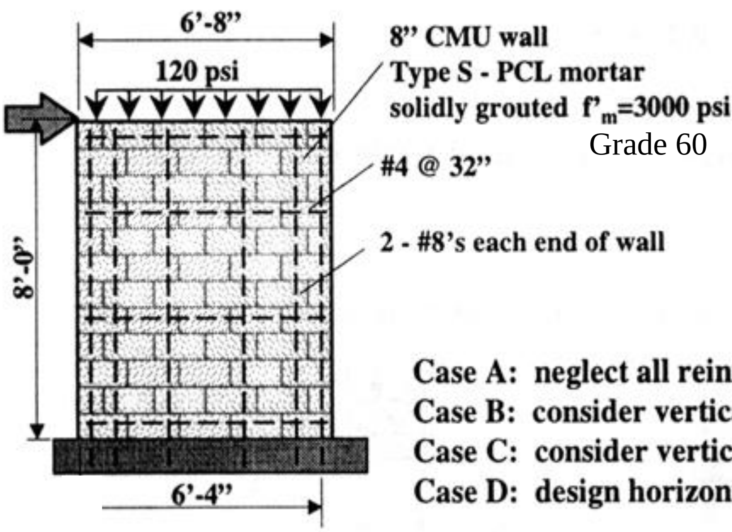
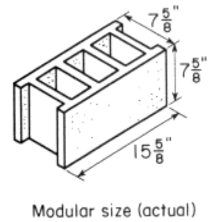
Examples: Masonry

Example 1

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





Case A: neglect all reinforcement
Case B: consider vertical reinf., neglect horizontal reinf.
Case C: consider vertical and horizontal reinf.
Case D: design horizontal reinforcement for max. shear

Case A: neglect all reinforcement

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

(M=Hx8' moment arm)

flexure

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

shear

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

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

$$V_{max} = \frac{2}{3}F_v bt = \frac{2}{3}(82.2\ psi)(7.63x80) = 33.4\ kips$$

(wall area)

Case B: consider only vertical reinforcement

Flexure: neglecting f_a

(allowed stress for grade 60 steel)

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

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

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

Example 1 (*continued*)

Shear

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

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

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

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

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

(actual width of 8" nominal CMU block)

Case C: consider all reinforcement

Flexure: same as case B

Shear

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

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

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

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

Case D: design horizontal reinforcement for maximum shear strength

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

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

using #4 rebars ($A_v = 0.20\ in^2$) <u>*use #4@16 in. horizontal*</u>

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 in x 0.289 = 3.3 in (where b is the smallest dimension)

so h/r = 16ft x 12 in/ft / 3.3 in = 58 < 99

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

A_s = 4 (0.20 in^2) = 0.8 in.^{2}

A_n = 11.5 in x 11.5 in – 0.8 in^2 = 131.5 in^2

F_s = 20 ksi,

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

Find the bending stress, f_b:

f_b = M/S, M = Pe, where e = 0.1(11.5 in) = 1.2 in.

f_b = 63k(1000 lb/k)(1.2 in)/(11.5x11.5^2/6)in^3= 298.2 psi

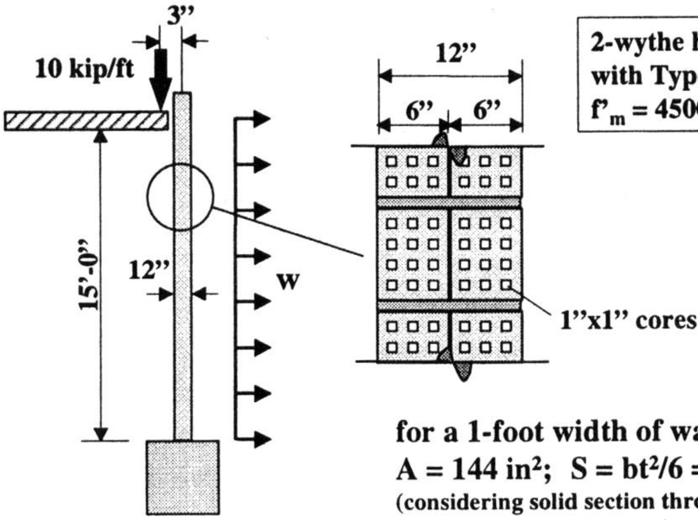
Is $\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1$ or equivalently $\frac{P}{P_a} + \frac{f_b}{F_b} \le 1$

F_b = 0.45f'_m = 0.45(5,300psi) = 2385 psi

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

Example 3

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



2-wythe hollow-clay tile
with Type S Portland cement lime mortar
f'_m = 4500 psi, special inspection provided

for a 1-foot width of wall:
A = 144 in²; S = bt²/6 = 288 in³; r = 0.289t = 3.47"
(considering solid section through mortar joint)

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

(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).

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

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

$$M = 338w + 15.0$$

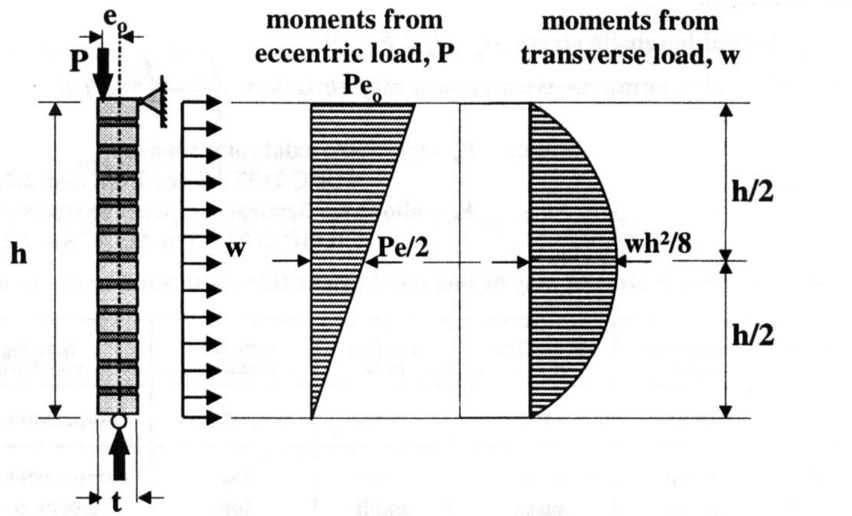
where w = ksf and M = kip - in

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

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

w = 60.1 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 + wh²/8

Case "A" with wind

compression criterion :

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

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

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

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

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

(psi)

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

Case "B" without wind

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

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

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

$$-0.0694\ ksi+0.0104\ ksi=0.0348\ ksi<0.053\ ksi \quad ok$$