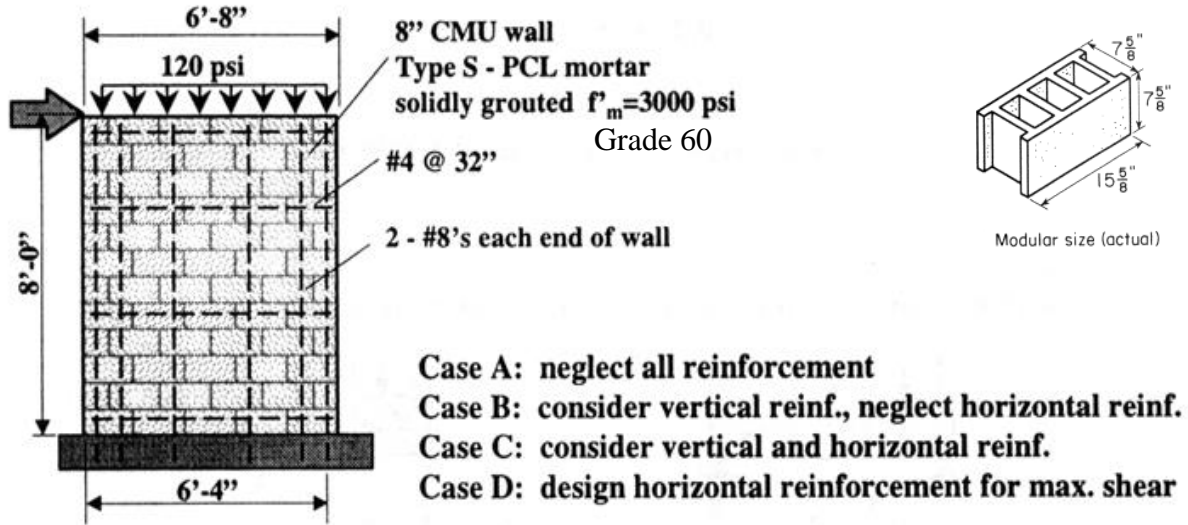


**Examples:
Masonry**

Example 1

Determine the maximum lateral force, H (by wind), as per MSJC.



Case A: neglect all reinforcement

(M=Hx8' moment arm)

$$S_x = \frac{7.63 \times 80^2}{6} = 8139 \text{ in}^3$$

flexure

$$-f_a + M / S = F_t \quad -120 + \frac{96 \times H}{8139} = 86 \quad H = 17,465 \text{ lbs.} = 17.5 \text{ kips}$$

shear

$$F_v = 1.5 \sqrt{f'_m} = 1.5 \sqrt{3000} = 82.2 \text{ psi}$$

$$f_v = \frac{VQ}{I_n b} = \frac{3V}{2A} \text{ (solid rectangle)}$$

$$V_{max} = \frac{2}{3} F_v bt = \frac{2}{3} (82.2 \text{ psi})(7.63 \times 80) = 33.4 \text{ kips}$$

(wall area)

Case B: consider only vertical reinforcement

Flexure: neglecting f_a (allowed stress for grade 60 steel)

$$M_s = A_s F_j d = \underbrace{2 \times 0.79 \text{ in}^2}_{\text{lumping 2 - \#8's}} (32 \text{ ksi})(0.909 \times 72.0'') = 3309 \text{ k-in}$$

$$H_{wind} = 34.5 \text{ kips} = \frac{3309 \text{ k-in}}{8 \text{ ft}(12 \text{ in/ft})}$$

(j=0.909 from ρ_b \ ave. d for 2 bars table in Note Set 23.1)

Example 1 (continued)**Shear**

$$M/Vd = \frac{8.0'}{6.0'} = 1.33 > 1 \quad \text{If } H = V, M/Vd = V(\text{height})/Vd = \text{height}/d$$

$$\text{for } \frac{M}{Vd} > 1 \quad F_{vm} = 2\sqrt{f'_m} = 2.0\sqrt{3000} = 109.5 \text{ psi} \quad f_v = \frac{V}{A_{nv}}$$

$$F_{vm} = \frac{1}{2} \left[\left(4.0 - 1.75 \left(\frac{M}{Vd} \right) \right) \sqrt{f'_m} \right] + 0.25 \left(\frac{P}{A_n} \right) = \frac{1}{2} \left[(4.0 - 1.75(1.0)) \sqrt{3000} \right] + 0.25(120 \text{ psi}) = 91.6 \text{ psi}$$

(M/Vd cannot exceed 1 in the eq.)

$$V_{max} = A_{nv} F_v = (7.63'')(80'')(91.6 \text{ psi})/1000 = 55.9 \text{ kips}$$

(actual width of 8" nominal CMU block)

