete), such combinations shall take precedence over the basic combinations in Chapter 16.

2) ***Basic ASD Combinations***

There are nine Basic ASD load combinations in IBC, however only three include seismic forces, *E*. The first two combinations represent the maximum gravity effect, and the third one represents the minimum gravity effect as discussed above.

- **Maximum Effect*** (IBC Eq.16-12 & 16-14):
  $D+H+F+0.7E$ <u>*OR*</u> $(1.0+0.14S_{DS})D+H+F+0.7\rho Q_E$

  $D+H+F+0.75(0.7E)+0.75L+0.75S$ <u>*OR*</u>
  $(1.0+0.105S_{DS})D+H+F+0.75(0.7\rho Q_E)+0.75L+0.75S$

- **Minimum Effect**** (IBC Eq. 16-16):
  $0.6(D+F)+0.7E+H$ <u>*OR*</u> $(0.6 - 0.14S_{DS})D+0.6F+0.7\rho Q_E +H$

* Maximum effect: same sign for D and Q_E.
** Minimum effect: opposite sign for D and Q_E.

Exception:
See IBC §1605.3.1 for exceptions to crane hook loads and flat roof snow loads of $\leq$ 30 *psf*.

Note:
<u>Increase in allowable stresses</u> shall **NOT** be used with Basic ASD load combinations.

3) ***Alternative Basic ASD Combinations***

Alternative Basic ASD combinations can be used <u>in lieu of Basic ASD</u> combinations. There are total of six Alternative Basic ASD load combinations, however only the following two combinations include seismic effects for the maximum and minimum gravity effects as discussed earlier.

- **Maximum Effect*** (IBC Eq. 16-21):
  $D+L+S+E/1.4$ <u>*OR*</u> $(1.0 + 0.14S_{DS})\ D+L+S+\rho Q_E/1.4$

- **Minimum Effect**** (IBC Eq. 16-22):
  $0.9D+E/1.4$ <u>*OR*</u> $(0.9 - 0.14S_{DS})\ D+\rho Q_E/1.4$

* Maximum effect: same sign for D and Q_E.
** Minimum effect: opposite sign for D and Q_E.

Exception:
See IBC §1605.3.2 for exceptions to crane hook loads and flat roof snow loads of $\leq$ 30 *psf*.

5.36 What is the response modification coefficient, *R*, that should be used for the design of the first story, and fifth story, respectively?

A 6, 6

B 8, 8

C 6, 8

D 8, 6

5.37 What is the overstrength factor, Ω_0, that should be used for the design of the first story, and fifth story, respectively?

A 1.25, 2.0

B 2.0, 3.0

C 2.0, 2.5

D 2.0, 2.0

5.38 What is the deflection amplification factor, C_d, that should be used for determining the displacement of the first story, and fifth story, respectively?

A 3.25, 5.0

B 5.0, 5.0

C 5.5, 5.0

D 5.5, 5.5

Given a 2 story building assigned to SDC = C. The building uses seismic force resisting system in the North-South direction composed of steel special moment frames at the exterior sides of the building, and steel special plate shear walls at all of the inside lines of resistance. Moment frames are designed to resist 20% of the base shear, *V*, in the North-South direction. Answer MCQs 5.39 and 5.40.

5.39 What is the response modification coefficient, *R*, that should be used for the design of the North-South direction?

A 6

B 7

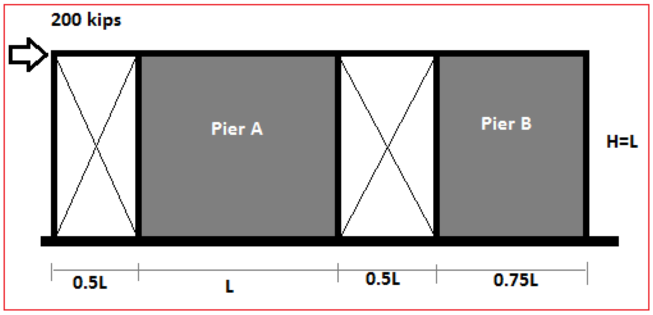
C 7.5

D 8

5.40 What is the overstrength factor, Ω_0, that should be used for the design of the North-South direction?

Example 6.6:

Two piers A, and B have the same thickness and modulus of elasticity and are fixed at the top. Assuming lateral force F = 200 kips is applied to the wall, what is the force resisted by each pier?.



Since the piers are assumed fixed at the top and are fixed to the foundation, refer to Table 6.2. Using *H/D* ratios of 1.0 and 1.33, the rigidities of piers A & B are estimated as R_A = 2.5 and R_B = 1.577. No correction is needed for thickness and modulus of elasticity, therefore the force resisted by each pier will be:

$$F_A = \frac{R_A}{R_A + R_B} F = \frac{2.5}{2.5 + 1.577}(200) = 122.6 kips$$

$$F_B = 200 - 122.6 = 77.4 kips$$

Note:
There are generally two accepted methods for the determination of shear wall deflections and hence rigidity for one or two story buildings: a cantilever wall with fixity only at the base, or fixity at each floor level.

Problem statement indicated a fixed - fixed conditions for the given wall piers. However, if no information were given in the problem statement, it is generally conservative to assume a cantilever wall conditions (i.e. Table 6.1)

6.20 Assuming concrete walls, what is the maximum force in wall C? Assume concrete walls have fixed-fixed conditions.

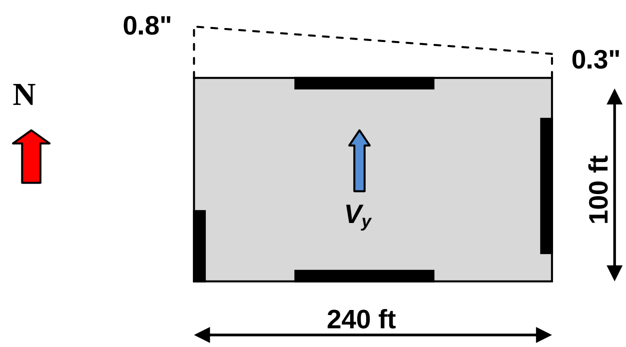
A 4.0 kips

B 6.7 kips

C 9.3 kips

D 9.8 kips

Plan view of the rigid roof diaphragm of a one story police station *(I_e = 1.5)* is shown. Elastic theoretical displacements, δ_{xe}, along one side are also shown. The seismic base shear was calculated as *V*=200 kips. The building has torsional horizontal irregularity. Given a story height of 16 feet, and deflection amplification factor, C_d, equals 4. SDC = B. Answer MCQs 6.21 and 6.22.



