4.1. Design the positive and negative flexural reinforcement using the strength design method of ACI 318. For the positive reinforcement, use $M_u^+ = 1{,}020{,}000$ ft-lbf. The section resisting positive flexure is a T-beam.

$\boxed{b_w +}$ $\boxed{18\text{ in} +}$

$$d^+ = 36\text{ in} - 2.5\text{ in} = 33.5\text{ in}$$

$$b \le \begin{cases} \dfrac{L}{4} = \dfrac{(36\text{ ft})\left(12\ \dfrac{\text{in}}{\text{ft}}\right)}{4} = \cancel{108}\ \boxed{126}\text{ in} \quad \text{[controls]} \\ b_w + 16h = 18\text{ in} + (16)(8\text{ in}) = 146\text{ in} \\ s = (22\text{ ft})\left(12\ \dfrac{\text{in}}{\text{ft}}\right) = 264\text{ in} \end{cases}$$

$$M_u = \phi M_n = \phi\rho b d^2 f_y\left(1 - 0.59\rho\left(\frac{f_y}{f'_c}\right)\right)$$

$$\begin{aligned}(1{,}020{,}000\text{ ft-lbf})\left(12\ \frac{\text{in}}{\text{ft}}\right) &= 0.9\rho(\cancel{108}\ \boxed{126}\text{ in})(33.5\text{ in})^2\left(60{,}000\ \frac{\text{lbf}}{\text{in}^2}\right) \\ &\quad\times\left(1 - 0.59\rho\left(\frac{60{,}000\ \frac{\text{lbf}}{\text{in}^2}}{4000\ \frac{\text{lbf}}{\text{in}^2}}\right)\right)\end{aligned}$$

$$\begin{aligned}\rho &= \cancel{0.0019}\ \boxed{0.001628} \\ A_s &= \rho b d \\ &= (\cancel{0.0019}\ \boxed{0.001628})(\cancel{108}\ \boxed{126}\text{ in})(33.5\text{ in}) \\ &= 6.87\text{ in}^2\end{aligned}$$

Check the neutral axis depth.

$$\begin{aligned}c = \frac{a}{\beta_1} &= \frac{A_s f_y}{0.85 f'_c b \beta_1} \\ &= \frac{(6.87\text{ in}^2)\left(60{,}000\ \frac{\text{lbf}}{\text{in}^2}\right)}{(0.85)\left(4000\ \frac{\text{lbf}}{\text{in}^2}\right)(\cancel{108}\ \boxed{126}\text{ in})(0.85)} \\ &= \cancel{1.32}\ \boxed{1.13}\text{ in}\end{aligned}$$

The compression region is well within the slab thickness, so the beam is a T-beam as assumed. Steel strain is well above 0.005, so the section is tension controlled with $\phi = 0.9$. Check the minimum steel required.

$$A_{s,\min} \ge \begin{cases} \dfrac{200 b_w d}{f_y} = \dfrac{\left(200\ \dfrac{\text{lbf}}{\text{in}^2}\right)(18\text{ in})(33.5\text{ in})}{60{,}000\ \dfrac{\text{lbf}}{\text{in}^2}} \\ \quad = 2.01\text{ in}^2 \quad \text{[does not control]} \\ \dfrac{3\sqrt{f'_c}\, b_w d}{f_y} = \dfrac{3\sqrt{4000\ \dfrac{\text{lbf}}{\text{in}^2}}\,(18\text{ in})(33.5\text{ in})}{60{,}000\ \dfrac{\text{lbf}}{\text{in}^2}} \\ \quad = 1.91\text{ in}^2 \end{cases}$$

Try no. 8 bars ($A_b = 0.79\text{ in}^2$).

$$n_{\text{bar}} = \frac{A_{s,\text{req}}}{A_b} = \frac{6.87\text{ in}^2}{0.79\text{ in}^2} = 8.7 \quad \text{[use 9]}$$

Bars may need to be bundled to fit the 18 in web width. Determine negative reinforcement for the left end.

$$M_u^- = \phi M_n = \phi\rho b_w d^2 f_y\left(1 - 0.59\rho\left(\frac{f_y}{f'_c}\right)\right)$$

$$\begin{aligned}(1{,}190{,}000\text{ ft-lbf})\left(12\ \frac{\text{in}}{\text{ft}}\right) &= 0.9\rho(18\text{ in})(33.5\text{ in})^2\left(60{,}000\ \frac{\text{lbf}}{\text{in}^2}\right) \\ &\quad\times\left(1 - 0.59\rho\left(\frac{60{,}000\ \frac{\text{lbf}}{\text{in}^2}}{4000\ \frac{\text{lbf}}{\text{in}^2}}\right)\right)\end{aligned}$$

$$\begin{aligned}\rho &= 0.0151 \\ A_s &= \rho b d = (0.0151)(18\text{ in})(33.5\text{ in}) = 9.11\text{ in}^2\end{aligned}$$

Check the neutral axis depth.

