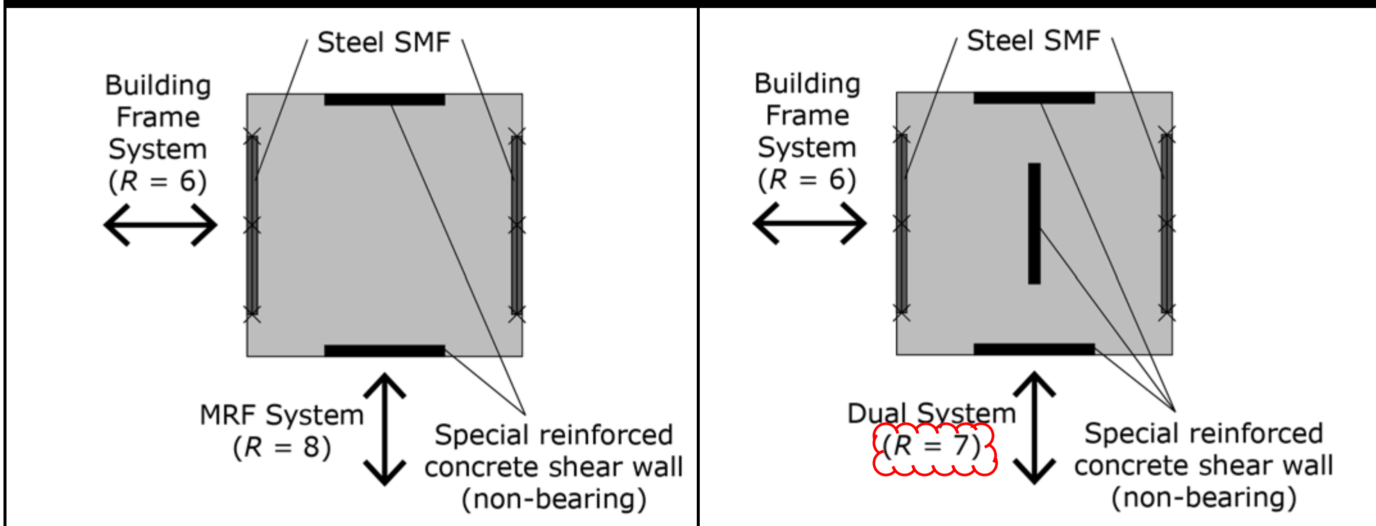
Table 3.2 - Design Spectral Response Acceleration Parameter at Short Periods (S_{DS})						
S_S	Site Class					
	A	B	C	D*	E	F
0.05	0.03	0.03	0.04	0.05	0.08	*Site-Specific Ground Motion Procedure Required - ASCE 7 - Chapter 21*
0.10	0.05	0.07	0.08	0.11	0.17	
0.15	0.08	0.10	0.12	0.16	0.25	
0.20	0.11	0.13	0.16	0.21	0.33	
0.25	**0.13**	**0.17**	**0.20**	**0.27**	**0.42**	
0.30	0.16	0.20	0.24	0.31	0.47	
0.35	0.19	0.23	0.28	0.35	0.51	
0.40	0.21	0.27	0.32	0.39	0.54	
0.45	0.24	0.30	0.36	0.43	0.56	
0.50	**0.27**	**0.33**	**0.40**	**0.47**	**0.57**	
0.55	0.29	0.37	0.43	0.50	0.59	
0.60	0.32	0.40	0.46	0.53	0.60	
0.65	0.35	0.43	0.49	0.55	0.61	
0.70	0.37	0.47	0.52	0.58	0.61	
0.75	**0.40**	**0.50**	**0.55**	**0.60**	**0.60**	
0.80	0.43	0.53	0.58	0.63	0.61	
0.85	0.45	0.57	0.60	0.66	0.61	
0.90	0.48	0.60	0.62	0.68	0.61	
0.95	0.51	0.63	0.65	0.71	0.61	
1.00	**0.53**	**0.67**	**0.67**	**0.73**	**0.60**	
1.05	0.56	0.70	0.70	0.76	0.63	
1.10	0.59	0.73	0.73	0.78	0.66	
1.15	0.61	0.77	0.77	0.80	0.69	
1.20	0.64	0.80	0.80	0.82	0.72	
1.25	**0.67**	**0.83**	**0.83**	**0.83**	**0.75**	
1.30	0.69	0.87	0.87	0.87	0.78	
1.35	0.72	0.90	0.90	0.90	0.81	
1.40	0.75	0.93	0.93	0.93	0.84	
1.45	0.77	0.97	0.97	0.97	0.87	
1.50	**0.80**	**1.00**	**1.00**	**1.00**	**0.90**	
1.55	0.83	1.03	1.03	1.03	0.93	
1.60	0.85	1.07	1.07	1.07	0.96	
1.65	0.88	1.10	1.10	1.10	0.99	
1.70	0.91	1.13	1.13	1.13	1.02	
1.75	**0.93**	**1.17**	**1.17**	**1.17**	**1.05**	
1.80	0.96	1.20	1.20	1.20	1.08	
1.85	0.99	1.23	1.23	1.23	1.11	
1.90	1.01	1.27	1.27	1.27	1.14	
2.00	**1.07**	**1.33**	**1.33**	**1.33**	**1.20**	
2.10	1.12	1.40	1.40	1.40	1.26	
2.20	1.17	1.47	1.47	1.47	1.32	
2.30	1.23	1.53	1.53	1.53	1.38	
2.40	1.28	1.60	1.60	1.60	1.44	
2.50	**1.33**	**1.67**	**1.67**	**1.67**	**1.50**	
2.60	1.39	1.73	1.73	1.73	1.56	
2.70	1.44	1.80	1.80	1.80	1.62	
2.80	1.49	1.87	1.87	1.87	1.68	
2.90	1.55	1.93	1.93	1.93	1.74	
3.00	**1.60**	**2.00**	**2.00**	**2.00**	**1.80**	