Case C: consider all reinforcement**Flexure: same as case B****Shear**

$$F_{vm} = 91.6 \text{ psi}$$

$$F_{vs} = 0.5 \left(\frac{A_v F_s d}{A_n s} \right) = 0.5 \left(\frac{(0.20 \text{ in}^2)(32 \text{ ksi})(72 \text{ in})}{(7.63 \text{ in})(80 \text{ in})(32 \text{ in})} \right) 1000 \text{ lb/k} = 11.8 \text{ psi}$$

$$F_v = F_{vm} + F_{vs} = 91.6 + 11.8 = 103.4 \text{ psi} \quad \& \text{ not to exceed } 109.5 \text{ psi } (F_{vmax})$$

$$V_{max} = 103.4 \text{ psi } (7.63 \text{ in})(80 \text{ in})/1000 = 63.1 \text{ kips} \quad f_v = \frac{V}{A_{nv}}$$

Case D: design horizontal reinforcement for maximum shear strength

$$F_{vs} = F_{vmax} - F_{vm} = 109.5 \text{ psi} - 91.6 \text{ psi} = 17.9 \text{ psi}$$

$$s = 0.5 \left(\frac{A_v F_s d}{A_n F_{vs}} \right) = 0.5 \left(\frac{(0.20 \text{ in}^2)(32 \text{ ksi})(72 \text{ in})}{(7.63 \text{ in})(80 \text{ in})(17.9 \text{ psi})} \right) 1000 \text{ lb/k} = 21.1 \text{ in.}$$

using #4 rebars ($A_v = 0.20 \text{ in}^2$) use #4@16 in. horizontal

Example 2

A 12 in. nominal solid brick column, 16 ft high, is built with brick, M mortar, and Grade 40 reinforcement. There are 4 - #4 bars with #2 ties at 8 in. on center. The column must carry an axial load of 63 kips. Check if the column design is adequate. $f'_m = 5,300$ psi.

SOLUTION:

Find the allowable axial load, P_a : which depends on h/r

$$r = \sqrt{I/A} = \sqrt{db^3/12bd} = b/\sqrt{12} = 11.5 \text{ in} \times 0.289 = 3.3 \text{ in} \quad (\text{where } b \text{ is the smallest dimension})$$

$$\text{so } h/r = 16 \text{ ft} \times 12 \text{ in/ft} / 3.3 \text{ in} = 58 < 99$$

$$P_a = [0.25f'_m A_n + 0.65A_{st} F_s] \left[1 - \left(\frac{h}{140r} \right)^2 \right]$$

$$A_s = 4 (0.20 \text{ in}^2) = 0.8 \text{ in}^2$$

$$A_n = 11.5 \text{ in} \times 11.5 \text{ in} - 0.8 \text{ in}^2 = 131.5 \text{ in}^2$$

$$F_s = 20 \text{ ksi},$$

$$P_a = [0.25(5.3 \text{ ksi})131.5 \text{ in}^2 + 0.65(0.8 \text{ in}^2)20 \text{ ksi}] \left[1 - \left(\frac{16 \text{ ft}(12 \text{ in} / \text{ft})}{140(3.3 \text{ in})} \right)^2 \right] = 152.7 \text{ k}$$

Find the bending stress, f_b :

$$f_b = M/S, M = Pe, \text{ where } e = 0.1(11.5 \text{ in}) = 1.2 \text{ in}.$$

$$f_b = 63 \text{ k}(1000 \text{ lb/k})(1.2 \text{ in}) / (11.5 \times 11.5^2 / 6) \text{ in}^3 = 298.2 \text{ psi}$$

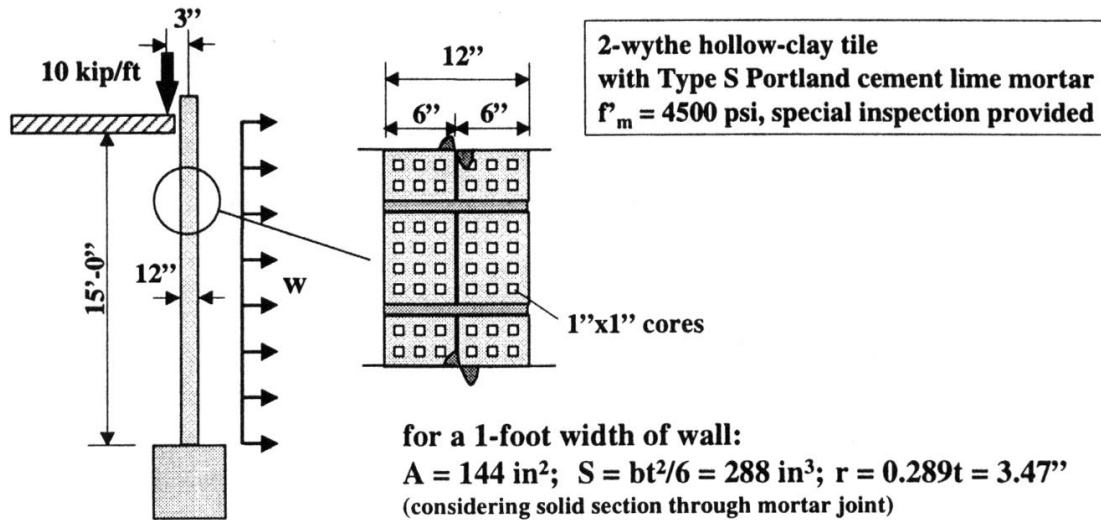
$$\text{Is } \frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \text{ or equivalently } \frac{P}{P_a} + \frac{f_b}{F_b} \leq 1$$

$$F_b = 0.45f'_m = 0.45(5,300 \text{ psi}) = 2385 \text{ psi}$$

$$\frac{63 \text{ k}}{152.7 \text{ k}} + \frac{298.2 \text{ psi}}{2387 \text{ psi}} = 0.54 < 1 \quad \text{OK}$$