6.21 Determine the accidental eccentricity amplification factor, A_x (ASCE 7 §12.8.4.3)?

A 1.0

B 1.25

C 1.47

D 3.0

6.22 What is the story drift ratio (Δ_1 / h_{s1})?

A 0.005

B 0.008

C 0.012

D 0.020

ANCHORAGE OF STRUCTURAL WALLS

The anchorage of structural walls to supporting construction (diaphragm) shall be capable of resisting the following (ASCE 7 §12.11.2.1):

$$F_p = 0.4\ S_{DS}\ k_a\ I_e\ W_p \qquad \text{(ASCE 7 Eq. 12.11-1)}$$

with a minimum value $$F_p \geq 0.2\ k_a\ I_e\ W_p$$

Where:

F_p = the design force in the individual anchor.
S_{DS} = design spectral response acceleration for short period (0.2 sec).
I_e = the importance factor.
W_p = the weight of the wall tributary to the anchor (*plf*), that is:
$w_{wall}\ (h_{w1}/2 + h_{w2}/2)$ for the anchor at the 2nd floor diaphragm, or
$w_{wall}\ (h_{w2}/2 + h_{parapet})$ for the anchor at the roof diaphragm with a parapet.
h_{w1} *and* h_{w2} are the height of the first story and second story wall and $h_{parapet}$ is the height of the attached parapet. See Fig. 7.2.
k_a = amplification factor for diaphragm flexibility
for *Rigid Diaphragm* $k_a = 1.0$
for *Flexible Diaphragm* $k_a = 1.0 + L_f / 100$
where L_f is the span, in feet, of the flexible diaphragm that supports the wall. The span is measured between the vertical elements that provide the lateral support to the diaphragm in the direction considered. See Fig. 7.3.

$$1.0 \leq k_a \leq 2.0$$

The force, F_p, shall have the same unit as the weight of the wall tributary to the anchor, W_p, that is, *plf*, pound per 1 foot strip along the wall length, at the level of the diaphragm being anchored.

Exception:
When the anchorage is <u>not</u> located at the roof and all diaphragms are <u>not</u> flexible, the anchorage force, F_p, is permitted to be reduced by multiplying times the factor:

$$(1 + 2\ z / h) / 3$$

Where:
z = the height of the anchor above the base, and
h = the height of the roof above the base.
Referring to Fig. 7.2, and considering F_p, for anchoring the diaphragm at the second floor:

$$z = h_{w1}, \text{ and } h = h_{w1} + h_{w2}.$$

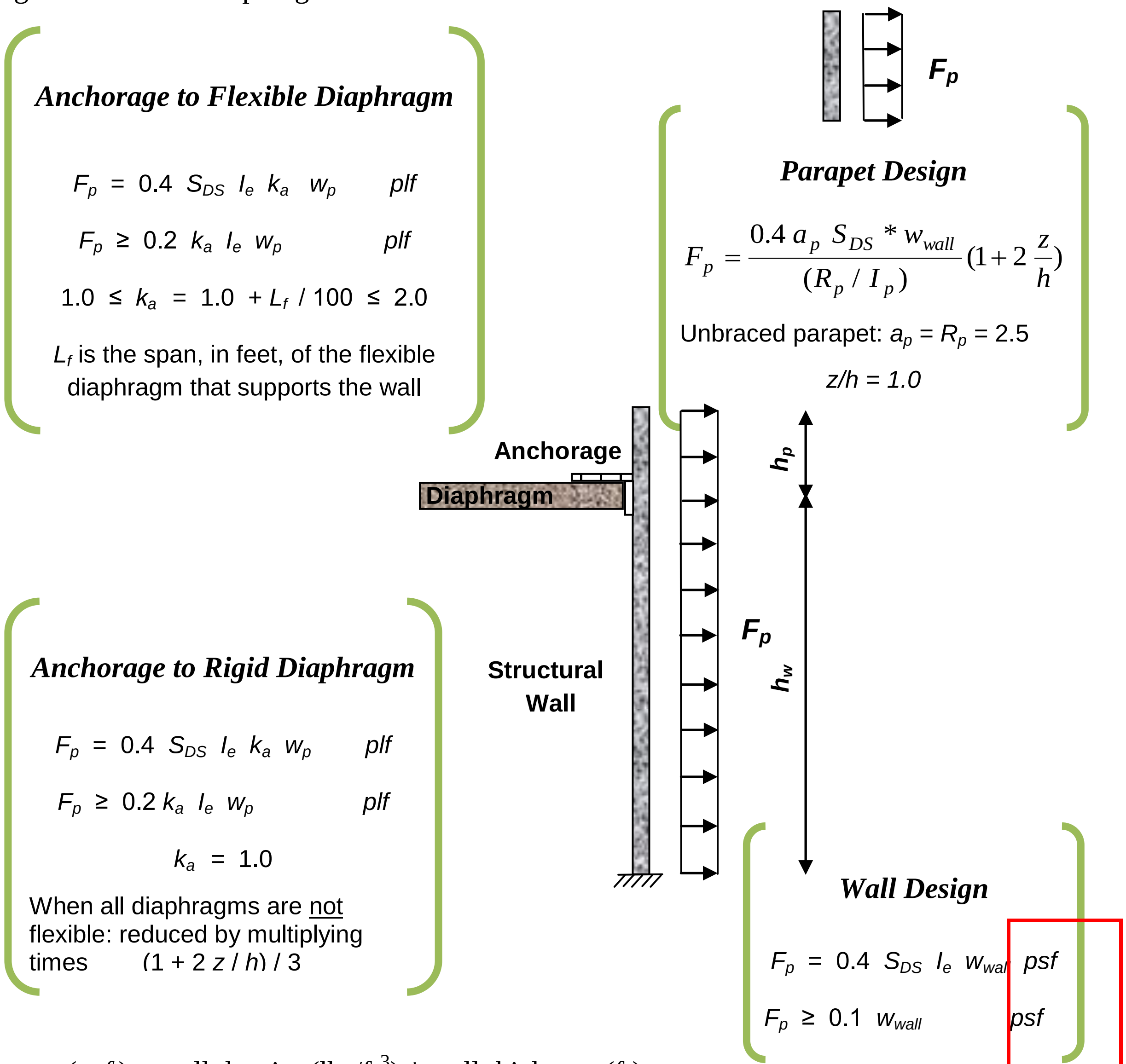
The reduced F_p shall not be less than the minimum value of $(0.2\ k_a\ I_e\ W_p)$.