- ***Moment-resisting frames*** (e.g., SMF, STMF, IMF, OMF) provide resistance to lateral loads primarily by flexural (bending) action of members (e.g., beams, columns).

D. Dual System - Structural system that is essentially a combination of a Building Frame System (e.g., shear walls, CBF's, EBF's) and a Moment-Resisting Frame System (e.g., SMF's or IMF's) oriented to resist lateral loads in the same direction.

> Per *ASCE 7 – §12.2.5.1*, the total seismic force resistance is to be provided by the combination of the *moment-resisting frames* and the *shear walls* (or *braced frames*) in proportion to their rigidities.
>
> Furthermore, the *moment-resisting frames* shall be designed to independently resist at least 25 percent of the design seismic forces.

Figure 4.2 – Dual System Example



Building A - Not a Dual System | **Building B –** Dual System (N-S axis)

E. Shear Wall-Frame Interactive System - a structural system that uses combinations of ordinary reinforced concrete *shear walls* and ordinary reinforced concrete moment frames (OMF's). Per *ASCE 7 – Table 12.2-1*.

This type of system is not permitted (i.e., NP) in Seismic Design Categories C, D, E or F.

F. Cantilevered Column System - Structural system relying on cantilever column elements for lateral resistance – see Figure 4.3.

G. Steel Systems Not Specifically Detailed for Seismic Resistance - excluding cantilever column systems - per *ASCE 7 – Table 12.2-1*.

This type of system is not permitted (i.e., NP) in Seismic Design Categories D, E or F.

> **NOTE:** The structural framing system shall also comply with the system specific requirements found in *ASCE 7 – §12.2.5* (i.e., *ASCE 7 – §12.2.5.1* through *ASCE 7 – §12.2.5.10*).

Horizontal Seismic Load Effect with Overstrength Factor, E_{mh} *ASCE 7 – §12.4.3.1*

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

➢ $E_{mh} = \pm\Omega_0 \cdot Q_E$ *ASCE 7 (12.4-7)*

where:

Q_E = effects of horizontal seismic forces from the seismic base shear V (per *ASCE 7 – §12.8.1*) or the seismic lateral force F_p (per *ASCE 7 – §13.3.1*). See *ASCE 7 – §12.5.3* & *ASCE 7 – §12.5.4* for consideration of orthogonal effects)

Ω_0 = Overstrength Factor … per *ASCE 7 – Table 12.2-1* ~~*or 12.14-1*~~

Exception: E_{mh} need not exceed the maximum force that can develop in the element as determined by … see *ASCE 7 – §12.4.3.1*

5.2 Load Combinations *IBC §1605*

General *IBC §1605.1*

Buildings (and other structures) and portions thereof shall be designed to resist the load combinations specified in:

- *IBC §1605.2* (Strength Design or Load & Resistance Factor Design – SD/LRFD) **or**
- *IBC §1605.3* (Allowable Stress Design – ASD), **and**
- *IBC Chapters 18* through *23*, **and**
- The load combinations with overstrength factor (Ω_0) specified in *ASCE 7 – §12.3.4.2* where require by *ASCE 7 – §12.2.5.2*, *§12.3.3.3* and/or *ASCE 7 – §12.10.2.1*

NOTE: When using the Simplified Procedure of *ASCE 7 – §12.14*, the load combinations with overstrength factor of *ASCE 7 – §12.14.3.2* shall be used (i.e., $\Omega_0 = 2.5$ assumed).

Load combinations are a way of considering the maximum (or minimum) forces on a structural element using principles of superposition.

The load combinations consider combined effects of gravity loads (e.g., dead load, floor live load, roof live load, rain load, snow load) and other load effects as a result of earthquake, wind, flood, earth pressure, fluid pressure, etc.

Notations –

D = dead load

E = seismic (i.e., earthquake) load effect

E_m = maximum seismic load effect of horizontal and vertical seismic forces per *ASCE 7 – §12.4.3*

F = load due to fluids with well-defined pressures and maximum heights

F_a = flood load

H = load due to earth pressure, ground water pressure or pressure of bulk materials

L = live load (except roof live load) … including any permitted live load reduction

L_r = roof live load … including any permitted live load reduction

R = rain load

9. **Controlled low-strength material (CLSM)** – *IBC §1803.5.9*
10. **Alternate setback and clearance*** – *IBC §1803.5.10*
11. **Seismic Design Category C, D, E & F*** – *IBC §1803.5.11*

    An investigation shall be conducted and shall include an evaluation of the following potential hazards resulting from earthquake motions:

    - Slope instability
    - Liquefaction
    - Differential settlement ~~slope instability~~
    - Surface displacement due to faulting or lateral spreading

*__Exception__: The *building official* shall be permitted to waive the requirement for a geotechnical investigation where satisfactory data from adjacent areas is available that demonstrates an investigation is not necessary for any of the conditions in *IBC §1803.5.1* through *§1803.5.6*, *§1803.5.10*, and *§1803.5.11*.