Example 3

Determine the maximum transverse wind load, w , per MSJC.



for a 1-foot width of wall:

$A = 144 \text{ in}^2$; $S = bt^2/6 = 288 \text{ in}^3$; $r = 0.289t = 3.47''$
(considering solid section through mortar joint)

$$I = bt^3/12 = 1728 \text{ in}^4$$
; $r = \sqrt{I/A} = \sqrt{1728/144} = 3.464 \text{ in}$

(b is the 1 ft width of wall and t is the thickness)

Case "A" with wind

Weak section has been assumed to be through mortar bed joint. This assumes that unit strength will be at least twice that of the mortar (ratio of mortar area to clay area).

at midheight of wall : $M = \frac{Pe}{2} + \frac{wh^2}{8}$

$$M = 10 \text{ kip} \times \frac{3 \text{ in.}}{2} + \frac{w(15)^2}{8} \times 12 \frac{\text{in.}}{\text{ft.}}$$

$$M = 338w + 15.0$$

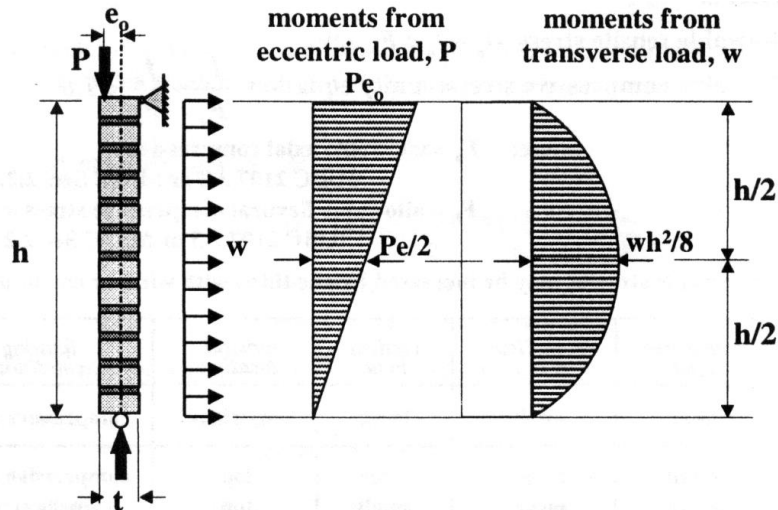
where $w = \text{ksf}$ and $M = \text{kip} \cdot \text{in}$

tension criterion : $-\frac{P}{A} + \frac{M}{S} = F_t = 53 \text{ kips}$ i

$$-\frac{10 \text{ kip}}{144 \text{ in}^2} + \frac{338w + 15.0}{288 \text{ in}^3} = 0.053 \text{ ksi}$$

$w = 60.1 \text{ psf}$

Note: assume F_t for solid units since mortar bed is full with respect to tension normal to bed joint.



for large P and small w : critical location is at *top* of wall: $M = Pe$
 for small P and large w : critical location is *near midheight*: $M = Pe/2 + wh^2/8$

Case "A" with wind

compression criterion :

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} < 1.0$$

$$M = 338 \times 0.060 \text{ ksf} + 15.0 = 35.3 \text{ kip-in}$$

$$f_a = \frac{P}{A} = \frac{10}{144} = 0.069 \text{ ksi} \quad f_b = \frac{M}{S} = \frac{35.3}{288} = 0.123 \text{ ksi} \quad F_b = 0.33f'_m = 0.33(4500 \text{ psi}) = 1500 \text{ psi}$$

$$\frac{h'}{r} = \frac{15 \times 12}{3.47} = 51.8 \quad F_a = 0.25 f'_m \left[1 - \left(\frac{h'}{140r} \right)^2 \right] = 0.216 f'_m = 970 \text{ psi}$$

$$= 0.25(4500 \text{ psi}) \left[1 - \left(\frac{15 \cdot 12 \text{ in}}{140 \cdot 3.47 \text{ in}} \right)^2 \right] = 970 \text{ psi}$$

(psi)

$$\frac{69}{970} + \frac{123}{1500} = 0.071 + 0.082 = 0.153 < 1.0 \text{ ok.}$$

Case "B" without wind

at top of wall : $M = Pe = 30 \text{ kip-in.}$

tension criterion : $-\frac{P}{A} + \frac{M}{S} = F_t = 53 \text{ psi}$

$$-\frac{10 \text{ kip}}{144 \text{ in}^2} + \frac{30 \text{ kip-in}}{288 \text{ in}^3} \leq 0.053 \text{ ksi} ?$$

$$-0.0694 \text{ ksi} + 0.0104 \text{ ksi} = 0.0348 \text{ ksi} < 0.053 \text{ ksi} \text{ ok}$$