Notes:

- If the spacing between the anchors exceeds 4 ft, the walls shall be designed to resist bending between the anchors.
- The F_p force is given at the Strength Design level (i.e., Q_E).

- ASCE 7 §13.5.2 required application of the F_p force, determined using ASCE 7 Eq. 13.3-1, at the center of gravity of the parapet to obtain the design moment and shear.
- However, in determining the design moment and shear for walls (with parapets) and the design force in the anchorage, ASCE 7 §12.11.1 is applied to the entire wall including the parapet.
- Note that for unbraced parapets, $a_p = R_p = 2.5$.

Figure 7.5 summarizes the different requirements for structural walls, parapets, anchorage to rigid and flexible diaphragms.



w_{wall} (psf) = wall density (lbs/ft^3) * wall thickness (ft)

w_p (plf) = $w_{wall} * (h_w/2 + h_p)$

Note: For a wall with no parapet, $h_p = 0.0$

Fig. 7.5 Summary of Design Requirements for Structural Walls, Parapets, Anchorage to Rigid and Flexible Diaphragms.

Maximum seismic forces in each bolt:

Seismic force *V* will be resisted by the four bolts in shear and either axial tension or axial compression.

Shear per bolt = 6.24 / 4 = 1.56 kips

Axial force per bolt is determined from the overturning moment.

The overturning moment will be resisted by <u>two</u> bolts on each side of the transformer.

OTM = V * 10 ft = 6.24 * 10 = 62.4 kips.ft

Resolving the OTM into force couple at the base gives the axial compression and axial tension (two bolts in compression and two bolts in tension):

C = T = OTM / 8 ft / 2 bolts = 62.4 / (8 * 2) = 3.90 kips / bolt

Thus: (assuming Compression (+), Tension (−))

Two bolts on one side, each subjected to:

shear = 1.56 kips and axial compression of +3.9 kips

Two bolts on the other side, each subjected to:

shear = 1.56 kips and axial tension of −3.9 kips

Maximum compression force per bolt at the basic SD/LRFD level (strength):

Each bolt is subjected to gravity force in addition to the seismic forces. To determine the maximum compression per bolt, use the basic SD/LRFD load combination of IBC §1605 for **maximum effects** (where <u>seismic force is additive to gravity</u>)

Maximum compression = $(1.2 + 0.2\, S_{DS})D + 1.2F + \rho Q_E + f_1 L + 1.6H + f_2 S$

where $F = L = H = S = 0.0$, with D and Q_E having the same sign.

$\rho = 1.0$ for non-building structures not similar to buildings

Dead load (Compression) = 16 kips per four bolts

D per bolt = 16 / 4 = +4 kips

Q_E = compression seismic force = +3.9 kips

Maximum compression = (1.2 + 0.2 * 1.30) 4 + 1.0 * 3.9 = 9.74 kips

Maximum tension force per bolt at the basic SD/LRFD level (strength):

Each bolt is subjected to gravity force in addition to the seismic forces. To determine the maximum tension per bolt (i.e., minimum compression), use the basic SD/LRFD load combination of IBC §1605 for **minimum effects** (where <u>seismic force is counteractive to gravity</u>)

Maximum tension = $(0.9 - 0.20 S_{DS})D + 0.9\,F + \rho Q_E + 1.6\,H$

where $F = H = 0.0$, with D and Q_E having opposite signs.

$\rho = 1.0$ for non-building structures not similar to buildings,

D per bolt = 16 / 4 = +4 kips

Q_E = tension seismic force = −3.9 kips

Maximum tension = (0.9 − 0.20 * 1.30) 4 + 1.0 * (−3.9)
= (0.9 − 0.26) 4 − 3.9
= 2.56 − 3.9 = −1.34 kips Tension

Note: if the result above indicated positive sign, then the compression dead load had exceeded seismic tension.

Maximum tension force per bolt at the basic ASD level (allowable design):
Each bolt is subjected to gravity force in addition to the seismic forces. To determine the maximum tension per bolt (i.e., minimum compression), use the basic ASD load combination of IBC §1605 for **minimum effects** (where seismic force is counteractive to gravity)

Maximum allowable tension = $(0.6 - 0.14S_{DS})D + 0.6F + 0.7\rho Q_E + H$
where $F = H = 0.0$, with D and Q_E having opposite signs.
$\rho = 1.0$ for non-building structures not similar to buildings,
D per bolt = 16 / 4 = +4 kips
Q_E = tension seismic force = −3.9 kips

$$\begin{aligned}\text{Maximum allowable tension} &= (0.6 - 0.14 * 1.30)\,4 + 0.7 * 1.0 * (-3.9)\\ &= (0.6 - 0.182)\,4 - 2.73\\ &= 1.672 - 2.73 = -1.058 \text{ kips Tension}\end{aligned}$$

Note: if positive sign, then the compression dead load had exceeded seismic tension.