12. **Seismic Design Category D, E & F** – *IBC §1803.5.12*

    The geotechnical investigation required by *IBC §1803.5.11* shall also include:

    ✓ Determination of lateral pressures on foundation walls and <u>retaining walls</u> due to earthquake motions.

    ✓ Potential for liquefaction and soil strength loss evaluated for site peak ground accelerations, magnitudes and source characteristics consistent with the design earthquake ground motions …

    ✓ An assessment of potential consequences of liquefaction and soil strength loss, including estimation of differential settlement, lateral movement, lateral loads on foundations, reduction in foundation soil-bearing capacity, increases in lateral pressures on retaining walls and flotation of buried structures.

    ✓ Discussion of mitigation measures such as, but not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements and forces, or any combination of these measures and how they shall be considered in the design of the structure.

Reporting — *IBC §1803.6*

Where geotechnical investigations are required, a written report of the investigations shall be submitted to the *building official* by the owner or authorized agent at the time of *permit* application. See *IBC §1803.6* for required information in the report.

Foundations — *IBC §1808*

General — *IBC – §1808.1*

Foundations shall be designed and constructed in accordance with *IBC §1808.2* through *§1808.9*. Shallow foundations shall also satisfy the requirements of *IBC §1809*. Deep foundations shall also satisfy the requirements *IBC §1810*.

Design Loads — *IBC – §1808.3*

Foundations shall be designed for the most unfavorable effects due to the combinations of loads specified in *IBC §1605.2* (i.e., SD/LRFD) or *§1605.3* (i.e., ASD). The dead load is permitted to include the weight

3. Maximum Chord Force on lines A & B, *CF*

max. $M = w_s \cdot L^2 / 8 = (490 \text{ plf})(40')^2 / 8 = 98{,}000$ lb-ft (SD/LRFD force level)

max. $CF = 0.7 \cdot M / d = (0.7)(98{,}000 \text{ lb-ft}) / (100') = \boxed{690 \text{ lbs}}$ (ASD force level)

4. Unit Wall Shear & Nailing on lines 1 & 2, v_w

- Wall Line 1: total shear wall length, $\Sigma b = 7' + 13' = 20'$

Maximum $h/b = 12'/7' = 1.71{:}1 < 2{:}1$ max. per *SDPWS Table 4.3.4* (see Table 9.2, p. 1-122) →
∴ A reduction of unit wall shears in *IBC Table 2306.3* **is not** necessary

wall $v_1 = \rho\,(0.7 \cdot V_1) / \Sigma b = (1.00)(0.7)(9{,}800 \text{ lbs}) / (20') = \boxed{343 \text{ plf}}$ (ASD force level)

wall $v_1 = 343$ plf → *IBC Table 2306.3* →

Wall Line 1 - use 15/32″ WSP Structural I sheathing w/ 8d common @ 4″ o.c. edge nailing & 12″ o.c. field nailing for both walls … allowable unit wall shear = 430 plf > 343 plf → **OK** (3x studs & blocking required at abutting panel edges & staggered nailing at all panel edges per *IBC 2306.3, footnotes i*)

- Wall Line 2: total shear wall length, $\Sigma b = 10' + 5' = 15'$

Minimum $h/b = 12'/10' = 1.2{:}1 < 2{:}1$ maximum per *SDPWS Table 4.3.4* (see Table 9.2, p. 1-122) →
∴ A reduction of unit wall shears in *IBC Table 2306.3* **is not** necessary for 10′ shear wall

Maximum $h/b = 12'/5' = 2.4{:}1 > 2{:}1$ maximum per *SDPWS Table 4.3.4* (see Table 9.2, p. 1-122) →
∴ A reduction of unit wall shears in *IBC Table 2306.3* **is** necessary for 5′ shear wall …
where the reduction factor = $2b/h = 2(5')/(12') = \underline{0.83}$

wall $v_2 = \rho\,(0.7 \cdot V_2) / \Sigma b = (1.00)(0.7)(9{,}800 \text{ lbs}) / (15') = \boxed{457 \text{ plf}}$ (ASD force level)

wall $v_2 = 457$ plf → *IBC Table 2306.3* →

15/32″ WSP Structural I sheathing w/ 8d common @ 3″ o.c. edge nailing, 12″ o.c. field nailing.

10′ shear wall - allowable unit wall shear = 550 plf > 457 plf → **OK** (w/ no reduction)

Wall Line 2 – 10′ shear wall - use 15/32″ WSP Structural I sheathing w/ 8d common @ 3″ o.c. edge nailing, 12″ o.c. field nailing … allowable unit wall shear = 550 plf > 457 plf → **OK** (3x studs & blocking required at abutting panel edges & staggered nailing at all panel edges per *IBC 2306.3, footnotes i*)

15/32″ WSP sheathing w/ 8d common @ 3″ o.c. edge nailing, 12″ o.c. field nailing.