$$c < 0.375d = (0.375)(33.5\text{ in}) = 12.7\text{ in}$$

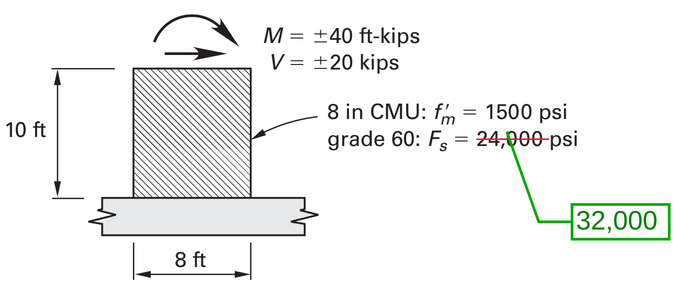
$$\begin{aligned}c = \frac{a}{\beta_1} &= \frac{A_s f_y}{0.85 f'_c b \beta_1} \\ &= \frac{(9.11\text{ in}^2)\left(60{,}000\ \frac{\text{lbf}}{\text{in}^2}\right)}{(0.85)\left(4000\ \frac{\text{lbf}}{\text{in}^2}\right)(18\text{ in})(0.85)} \\ &= 10.5\text{ in} < 12.7\text{ in}\end{aligned}$$

SOLUTION 1

1.1. Design a typical grouted masonry shear wall for the given seismic forces. Neglect the gravity loads, and use grade 60 reinforcement and $f'_m = 1500$ psi, per the problem statement.

Use the working stress design method from ACI 530 Chap. 8. Convert the strength level seismic loads to working stress level. Use the basic load combinations of IBC Sec. 1605.3.1 (with no increase in allowable stresses, per IBC Sec. 1605.3.1.1). According to ACI 530 Sec. 7.3.2.6.1.2, shear must be increased by a factor of 1.5 when checking shear strength, but no increase is required for the overturning moment.

$$\begin{aligned} V &= (0.7)(\pm 28.6 \text{ kips}) \\ &= \pm 20 \text{ kips} \quad \text{[working stress level]} \\ M &= (0.7)(\pm 57.1 \text{ ft-kips}) \\ &= \pm 40 \text{ ft-kips} \quad \text{[working stress level]} \end{aligned}$$



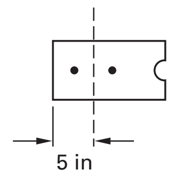
At the base of the wall,

$$\begin{aligned} M_o &= M + Vh = \pm\big(40 \text{ ft-kips} + (20 \text{ kips})(10 \text{ ft})\big) \\ &= \pm 240 \text{ ft-kips} \quad (240{,}000 \text{ ft-lbf}) \end{aligned}$$

Try $d \approx 0.9h = (0.9)(8 \text{ ft})(12 \text{ in/ft}) = 86.4$ in.

$$\begin{aligned} A_s &\approx \frac{M_o}{F_s d} = \frac{(240{,}000 \text{ ft-lbf})\left(12 \ \frac{\text{in}}{\text{ft}}\right)}{\left(32{,}000 \ \frac{\text{lbf}}{\text{in}^2}\right)(86.4 \text{ in})} \\ &= 1.04 \text{ in}^2 \quad \text{[say two no. 7 at each end]} \end{aligned}$$

Two no. 7 bars would give an area of $A_s = 1.20 \text{ in}^2$. Check stresses based on this trial value of A_s (see ACI 530 Chap. 8). Calculate a more accurate estimate of d. Position the bars 4 in and 6 in from the end of the wall. The centroid of the bars is $(4 \text{ in} + 6 \text{ in})/2 = 5$ in from the end of the wall.



$$d = (8 \text{ ft})\left(12 \ \frac{\text{in}}{\text{ft}}\right) - 5 \text{ in} = 91 \text{ in}$$

$$\rho = \frac{A_s}{bd} = \frac{1.20 \text{ in}^2}{(7.625 \text{ in})(91 \text{ in})} = 0.00173$$

$$\begin{aligned} n &= \frac{E_s}{E_m} = \frac{29{,}000{,}000 \ \frac{\text{lbf}}{\text{in}^2}}{900 f'_m} \\ &= \frac{29{,}000{,}000 \ \frac{\text{lbf}}{\text{in}^2}}{(900)\left(1500 \ \frac{\text{lbf}}{\text{in}^2}\right)} \\ &= 21.5 \\ \rho n &= (0.00173)(21.5) = 0.037 \\ k &= \sqrt{(\rho n)^2 + 2\rho n} - \rho n \\ &= \sqrt{(0.037)^2 + (2)(0.037)} - 0.037 \\ &= 0.238 \\ j &= 1 - 0.333k = 1 - (0.333)(0.238) = 0.92 \end{aligned}$$

$$\begin{aligned} f_s &= \frac{M_o}{A_s jd} = \frac{(240{,}000 \text{ ft-lbf})\left(12 \ \frac{\text{in}}{\text{ft}}\right)}{(1.20 \text{ in}^2)(0.92)(91.0 \text{ in})} \\ &= 28{,}700 \text{ psi} \quad [< F_s = 32{,}000 \text{ psi}] \end{aligned}$$

$$\begin{aligned} f_c &= \left(\frac{M_o}{bd^2}\right)\left(\frac{2}{jk}\right) \\ &= \left(\frac{(240{,}000 \text{ ft-lbf})\left(12 \ \frac{\text{in}}{\text{ft}}\right)}{(7.625 \text{ in})(91.0 \text{ in})^2}\right)\left(\frac{2}{(0.92)(0.238)}\right) \\ &= 417 \text{ psi} \end{aligned}$$

$$\begin{aligned} F_c &= 0.45 f'_m = (0.45)\left(1500 \ \frac{\text{lbf}}{\text{in}^2}\right) \\ &= 675 \text{ psi} \quad [f_c < F_c] \end{aligned}$$

Masonry Design