Part II: Transformer pad supported on the roof of a building

Seismic shear force of the transformer:
Since the transformer is mounted on the roof of the building, and assuming that the weight of the transformer is less than 25% of the total combined weight, Thus transformer pad is considered electrical component attached to the building.

$$F_P = \frac{0.4 a_P . S_{DS} . W_P}{(R_P / I_P)} (1 + 2\frac{z}{h}) \qquad \text{ASCE 7 13.3-1}$$

W_p = 16 k
I_p = 1.5 Electrical component is attached to building with RC = IV, Table 7.1
S_{DS} = 1.30
Since the transformer is attached to the roof: z/h = 1.0
a_p = component amplification factor = 1.0 Table 7.3 (ASCE 7 Table13.6-1)
R_p = component response modification factor = 2.5 Table 7.3 (ASCE 7 Table13.6-1)

$$F_P = \frac{0.4 * 1.0 * 1.30 * 16}{(2.5/1.5)} (1 + 2 \times 1)$$

$$F_P = (0.936)\, W_P$$

$$F_P = (0.936)\, 16.0 = 14.976 \; kips$$

Check upper and lower limit of F_p

$$0.3 S_{DS} . I_P . W_P \le F_P \le 1.6 S_{DS} . I_P . W_P$$

$$0.3 * 1.30 * 1.5 * 16 \le F_P \le 1.6 * 1.30 * 1.5 * 16$$

$$9.36 \le F_P \le 49.92$$

Aspect Ratio for WSP Horizontal Diaphragm

Size and shape of horizontal diaphragm shall be limited in accordance with AF&PA SDPWS Table 4.2.4. The maximum length to width ratio of a wood structural panel horizontal diaphragm and other sheathing materials are shown in Table 11.1.

Table 11.1 Maximum Diaphragm Aspect Ratio

Horizontal Diaphragm	**Aspect Ratio *L/W***
Wood Structural Panel	
Unblocked	**3 : 1**
Blocked	**4 : 1**
Single straight lumber sheathing	**2 : 1**
Single diagonal lumber sheathing	**3 : 1**
Double diagonal lumber sheathing	**4 : 1**

Thickness of wood structural panel horizontal diaphragms shall be determined according to IBC Tables 2306.2(1), and 2306.2(2) (or SDPWS Tables 4.2A, 4.2B and 4.2C) depending on sheathing grade, framing arrangement and magnitude of seismic loading.

Deflection of wood horizontal diaphragm shall be limited to the permissible deflection of the attached distributing and resisting elements such as walls (IBC §2305.2).

Allowable Shear Capacity for WSP Horizontal Diaphragm

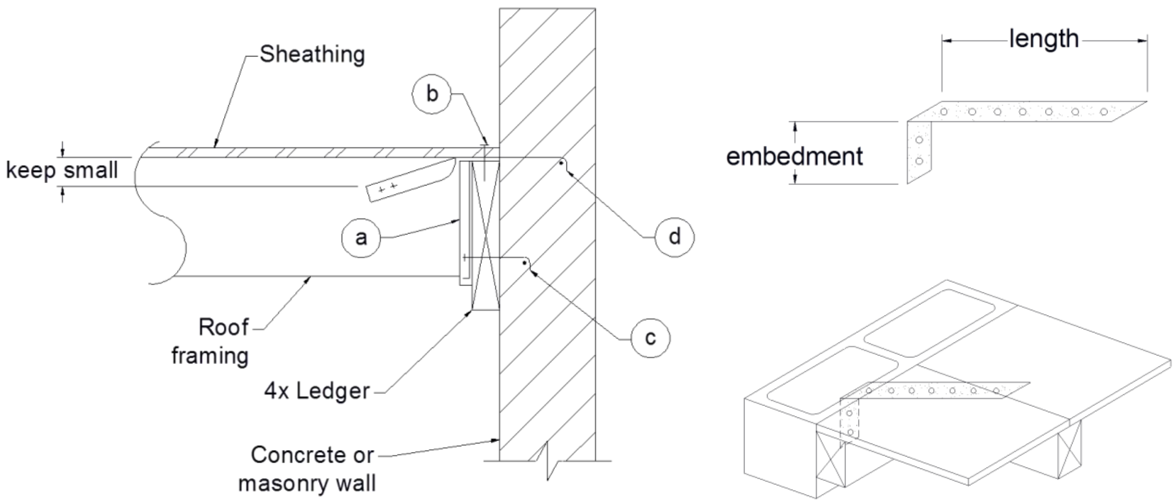
The strength of the wood structural panel horizontal diaphragm depends on sheathing thickness, grade, and orientation; the width of the framing members; support of the panel edges; and the nail spacing, type and penetration.

- IBC Tables 2306.2(1) and 2306.2(2) give the allowable design shear for horizontal Wood Structural Panel diaphragms with Douglas Fir Larch or Southern Pine framing for wind and seismic loading (lb/ft = plf), where the panels are fastened to framing members with staples.
- SDPWS Tables 4.2A, 4.2B, and 4.2C give the nominal design shear for horizontal Wood Structural Panel diaphragms with Douglas Fir Larch or Southern Pine framing for wind and seismic loading (lb/ft = plf), where the panels are fastened to framing members with nails (common or galvanized box).

Note that AF&PA SDPWS nominal design shear values are 2 times the design shear values used with Allowable Stress Design, ASD, load combination and 1.25 times the design shear values used with Load and Resistance Factor Design, LRFD, load combination.

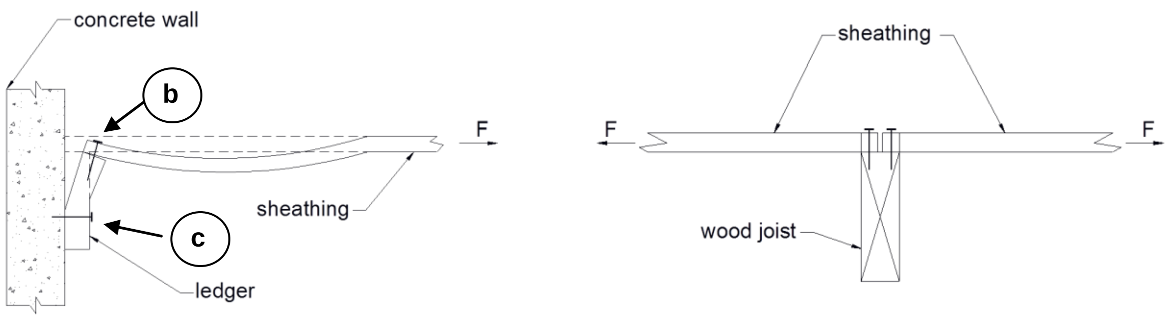
Thus: for WSP diaphragms fastened with nails (common or galvanized box):

- Use SDPWS Tables to determine the nominal design shear capacity.
- Multiply the Table nominal values by (0.5) when using ASD load combination (allowable capacity).
- Multiply the Table nominal values by (0.8) when using LRFD load combination (strength capacity).