5′ shear wall - allowable unit wall shear = (0.83)(550 plf) = 456 plf ≈ 457 plf → **OK**

Wall Line 2 – 5′ shear wall - use 15/32″ WSP Structural I sheathing w/ 8d common @ 3″ o.c. edge nailing, 12″ o.c. field nailing … allowable unit wall shear = 0.83(550 plf) = 456 plf ≈ 457 plf → **OK** (3x studs & blocking required at abutting panel edges & staggered nailing at all panel edges per *IBC 2306.3, footnotes i*)

NOTE: – the allowable unit wall shear reduction factor $2b/h$ per *SDPWS Table 4.3.4, footnote 1* can easily result in separate wood structural panel shear walls on the same wall line with different required edge nail spacing … as nearly occurred in this example on Wall Line 2.

$$\text{Maximum } F_B = \left(V_Y \cdot \frac{R_B}{\sum R_Y}\right) + \left(\frac{M_{T1} \cdot R_B \cdot d_B}{\sum R \cdot d^2}\right)$$

$= [73.4 \text{ kips } (2.54) / (7.54)] + [888.1 \text{ kip-ft } (2.54) (53.1 \text{ ft}) / (14{,}615 \text{ ft}^2)]$

$= 24.7 \text{ kips} + 8.2 \text{ kips} = \boxed{32.9 \text{ kips}}$

D.) E-W DIRECTION – Design Force to Shear Walls C, D & E

$V = 73.4$ kips

Accidental eccentricity, $e_y = \pm\, 0.05 \cdot L_\perp = \pm\, 0.05\ (40') = \pm\, 2.0'$
Accidental torsional moment, $M_{ta} = V \cdot (\pm\, 0.05 \cdot L_\perp) = 73.4$ kips $(\pm\, 2.0') =$ $\underline{\pm 146.8 \text{ kip-ft}}$

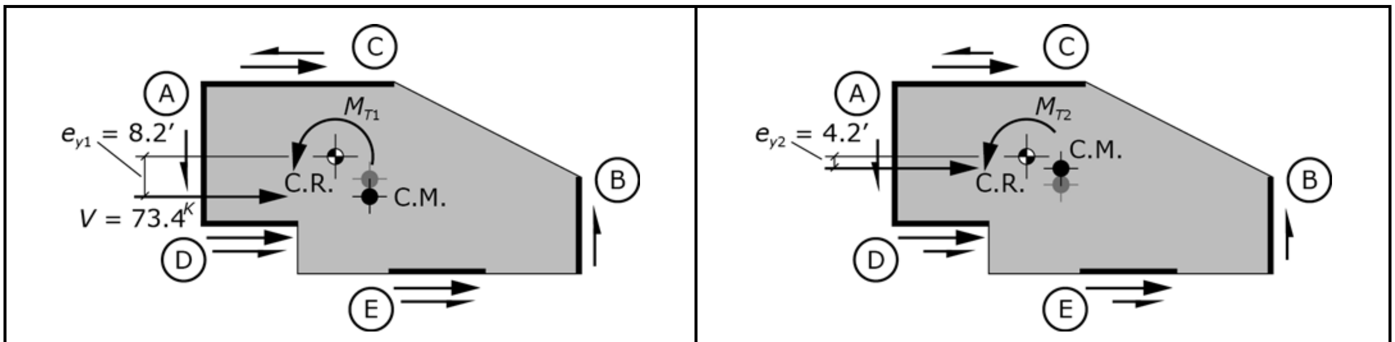
Calculated $e_y = 6.2'$ (from plan)
Inherent torsional moment, $M_t = V \cdot e_y = 73.4$ kips $(6.2') =$ $\underline{+455.1 \text{ kip-ft}}$

Design $e_y = e_y \pm 0.05 \cdot L_\perp = 6.2' \pm 2.0'$

$e_{y1} = 6.2' + 2.0' =$ $\underline{+8.2 \text{ feet}}$
$e_{y2} = 6.2' - 2.0' =$ $\underline{+4.2 \text{ feet}}$

$M_{T1} = V \cdot (e_y + 0.05 \cdot L_\perp)$
$= M_t + M_{ta} = 455.1 + 146.8 = \underline{+601.9 \text{ kip-ft}}$

$M_{T2} = V \cdot (e_y - 0.05 \cdot L_\perp)$
$= M_t - M_{ta} = 455.1 - 146.8 = \underline{+308.3 \text{ kip-ft}}$



e_{y1} = calc. $e_y + 0.05 \cdot L_y$	e_{y2} = calc. $e_y - 0.05 \cdot L_y$

NOTE: By observation, e_{y1} will govern the design of shear walls D & E (i.e., maximum additive torsional shear) and e_{y2} will govern the design of shear wall C (i.e., minimum subtractive torsional shear).

