Solution:
Office building = Risk Category II → 2015 IBC Table 1604.5

\[ S_{DS} = 0.95 \rightarrow \text{given} \]

\[ I_e = 1.0 \rightarrow \text{ASCE 7 – Table 1.5-2; Risk Category II} \]

\[ R = 5\frac{1}{2} \rightarrow \text{ASCE 7 – Table 12.2-1, item B.16: Building Frame System – special reinforced masonry shear walls} \]

**NOTE:** by observation, for a single story building … \( T < T_S \) → \( \therefore \) ASCE 7 (12.8-2) will govern \( C_S \)

**Seismic Response Coefficient, \( C_S \)**

\[ C_S = \frac{S_{DS}}{R/I_e} = \frac{(0.95)}{5.5/1.0} = 0.173 \]  \hspace{1cm} (ASCE 7 (12.8-2))

**Seismic Base Shear, \( V \)**

\[ V = C_S W \]  \hspace{1cm} (ASCE 7 (12.8-1))

\[ \Rightarrow V = 0.173 W \text{ ← use for shear wall design} \]

**Diaphragm Design Force at Roof, \( F_{px} \)**

\[
F_{px} = \sum_{i=1}^{n} \frac{F_i}{w_i} \sum_{i=1}^{n} w_i
\]

(ASCE 7 (12.10-1))

For a single-story building (i.e., \( F_1 = V \)):

\[ F_{p1} = C_S W \frac{w_{p1}}{w} = C_S w_{p1} = 0.173 \cdot w_{p1} \]

\[ F_{p1} \leq 0.4 S_{DS} I_e w_{p1} = 0.4 \cdot (0.95)(1.0) w_{p1} = 0.380 w_{p1} \text{ maximum} \]

\[ F_{p1} \geq 0.2 S_{DS} I_e w_{p1} = 0.2 \cdot (0.95)(1.0) w_{p1} = 0.190 w_{p1} \text{ minimum} \text{ ← use for roof diaphragm design} \]

A.) **N-S DIRECTION:** \( L = 70', d = 40' \)

1. **Design Seismic Force to Diaphragm, \( w_s = f_{p1} = F_{p1}/L \)**

   \[ \text{roof DL + 20\% snow} \]

   North & South exterior walls

   \[ w_{p1} = (16 \text{ psf} + 20\% \cdot 100 \text{ psf})(40')(70') + (85 \text{ psf})(14'/2 + 2')(2 \text{ walls})(70') \]

   \[ = 100,800 \text{ lbs} + 107,100 \text{ lbs} = 207,900 \text{ lbs} \]

   \[ F_{p1} = 0.190 w_{p1} = 0.190 (207,900 \text{ lbs}) = 39,501 \text{ lbs} \]

   \[ w_s = f_{p1} = F_{p1}/L = (39,501 \text{ lbs}) / (70') = 564 \text{ plf} \]

2. **Unit Roof Shear on lines A & B, \( V_r \)**

   \[ V_A = V_B = w_s L / 2 = (564 \text{ plf})(70'/2) = 19,740 \text{ lbs} \]

   Unit roof shear, \( V_A = V_B / d = (19,740 \text{ lbs}) / 40' = 494 \text{ plf} \text{ (SD/LRFD force level)} \)

3. **Maximum Chord Force on lines 1 & 2, \( CF \)**

   \[ \max. M = w_s L^2 / 8 = (564 \text{ plf})(70')^2 / 8 = 345,450 \text{ lb-ft} \]

   \[ \max. CF = (345,450 \text{ lb-ft}) / 40' = 8,636 \text{ lbs} \text{ (SD/LRFD force level)} \]
53. The figure below shows a 1-story Storage building with 8" nominal reinforced concrete masonry (CMU) walls and a flexible metal deck roof diaphragm. For the North-South load direction, the lateral loads will be resisted by 2 exterior masonry shear wall lines (1 & 2) and 3 interior masonry shear wall lines. For the East-West direction, lateral loads will be resisted by the 2 exterior masonry shear wall lines (A & B). Determine the unit roof shear on lines A & B for East-West seismic loads. Given:

- CMU wall weight = 75 psf
- Roof dead load weight = 15 psf
- Wall height (to roof) = 12 feet ... no parapet
- \( V = C_s W = 0.158 W \)

\[
\begin{align*}
1 & \quad 80' \\
& \quad 8'' \text{ CMU walls} \\
& \quad 50' \\
A & \quad \text{N} \\
& \quad \text{W} \\
& \quad \text{E} \quad \text{S}
\end{align*}
\]

\[ \text{Plan} \]

a. 105 plf
b. 170 plf
c. 210 plf
d. 240 plf

54. A light-framed shear wall is framed with 2x6 Douglas Fir studs at 16" on center and sheathed with 15/32" APA rated plywood sheathing attached to the 2x studs and 2x blocking with 8d common nails at 4" on center at panel edges and 12" on center at intermediate framing members. If the shear wall is 20 feet long and 16 feet tall, what is the LRFD and ASD unit shear capacity for resisting seismic forces?

\[ \begin{align*}
a & = 690 \text{ plf LRFD} \quad \text{&} \quad 430 \text{ plf ASD} \\
b & = 860 \text{ plf LRFD} \quad \text{&} \quad 430 \text{ plf ASD} \\
c & = 610 \text{ plf LRFD} \quad \text{&} \quad 380 \text{ plf ASD} \\
d & = 760 \text{ plf LRFD} \quad \text{&} \quad 380 \text{ plf ASD}
\end{align*} \]

55. Nonstructural components that require design in accordance with ASCE 7-10 – Chapter 13 and for which the component importance factor is greater than 1.0 are known as:

a. Seismic force-resisting systems
b. Life-safety systems
c. Essential systems
d. Designated seismic systems
<table>
<thead>
<tr>
<th>Problem</th>
<th>Answer</th>
<th>Reference / Solution</th>
</tr>
</thead>
</table>
| 8       | c      | p. 1-118 - Center of Rigidity, $CR$  
By observation, the $CR$ will be located in the center of the 125 foot building dimension (in the X-direction) because the total rigidity on the left wall line is equal to the total rigidity on the right wall line (i.e., $\sum R = \frac{3}{4} + \frac{3}{4} = 1\frac{1}{2}$).  
$\bar{X}_{CR} = \frac{\sum R_{x} \bar{x}}{\sum R_{x}} = 125' / 2 = 62.5'$ (by observation)  
$\bar{Y}_{CR} = \frac{\sum R_{y} \bar{y}}{\sum R_{y}} = 1.25(0) + 1.25(25') + (1.25 + 1.25)(75') = 43.8'$  
$\therefore (62.5', 43.8')$ |
| 9       | b      | p. 1-185 - Liquefaction  
Shallow saturated cohesionless silty sand |
| 10      | a      | p. 1-151 & ASCE 7-10 p. 76 - §12.11.2.2.1  
max. $L/d = 2.5$ to 1 for subdiaphragms ($SDC = C$ to $F$)  
min. $d = L / 2.5 = (20') / 2.5 = 8.0'$  
$\therefore 8'-0''$ |
| 11      | c      | p. 1-42 - Response Modification Coefficient, $R$  
The $R$ coefficient is representative of the … global ductility of a seismic force-resisting system (SFRS).  
$\therefore$ Response modification coefficient |
| 12      | a      | p. 1-179 - Flat Slab (or Lift Slab) Concrete Buildings  
Partial or total collapse of roof and/or floor slabs due to inadequate slab shear strength at the slab-column joints (i.e., punching shear failure).  
$\therefore$ Punching shear failure of slabs at columns |
| 13      | c      | p. 1-59 - Overturning Moment & ASCE 7-10 p. 73 - §12.8.5  
$OTM_{base} = F_{1} h_{1} + F_{2} h_{2} + F_{3} h_{3}$  
= 15 kips (16') + 25 kips (30') + 20 kips (44')  
= 1870 kip-ft |
| 14      | d      | p. 1-195 - UNSAFE (red) placard  
$\therefore$ Unsafe - do not enter or occupy |
| 15      | c      | p. 1-56 & ASCE 7-10 p. 72 - §12.8.2.1  
$T_{u} = C_{t} h_{n}^{\frac{x}{1 - x}}$  
$C_{t} = 0.03$ & $x = 0.75$  
$\therefore$ ASCE 7 (12.8-7)  
$C_{t} = 0.03$ & $x = 0.75$  
$\rightarrow$ ASCE 7-10 p. 72 - Table 12.8-2 (steel BRBF)  
$T_{u} = 0.03 (140')^{0.75} = 1.22$ sec  
Or using Table C1 (p. 5-18)  
Steel BRBF & $h_{n} = 140'$  
$\therefore T_{u} = 1.22$ sec  
$\therefore T_{u} = 1.2$ second |
<table>
<thead>
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</tr>
</thead>
</table>
| 27      | d      | p. 1-110 - Vertical Flexible Diaphragm Analysis  
**NOTE:** this is a flexible diaphragm and you do not distribute the story shear (or base shear for a 1-story building) based on the shear wall rigidities provided. The rigidities are ignored to determine $V_1$ & $V_2$ (i.e., use tributary area).  
$\therefore V_1 = V_2 = V/2 = 116 \text{ kips}/2 = 58 \text{ kips}$ ← |
| 28      | a      | ASCE 7-10 p. 63 - §12.2.5.2 – Cantilever Column Systems  
ASCE 7-10 – Table 12.2-1, Type G refers to ASCE 7-10 – §12.2.5.2 - “… shall not exceed 15% of the available axial strength, …”  
$\therefore 15\%$ ← |
| 29      | d      | p. 1-117 to 122  
From the figure:  
$\bar{X}_{CM} = 150' \div 2 = 75'$ & $\bar{Y}_{CM} = 100' \div 2 = 50'$  
$\bar{X}_{CR} = 60'$ & $\bar{Y}_{CR} = 50'$  
calculated/inherent eccentricity $e_x = \bar{X}_{CM} - \bar{X}_{CR} = 75' - 60' = 15'$  
accidental eccentricity $e_x = \pm 5\% \ L_x = 5\% (150') = \pm 7.5'$  
The governing (i.e., maximum) force to the shear wall on line 2 will occur when the $CM$ is moved nearest to line 2 where the maximum additive torsional shear will occur:  
$e_{x1} = 15' + 7.5' = 22.5'$  
$M_{T1} = V e_{x1} = 155 \text{ kips} (22.5') = 3488 \text{ kip-ft}$  
$\sum R d^2 = R_x d_1^2 + R_y d_2^2 + R_z d_3^2 + R_{xy} d_4^2$  
$= 3 (60')^2 + 2 (90')^2 + 1.5 (50')^2 + 1.5 (5')^2 = 34,500 \text{ ft}^2$  
$max. \ F_2 = V \frac{R_2}{R_1 + R_2} + \frac{M_{T1} R_{R2}}{\sum R d^2}$  
$= 155 \text{ kips} (2) / (2 + 3) + 3488 \text{ kip-ft} (2)(90 \text{ ft}) / 34,500 \text{ ft}^2$  
$= 62 \text{ kips} + 18 \text{ kips} = 80 \text{ kips}$ ← |
| 30      | b      | p. 1-72 - Redundancy Factor, $\rho$  
Redundancy is a characteristic of structures in which multiple paths of resistance to loads are provided.  
$\therefore \text{ Redundancy}$ ← |
| 31      | b      | p. 1-33 to 35 & 2015 IBC p. 398 - Tables 1613.3.5(1) & 1613.3.5(2)  
Office building w/ fire station in 1st story → IBC Table 1604.5 → RC = IV  
Site Class E & $S_S = 2.13$ → Table 3.2 → $SD_S = 1.28$  
Site Class E & $S_1 = 0.74$ → Table 3.3 → $SD_1 = 1.18$  
$S_1 = 0.74 < 0.75$ → must use Tables 1613.3.5(1) & (2) to determine SDC  
$SD_S = 1.28 \& RC = IV$ → Table 1613.3.5(1) → SDC = D  
$SD_1 = 1.18 \& RC = IV$ → Table 1613.3.5(2) → SDC = D  
$\therefore S'DC = D$ ← |