(a) Ledger and Joist connection **(b)** Alternative Tie connection

Fig. 11.11 Typical Ledger Anchorage to a Concrete or Masonry Wall



(a) **(b)**

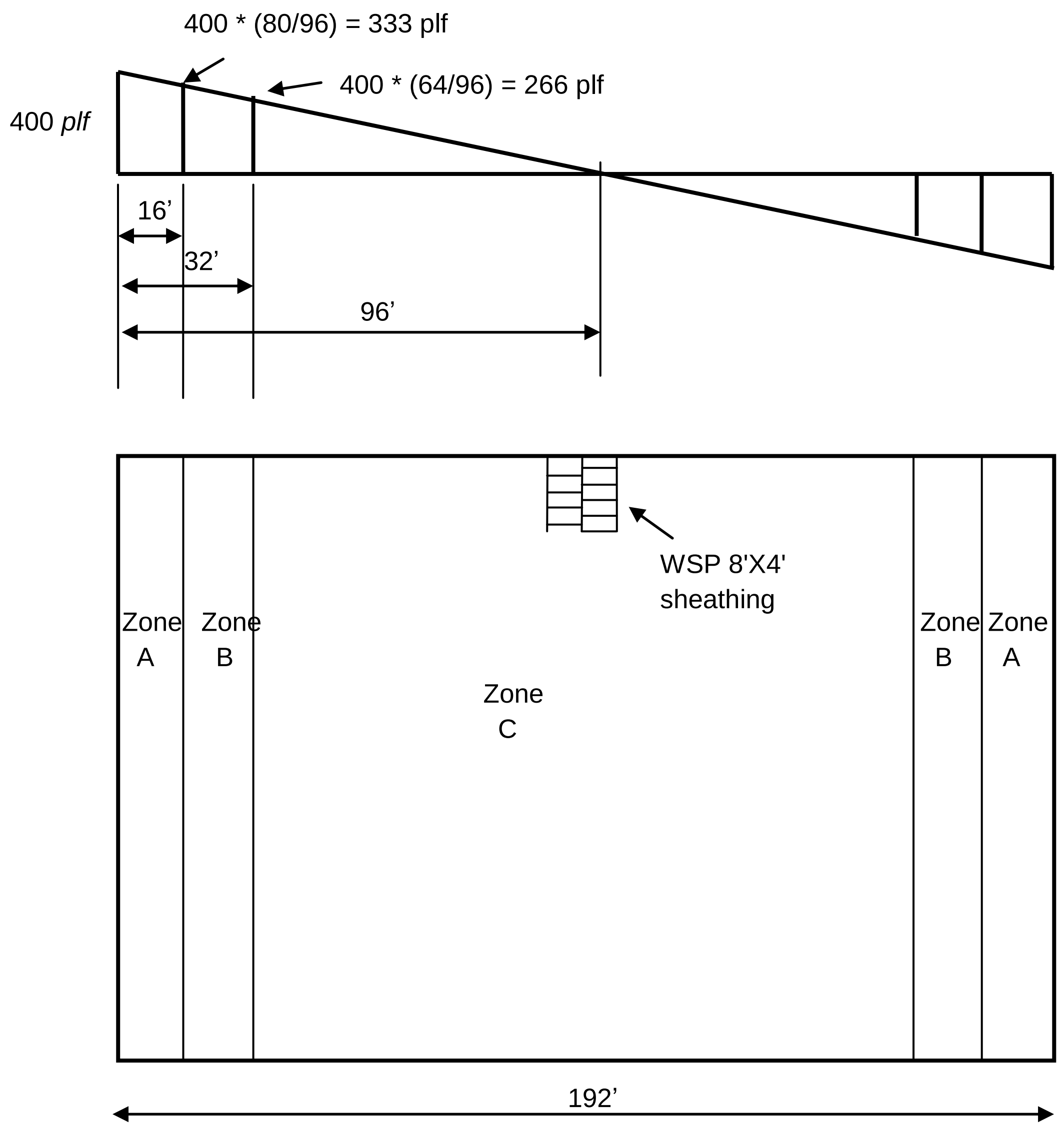
Fig. 11.12 a) Cross Grain Bending, b) Cross Grain Tension

CRIPPLE WALLS

Cripple walls are short walls that support the first story floor when the building is constructed above the original ground (i.e., raised structure).

Cripple walls shall be framed of studs, not less in size than the studs above it, extending from the foundation to the first level and can vary in height. To prevent the failure of these wall studs during an earthquake, and hence damage to the entire structure above, IBC requires bracing of the cripple wall studs.

- Short cripple walls (less than 14 in.) require solid blocking (IBC §2308.9.4).
- For cripple walls with stud height greater than 14 in., IBC requires that they be braced in accordance with Table 2308.9.3 (1) for SDC = A, B and C.
- For SDC = D and E, IBC §2308.12.4 shall apply.