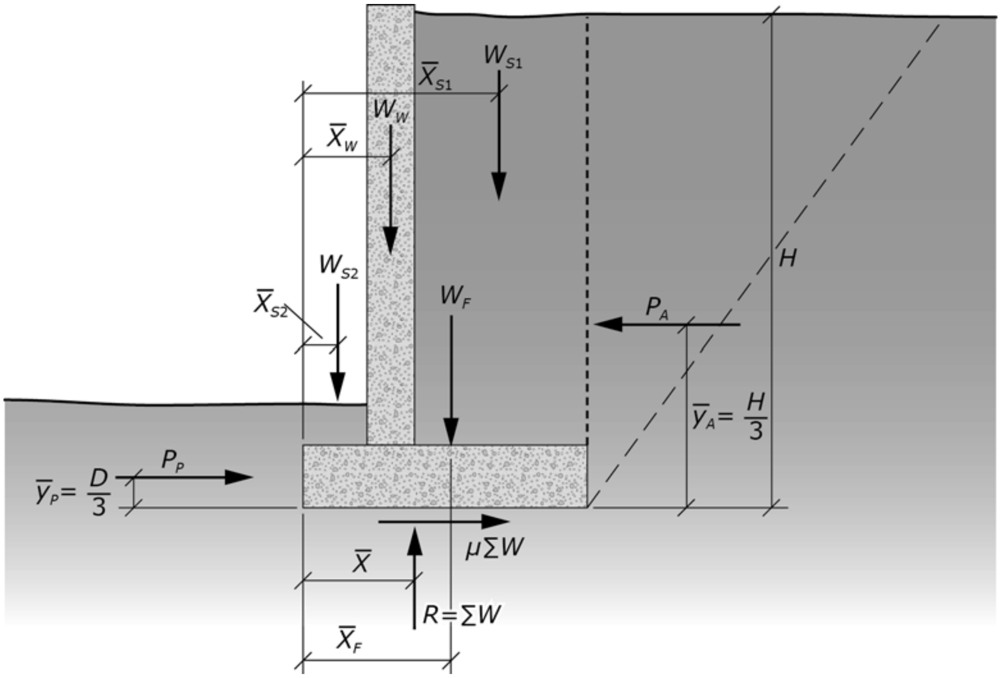
Neither eccentricity will govern the design of shear walls A & B since the force direction is not parallel to these walls (i.e., no direct shear).

$\sum R_X = R_C + R_D + R_E = 7.49 + 2.54 + 2.54 = \underline{12.57}$

$\sum R \cdot d^2 = \underline{14{,}615 \text{ ft}^2}$ … from Part C

Solution:

A.) STATIC CONDITION, K_A:



Static Active Soil Pressure

Total static active force, P_A	Total static passive (resisting) force, P_P
$P_A = \tfrac{1}{2}\, K_A \cdot \gamma \cdot H^2$ $= \tfrac{1}{2}(0.318)(110\text{ pcf})(11.25')^2 = \underline{2{,}214\text{ lbs/ft}}$	$P_P = \tfrac{1}{2}\, K_P \cdot \gamma \cdot D^2$ $= \tfrac{1}{2}(3.18)(110\text{ pcf})(2.25')^2 = \underline{885\text{ lbs/ft}}$
resultant height, $\bar{y}_A = H/3 = (11.25')/3 = 3.75'$	resultant height, $\bar{y}_P = D/3 = (2.25')/3 = 0.75'$

Weights

Soil over heel, $W_{S1} = (10')(4')(110\text{ pcf}) = 4{,}400$ lbs/ft
Soil over toe, $W_{S2} = (1')(1.5')(110\text{ pcf}) = 165$ lbs/ft
Concrete stem wall, $W_W = (10')(1')(150\text{ pcf}) = 1{,}500$ lbs/ft
Concrete footing, $W_F = (1.25')(6.5')(150\text{ pcf}) = 1{,}219$ lbs/ft

Resultant weight, $R = \Sigma W = 4{,}400 + 165 + 1{,}500 + 1{,}219 = \underline{7{,}284\text{ lbs/ft}}$

1. Sliding Factor of Safety

Sliding force, $F_S = P_A = \underline{2{,}214\text{ lbs/ft}}$

Resisting force, F_R = passive force + friction force
$= P_P + \mu \cdot \Sigma W$
$= 885\text{ lbs/ft} + 0.4\,(7{,}284\text{ lbs/ft}) = \underline{3{,}799\text{ lbs/ft}}$

Sliding factor of safety, $FS = \dfrac{F_R}{F_S} = \dfrac{3{,}799}{2{,}214} = \boxed{1.72} > 1.5$ minimum per *IBC §1807.2.3* **OK**

2. Overturning Factor of Safety

Overturning moment, $OTM = P_A \cdot \bar{y}_A$

$= (2{,}214 \text{ lbs/ft})(11.25'/3) = \underline{8{,}302 \text{ lb·ft/ft}}$

Resisting moment,

$RM = P_P \cdot \bar{y}_P + W_{S1} \cdot \bar{x}_{S1} + W_{S2} \cdot \bar{x}_{S2} + W_W \cdot \bar{x}_W + W_F \cdot \bar{x}_F$

$= 885 \text{ lbs/ft } (2.25')/3 + 4{,}400\ (4.5') + 165\ (0.75') + 1{,}500\ (2') + 1{,}219\ (3.25')$