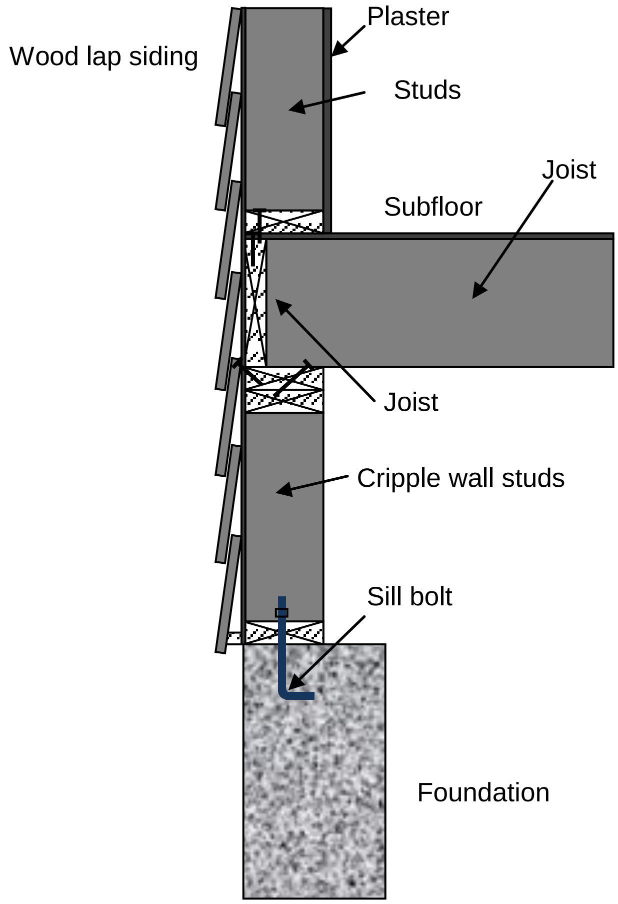


Determine the maximum <u>allowable</u> chord force for the seismic loading in the N-S direction

Loading is given at the ASD level. (i.e., no need to multiply times 0.7)

Maximum chord force = $w_s L^2/(8*d)$ = 500 * $(192)^2$ /(8*120) = 19200 lbs (Allowable Stress Design Level)

D yielding of anchor bolt



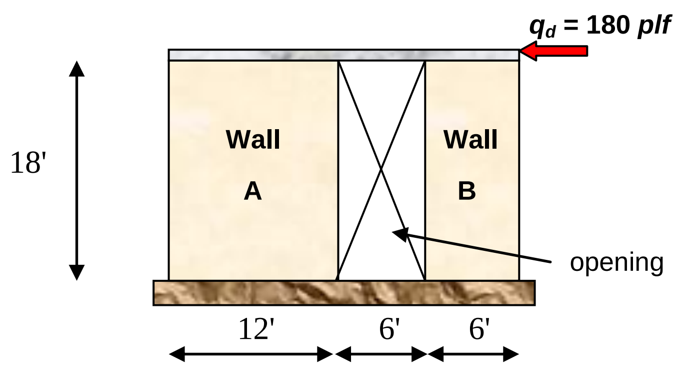
11.30 Figure shows two light frame wood walls with flexible roof diaphragm. Determine the force for the hold down device at the ends of <u>Wall A</u>. Assume roof dead load = 140 *plf* and Wall A self weight = 20 *psf*. Use segmented shear wall method (i.e., ignore self weight of Wall B). Diaphragm unit shear, q_d, is given at 180 plf. Given redundancy factor, ρ = 1.0, the seismic response coefficient, C_S = 0.22, and S_{DS} = 1.2. Use ASD load combination of IBC §1605.3.

A 520 *lbs*

B 780 *lbs*

C 1220 *lbs*

D 2227 *lbs*



CANTILEVER RETAINING WALLS

Cantilever retaining walls may be constructed from concrete, masonry, or steel and timber. The wall resists the horizontal active earth pressure that has a distribution similar to hydrostatic (water) pressure. The cantilever walls are checked against: Overturning, Sliding, and Soil Bearing pressure if supported on shallow foundations. Furthermore, stem and footing are structurally designed as reinforced concrete components.

Design of Cantilever Retaining Walls for Seismic Loading

In addition to the static earth pressure, earthquake will exert extra active earth pressure on the wall. The coefficient of active seismic earth pressure (K_{AE}) for cohesionless soils was developed by Mononobe-Okabe in 1920's. The resultant seismic force is considered to act between one-half to two-thirds of the retained height of the wall. In some cases, the Geotechnical Report may simplify the seismic force to an equivalent uniform soil pressure. In that case the resultant seismic force will act in the middle of the retained height of the wall.

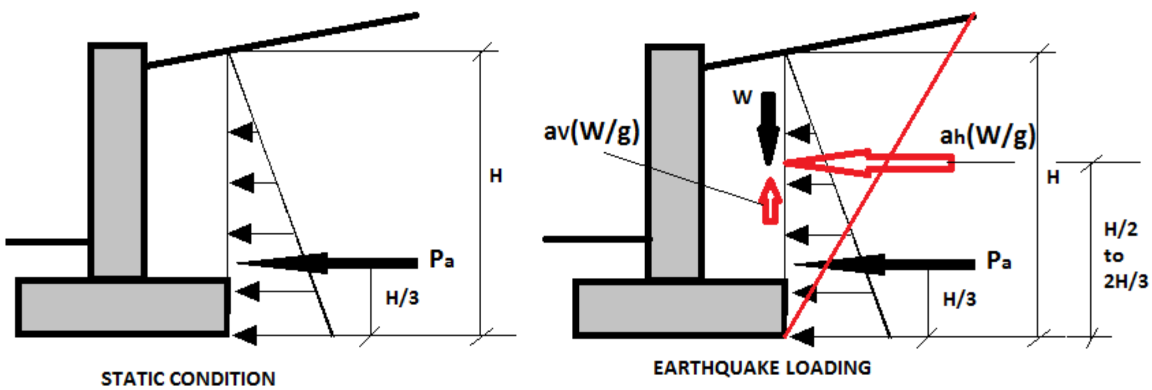


Fig. 13.4 Cantilever Retaining Wall Forces

Factor of safety against overturning or lateral sliding can be determined as the resisting moment/force divided by the driving moment/force. The factor of safety (FS) shall be at least 1.5 (IBC §1807.2.3). However, where earthquake loads are included, the minimum safety factor for retaining wall sliding and overturning shall be 1.1.

The load combinations of IBC §1605 shall not apply to this requirement. Instead, design shall be based on 0.7 times nominal earthquake loads, 1.0 times other nominal loads (i.e., soil pressure *H*), and investigation shall be conducted with one or more of the variable loads set to zero.

CHAPTER 3

Question No.	Brief Explanation	Correct Answer (A B C D)
3.1	Member's rigidity is the same as member's stiffness and is the inverse of deflection	A
3.2	When the mass increases, natural period increases and base shear (mass*acceleration) also increases	C
3.3	$\boldsymbol{k_2}$ and $\boldsymbol{k_3}$ are in series $\boldsymbol{k_{eqv\ 2\text{-}3}}$ = 60*40/(60+40) = 24 k/in $\boldsymbol{k_2}$ and $\boldsymbol{k_{eqv\ 2\text{-}3}}$ are in parallel $\boldsymbol{k_{eqv\ 1\text{-}2\text{-}3}}$ = 80+24 = 104 k/in	C
3.4	Spectral velocity of a SDOF system is the maximum velocity experienced by the structure due to a specific ground motion. That is equals to the spectral displacement times the angular frequency ($\boldsymbol{Sv = \omega.Sd}$)	D
3.5	Maximum acceleration experienced by a building due to a specific ground motion is the spectral acceleration	C
3.6	$k_1 = 12EI_1/H_1^3$; $k_2 = 12EI_2/H_2^3$ $k_2 = 12E(0.5I_1)/(0.5H_1)^3 = 12E(I_1)/(0.5^2 * H_1^3)$ $k_1 / k_2 = (0.5^2) = 0.25 = 1/4$; $k_2 = 4 . k_1$ $T_1 = 2\pi\sqrt{W/k_1.g}$; $T_2 = 2\pi\sqrt{W/k_2.g}$ $T_2 = 2\pi\sqrt{W/k_2.g} = 2\pi\sqrt{W/4.k_1.g}$ $T_2 = 1/2 * (2\pi\sqrt{W/k_1.g}) = 1/2 * T_1$	A
3.7	$\boldsymbol{4I_1 = 2I_2 = I_3}$ Thus, $\boldsymbol{I_3}$ is stiffer than $\boldsymbol{I_1}$ For example if $I_1 = 100$, then, $I_2 = 200$ and $I_3 = 400$ Forces are distributed to the three columns based on their relative stiffness (relative rigidity). Since boundary conditions (i.e., $k = 12EI/H^3$), and column heights are the same, thus distribution is based on ***I***. Ratio = 1+2+4 = 7; so Col 1 = 1/7; Col 2 = 2/7; Col 3 = 4/7	A
3.8	Natural period of a SDOF system are derived from the system mass and stiffness	D
3.9	The damping of an oscillating SDOF is the decay of amplitude (displacement, acceleration, etc.) with elapsing time	D
3.10	Higher modes refer to the larger frequencies. Since frequency is the inverse of period, higher frequency modes has the shortest periods	B
3.11	Critical damping of an oscillating harmonic system is the amount of damping that bring the system to static position in the shortest possible time.	A

CHAPTER 5

Question No.	Brief Explanation	Correct Answer A	B	C	D
5.1	From IBC Fig. 1613.3.1 (1) &(2) or ASCE 7 Fig. 22-1 & 22-2. S_S = 1.25g, and S_1 = 0.5g. The g term is already factored in the equations, thus use only the digit.	●	○	○	○
5.2	Shear wave velocity = 7500 in/sec = 625 ft/sec. From Table 5.1 (ASCE 7 Table 20.3-1), Site Class D.	○	○	○	●
5.3	Table 5.2: Site Class C, $S_S = 0.85$, Use straight line interpolation for intermediate values. Knowing S_s=0.85, *required*, F_a. Reading: $S_{s1} = 0.75 < S_{s2} = 1.0$, and $F_{a1} = 1.1 > F_{a2} = 1.0$ $F_a = F_{a2} + [\ (S_{s2} - S_s)/(\ S_{s2} - S_{s1})]*(\ F_{a1} - F_{a2})$ $F_a = 1.0 + [\ (1.0 - 0.85)/(1.0 - 0.75)]*(1.1 - 1.0) = 1.06$ and $S_1 = 0.4$, $F_v = 1.4$.	○	●	○	○
5.4	Table 5.2: Site Class D, $S_S = 0.90$ and $S_1 = 0.4$ Use straight line interpolation for intermediate values. Reading: $S_{s1} = 0.75 < S_{s2} = 1.0$, and $F_{a1} = 1.2 > F_{a2} = 1.1$ $F_a = F_{a2} + [\ (S_{s2} - S_s)/(\ S_{s2} - S_{s1})]*(\ F_{a1} - F_{a2})$ $F_a = 1.1 + [\ (1.0 - 0.90)/(1.0 - 0.75)]*(1.2 - 1.1) = 1.14$ $F_a = 1.14$, $F_v = 1.6$ $S_{MS} = 0.9*1.14 = 1.026$; $S_{M1} = 1.6*0.4 = 0.64$ $S_{DS} = 2/3*1.026 = 0.69$; $S_{D1} = 2/3*0.64 = 0.43$	○	○	●	○
5.5	Table 5.2: Site Class E, $S_S = 1.12$ and $S_1 = 0.35$ $F_a = 0.9$, $F_v = 2.6$ (by interpolation) $S_{MS} = 0.9*1.12 = 1.01$; $S_{M1} = 2.6*0.35 = 0.91$ $S_{DS} = 2/3*1.01 = 0.67$; $S_{D1} = 2/3*0.91 = 0.61$	○	●	○	○
5.6	Shear wave 2400 ft/sec. From Table 5.1 (ASCE 7 Table 20.3-1), Site Class C Table 5.2: Site Class C, $S_S = 0.95$ and $S_1 = 0.52$ $F_a = 1.02$ (by interpolation), $F_v = 1.3$ $S_{MS} = 0.95*1.02 = 0.97$; $S_{M1} = 1.3*0.52 = 0.68$ $S_{DS} = 2/3*0.97 = 0.65$; $S_{D1} = 2/3*0.68 = 0.46$	●	○	○	○
5.7	$S_{DS} = 0.65$, and $S_{D1} = 0.46$ $T_0 = 0.2 * S_{D1}/S_{DS} = 0.2 * 0.46/0.65 = 0.142$ $T_S = S_{D1}/S_{DS} = 0.46/0.65 = 0.71$	○	○	●	○
5.8	$S_{DS} = 0.9$, and $S_{D1} = 0.4$ $T_0 = 0.2 * S_{D1}/S_{DS} = 0.2 * 0.4/0.9 = 0.09$ $T_S = S_{D1}/S_{DS} = 0.4/0.9 = 0.44$	○	○	○	●
5.9	ASCE 7 Fig 22-12, T_L = 16 sec	○	○	○	●