$= \underline{27{,}550 \text{ lb·ft/ft}}$

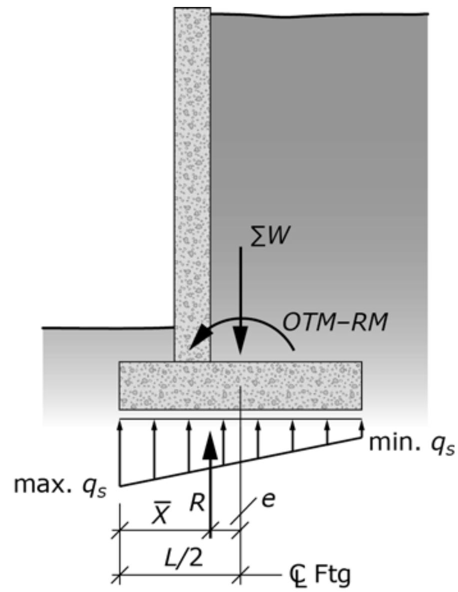
Overturning factor of safety, $FS = \dfrac{RM}{OTM} = \dfrac{27{,}550}{8{,}302} = \boxed{3.32} > 1.5$ minimum per *IBC §1807.2.3* **OK**

3. Maximum Soil Bearing Pressure

$$\bar{x} = \frac{RM - OTM}{R} = \frac{(27{,}550 - 8{,}302)}{7{,}284} = 2.64'$$

eccentricity from centerline of footing,

$e = L/2 - \bar{x} = (6.5')/2 - 2.64' = \underline{0.61'}$



Soil Bearing Pressure

if $e < L/6$ (i.e., R is within middle 1/3 of footing) $\rightarrow$ the soil pressure distribution is <u>trapezoidal</u>

if $e \geq L/6$ (i.e., R is outside of middle 1/3 of footing) $\rightarrow$ the soil pressure distribution is <u>triangular</u>

$e = 0.61' < L/6 = 6.5'/6 = 1.08'$ $\rightarrow$ $\therefore$ <u>soil pressure distribution is trapezoidal</u>

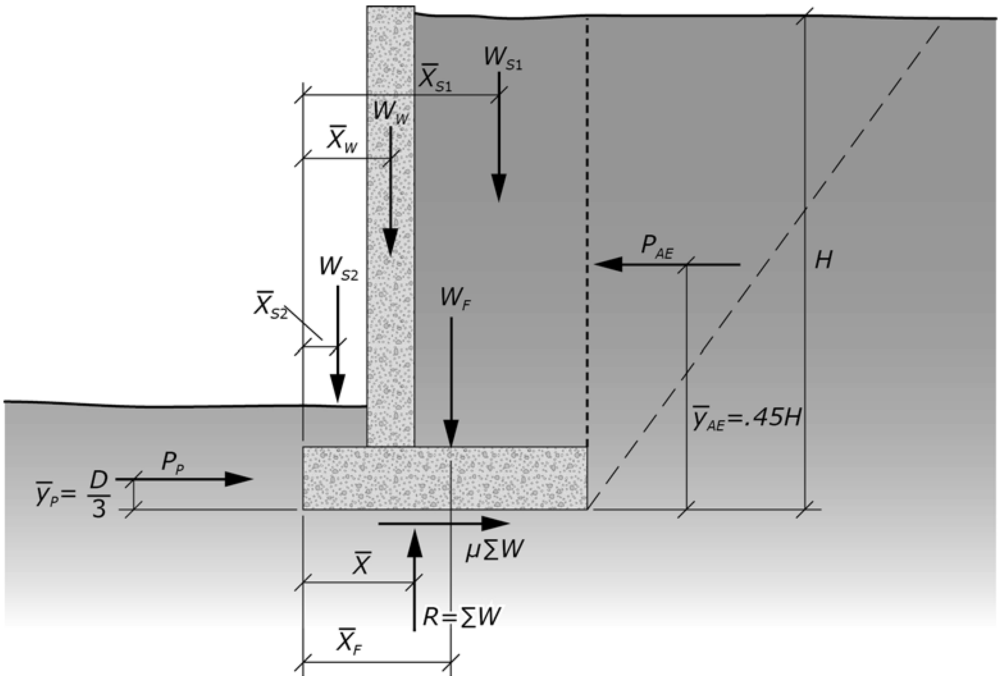
for a trapezoidal soil pressure distribution,

maximum soil pressure, $q_s = \dfrac{R}{L}\left(1 + \dfrac{6 \cdot e}{L}\right)$

minimum soil pressure, $q_s = \dfrac{R}{L}\left(1 - \dfrac{6 \cdot e}{L}\right)$

therefore, max. $q_s = \dfrac{7{,}284}{6.5'}\left(1 + \dfrac{6 \cdot (0.61')}{6.5'}\right) = \boxed{1{,}750 \text{ psf/ft}} < 3{,}000$ psf allowable $(D + L)$ **OK**

B.) STATIC PLUS SEISMIC CONDITION, K_{AE}:



Static plus Seismic Active Soil Pressure

Total static plus seismic active force, P_{AE}	Total static passive (resisting) force, P_P (from part A)
$P_{AE} = \frac{1}{2}\, K_{AE} \cdot \gamma \cdot H^2$ $= \frac{1}{2}\,(0.538)(110 \text{ pcf})(11.25')^2 = \underline{3{,}745 \text{ lbs/ft}}$ resultant height, $\overline{y}_{AE} = 0.45 \cdot H$ $= 0.45\,(11.25') = 5.06'$	$P_P = \underline{885 \text{ lbs/ft}}$ resultant height, $\overline{y}_P = D/3 = 0.75'$

Weights (from part A)

Resultant weight, $R = \Sigma W = \underline{7{,}284 \text{ lbs/ft}}$

1. Sliding Factor of Safety

Sliding force, $F_S = P_{AE} = \underline{3{,}745 \text{ lbs/ft}}$

Resisting force, F_R = passive force + friction force

$= P_P + \mu \cdot \Sigma W$

$= 885 \text{ lbs/ft} + 0.4\,(7{,}284 \text{ lbs/ft}) = \underline{3{,}799 \text{ lbs/ft}}$

Sliding factor of safety, $FS = \dfrac{F_R}{F_S} = \dfrac{3{,}799}{3{,}745} = \boxed{1.01} < 1.5 / 1.33 = 1.1$ **NG!**

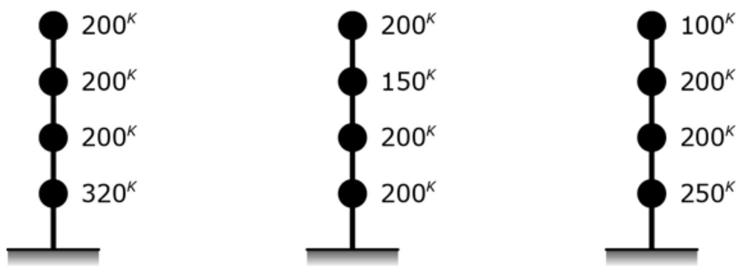
NOTE: *2009 IBC §1605.3.2 & §1806.1* (and most Geotechnical reports) allow a one-third increase in allowable stress for all load combinations that include short-term loads such as earthquake (or wind). Although not specifically addressed in the *IBC*, many designers allow a reduced factor for safety (for sliding and overturning) when considering these short-term loads … i.e., short term $FS = 1.5 / 1.33 = 1.1$

4.68 A structural analysis has been performed on a two-story apartment building (with parking garage in the first-story). The lateral story strength of the first and second stories were determined to be 57 kips and 76 kips respectively. The story stiffness of the first and second stories was determined to be 14 kips/inch and 19.5 kips/inch respectively. Which of the following vertical irregularities are present in this structure?

I. Stiffness – Soft Story
II. Discontinuity in Lateral Strength – Weak Story

a. I
b. II
c. I & II
d. None of the above

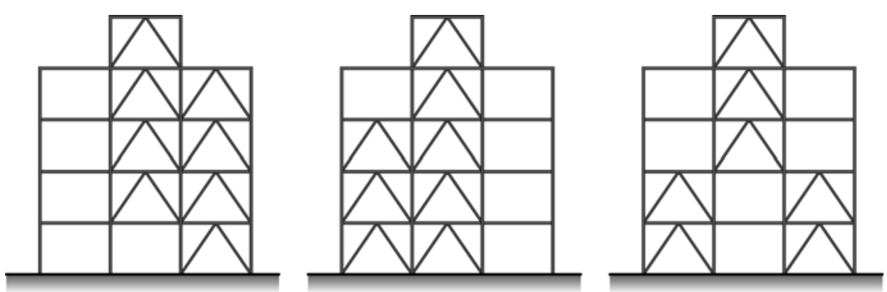
4.69 Which of the following structures are considered to have a Weight (Mass) Irregularity?



I. II. III.

a. I
b. I & II
c. II & III
d. I, II & III

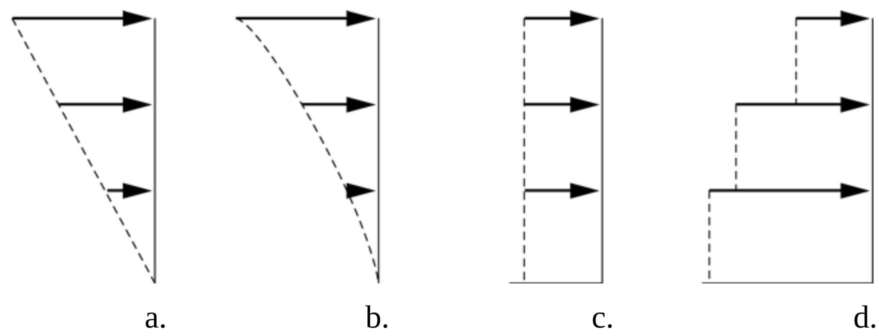
4.70 Which of the following braced frame structures is likely to have a Stiffness-Soft Story Irregularity?



I. II. III.

a. I
b. I & II
c. I & III
d. I, II & III

Using the figures below, answer questions 4.79 through 4.82



4.79 Which figure best represents the *Equivalent Lateral Force* (ELF) procedure vertical distribution of seismic forces (F_x) for structures with a period (T) less than or equal to 0.5 seconds?

4.80 Which figure best represents the *Equivalent Lateral Force* (ELF) procedure story shear distribution (V_x) for structures with a period (T) less than or equal to 0.5 seconds?

4.81 Which figure best represents the *Simplified Design* procedure vertical distribution of seismic forces (F_x)?

4.82 Which figure best represents the *Equivalent Lateral Force* (ELF) procedure vertical distribution of seismic forces (F_x) for structures with a period (T) greater than or equal to 2.5 seconds?

4.83 Given a 15-story Office Bldg w/ steel moment frames, which site class is likely to result in the largest seismic forces?

a. Site Class B (rock)
b. Site Class C (dense soil)
c. Site Class D (stiff soil)
d. Site Class E (soft soil)

4.84 Given two structures with the same R, I, S_{DS} = 0.73 & S_{D1} = 0.30. Structure A has a period (T_A) of 0.35 second. Structure B has an effective seismic weight of 3 times that of Structure A (i.e., $W_B = 3 \cdot W_A$). What would be the period of Structure B such that the Base Shear (V) of the two structures would be equal?

a. 0.35 second
b. 0.65 second
c. 1.25 seconds
d. 2.15 seconds

Problem	Answer	Reference / Solution
9.6	b	p. 1-122, Table 9.2 & *SDPWS Table 4.3.4* $h/b \le 3.5$ maximum (requires reduction factor $2b/h$ for seismic) $\therefore$ minimum $b = (h/3.5) = 10' / 3.5 = 2.86' \approx$ <u>2′-10″</u> ←
9.7	a	p. 1-121, Wood structural panel diaphragms $V_{max} = w \cdot L / 2 = V / 2 = 33.6$ kips / 2 = 16.8 kips for ASD, roof $v = (0.7 \cdot V_{max}) / d = 0.7(16.8 \text{ kips}) / 50' = 235$ plf ≈ <u>240 plf</u> ←
9.8	b	p. 1-104, Drag force Maximum drag force occurs on right (i.e., East) wall line at 20′ from South end of collector (i.e., South end of 30′ shear wall) – for ASD, max F_d = roof $v(20') = (240 \text{ plf})(20') = 4{,}800$ lbs = <u>4.8 kips</u> ←
9.9	b	p. 1-130, Shear wall overturning / Hold-downs $\rho = 1.0$ (given) $V_1 = V_2 = V / 2 = 33.6$ kips / 2 = 16.8 kips $T = \dfrac{-0.7\rho(V_1 \cdot h)}{b} = \dfrac{-0.7(1.0)(16.8)(10')}{30'} = -3.92$ kips $\therefore$ for ASD, uplift T = <u>4.0 kips</u> ←
9.10	b	p. 1-121, Wood structural panel diaphragms $V = C_S \cdot W = 0.196 \cdot W$ (given) For a single-story building – $w_s = f_{p1} = F_{p1} / L = C_S \cdot w_{p1}$ East-West: $w_s = 0.196\,[(25 \text{ psf})(75') + (15 \text{ psf})(12'/2) \cdot 4 \text{ walls}] = 438$ plf $V_{max} = w_s \cdot L / 2 = (438 \text{ plf})(40') / 2 = 8{,}760$ lbs for ASD, roof $v = (0.7 \cdot V_{max}) / d = 0.7 \cdot (8{,}760 \text{ lbs}) / 75' = 82$ plf ≈ <u>80 plf</u> ←
9.11	c	*2009 IBC* p. 474, *Table 2306.3* 3/8″ rated sheathing w/ 8d common (2½″ x 0.131″) @ 2″ o.c. = 530 plf 15/32″ Structural I w/ 10d common @ 6″ o.c. = 340 plf < 530 plf **<u>NG!</u>** 15/32″ Structural I w/ 10d common @ 4″ o.c. = 510 plf < 530 plf **<u>NG!</u>** 15/32″ Structural I w/ 10d common @ 3″ o.c. = 665 plf > 530 plf **<u>OK</u>** 15/32″ Structural I w/ 10d common @ 2″ o.c. = 870 plf >> 530 plf **<u>OK</u>** ∴ use 15/32″ Structural I w/ <u>10d common @ 3″ o.c.</u> = 665 plf > 530 plf ←
9.12	d	*2009 IBC* p. 470, *Table 2306.2.1(1)* Load <u>parallel</u> to continuous panel joints = <u>Case 3</u> (weak direction) 15/32″ sheathing w/ 8d @ 6″ o.c. unblocked = 180 plf < 275 plf **<u>NG!</u>** 15/32″ sheathing w/ 10d @ 6″ o.c. unblocked = 190 plf < 275 plf **<u>NG!</u>** 15/32″ sheathing w/ 8d @ 6″ o.c. blocked = 270 plf < 275 plf **<u>NG!</u>** 15/32″ sheathing w/ 10d @ 6″ o.c. blocked = 290 plf > 275 plf **<u>OK</u>** ∴ use 15/32″ sheathing w/ <u>10d common @ 6″ o.c.</u> = 290 plf > 275 plf ←
9.13	d	p. 1-100, Flexible diaphragm analysis $V = C_S \cdot W$ *(continued)*