intermediate) that is designed to resist at least 25% of seismic loading, along with a building frame (concentric steel braces, eccentric steel braces, or shear walls).

5.104 floor live load 50 psf
25% of floor live load, *L*, for storage areas and warehouses. However, where the inclusion of storage loads adds no more than 5% of the effective seismic weight at that level, it need not be included. Also, at public garages and open parking structures, it need not be included.

○ ● ○ ○

5.105 Steel Eccentrically Braced Frames with Special Moment Resisting Frames (Dual System); h_n = 60 ft.
The approximate fundamental period T_a:

$$T_a = 0.03\,(60)^{0.75} = 0.646 \text{ sec.}$$

This is the only Dual system that is calculated using this eq. All other Dual Systems are calculated using $T_a = 0.02\,(h_n)^{0.75}$ eq.

○ ○ ● ○

5.106 Highest (largest) ductility is related to the response modification coefficient, *R*.
<u>The largest the *R*, the highest the ductility</u>.
light framed (wood) wall with wood structural panels *R* = 7;
ordinary RC shear wall *R* = 5;
steel special concentrically braced frame *R* = 6;
dual special steel moment frame with special reinforced masonry shear wall *R* = 5.5.

● ○ ○ ○

5.107 Seismic base shear is calculated as:
$\boldsymbol{V} = S_{DS} * (I_e/R) * W$ *but not larger than* $(S_{D1}/T) * (I_e/R) * W$
Since $\boldsymbol{W}$, $\boldsymbol{S_{DS}}$, $\boldsymbol{S_{D1}}$, $\boldsymbol{T}$, and $\boldsymbol{I_e}$ are assumed to be the same, the only difference is the response modification coefficient, ***R***. <u>The higher the ***R***, the lower the base shear ***V***</u>.
light framed (wood) wall sheathed with wood structural panels (bearing wall) ***R*** = 6.5;
special reinforced masonry shear wall ***R*** = 5.5;
steel ordinary concentrically braced frame ***R*** = 3.25;
dual special steel moment frame with steel eccentrically braced frame ***R*** = 8.

○ ○ ○ ●

5.108 Actual seismic force developed in the structure, $\boldsymbol{V_M}$, is larger than the design base shear, ***V***, determined using the ELFA procedure. This is because of the reserved capacity of the structure. Actual seismic force, $\boldsymbol{V_M}$, is determined using the overstrength factor, Ω_0.

● ○ ○ ○

5.109 Public assembly: RC = III. Four story. Steel eccentrically braced frame.
From Table 5.11 All other structures
$\boldsymbol{\Delta_a} = 0.015\, \boldsymbol{h_{sx}}$

○ ● ○ ○

CHAPTER 8

Question No.	Brief Explanation	Correct Answer A B C D
8.1	ASCE7 §11.1.2: Exempt Structures → Hydraulic Structures	A
8.2	Values of R for non-building structures not similar to buildings - ASCE 7 Table 15.4-2→ ground supported steel tank mechanically anchored R=3.0	B
8.3	Rigid Non-building Structures - ASCE 7 §15.4.2. If $T < 0.06$ Sec. → Rigid	B
8.4	Non-building structures not similar to buildings can be: signs, billboards, amusement structures, silos & chimneys, tanks & vessels, cooling towers, bins & hoppers, monuments, trussed towers, cantilever columns, and earth retaining walls. Steel storage racks are non-building structures similar to buildings.	A
8.5	Non-building structures similar to buildings can be: pipe racks, steel storage racks, electrical power generation facilities, structural towers for tanks and vessels, and piers and wharves. Earth retaining structures are non-building structures not similar to buildings.	C
8.6	Values of R for non-building structures not similar to buildings- ASCE 7 Table 15.4-2→ billboards and signs $R = 3.0$	C
8.7	Values of R and C_d for non-building structures not similar to buildings - ASCE 7 Table 15.4-2→ telecommunication tower, steel truss: $R = 3.0$ and $C_d = 3.0$.	B
8.8	Steel storage rack: non building structure similar to building RC = III, Importance Factor, $I_e = 1.25$.	C
8.9	Values of R and Ω_0 for non-building structures similar to buildings - ASCE 7 Table 15.4-1→ steel storage racks: $R = 4.0$ and $\Omega_0 = 2.0$.	B
8.10	Importance Factor $I_e = 1.25$, $R = 4.0$ $C_S = S_{DS} * (I_e / R) = 0.9 * (1.25 / 4.0) = 0.28$ The *maximum* seismic design coefficient for $T \leq T_L$: $T = 0.2 \leq T_L =$ (8 sec to 16 sec., in California) $C_{S,\,max} = (S_{D1} / T) * (I_e / R) = (0.35 / 0.2) * (1.25 / 4.0) = 0.55$ The *minimum* seismic design coefficient: $C_{S,\,min} = 0.044 * S_{DS} * I_e \geq 0.01$ $C_{S,\,min} = 0.044 * 0.9 * 1.25 = 0.049 \geq 0.01$ Since $S_1 = 0.60 \geq 0.6$ $C_{S,\,min} = 0.5 * S_1 * (I_e/R) = 0.5 * 0.6 * (1.25 / 4.0) = 0.094$ Thus: $0.094 \leq C_S = 0.28 \leq 0.55$	B