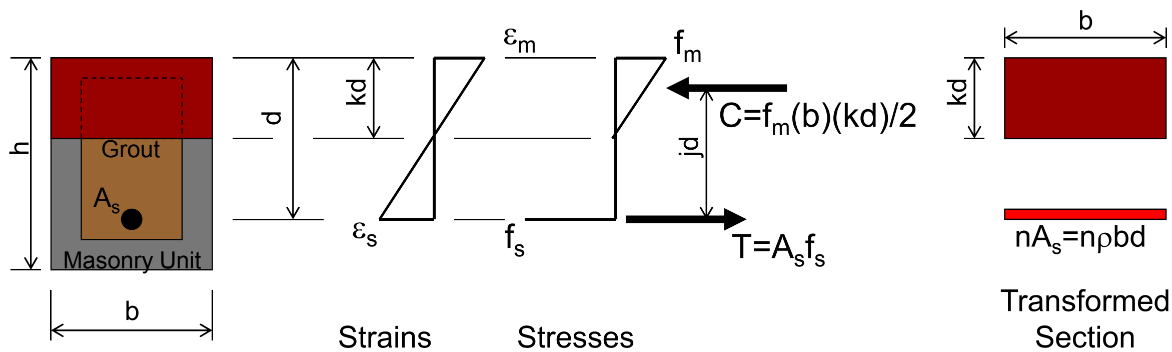
Allowable Stress Design

- 2008: Allowable stresses permitted to be increased by 1/3 for wind and seismic load combinations
- 2011: Major change
  - Allowable stresses no longer allowed to be increased
  - Recalibration of allowable stresses; most increase by approximately one-third
  - Major change to shear design provisions; similar to strength design

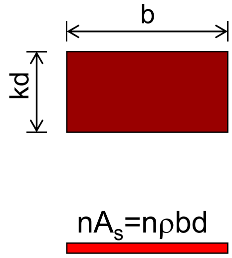
Flexural Members - Allowable Stress Design



Assumptions:
1. Plane sections remain plane
2. Stress-strain relationship for masonry is linear in compression
3. All masonry in tension is neglected
4. Perfect bond between steel and grout
5. Member is straight prismatic section

$$\rho = \frac{A_s}{bd} \qquad n = \frac{E_s}{E_m}$$

Allowable Stress Design



Transformed
Section

To find neutral axis, equate moments of areas about neutral axis.

$$(bkd)(\tfrac{1}{2}kd) = (n\rho bd)(d - kd)$$

$$\boxed{k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho}$$

$$\boxed{j = 1 - \frac{k}{3}}$$

Steel moment: $M_s = A_s f_s jd$

Steel stress: $f_s = \dfrac{M}{A_s jd}$

Masonry moment: $M_m = b(kd)\dfrac{f_m}{2}(jd)$

Masonry stress: $f_m = \dfrac{2M}{b(kd)(jd)}$

<u>Allowable stresses</u> (2.3.2.1, 2.3.3.2.2)

Allowable masonry stress = $^1/_3 f'_m$

Allowable steel stress:

20 ksi	Grade 40 steel
24 ksi	Grade 60 steel
30 ksi	Wire joint reinforcement

Example - Masonry Beam

<u>Given:</u> M=250k-in; Grade 60 steel, f'_m=1500psi; 8 in CMU; Type S mortar; 4 course high beam (d=28 in.); #6 rebar

<u>Required:</u> Is section adequate?

<u>Solution:</u>

F_b =

F_s =

E_m = 1.35 x 10^6 psi

E_s = 29 x 10^6 psi

n = E_s/E_m = 21.5

ρ =

nρ = 21.5(0.00206) = 0.0443

$$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho = \sqrt{(0.0443)^2 + 2(0.0443)} - 0.0443 =$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.256}{3} =$$

Example - Masonry Beam, cont

k=0.256   j=0.914

$$f_s = \frac{M}{A_s jd} =$$

$$f_m = \frac{2M}{b(kd)(jd)} =$$

What is maximum moment beam could carry?

$$M_s = A_s f_s jd =$$

$$M_m = b(kd)\frac{f_m}{2}(jd) =$$

M_{all} = 270 kip-in

Example - Masonry Beam

Given: M=250k-in; Grade 60 steel, f'_m=1500psi; 8 in CMU; depth of section limited to three courses; Type S mortar

Required: Design section

Solution:

F_b = 500 psi
F_s = 24000 psi
E_m = 1.35×10^6 psi
E_s = 29×10^6 psi
n = E_s/E_m = 21.5

For three units, d = 2(8in) + 4in = 20 inches

Assume j=0.9

$$A_s = \frac{M}{F_s jd} = \frac{250k - in}{24ksi(0.9)(20in)} = 0.579in^2$$

Steel	A_s (in^2)	ρ	k	j	f_s (ksi)	f_m (psi)
1 - #7	0.60	0.00393	0.335	0.888	23.5	551
1 - #8	0.79	0.00518	0.374	0.875	18.1	501
1 - #7	0.60	0.00393	0.309	0.897	23.2	591

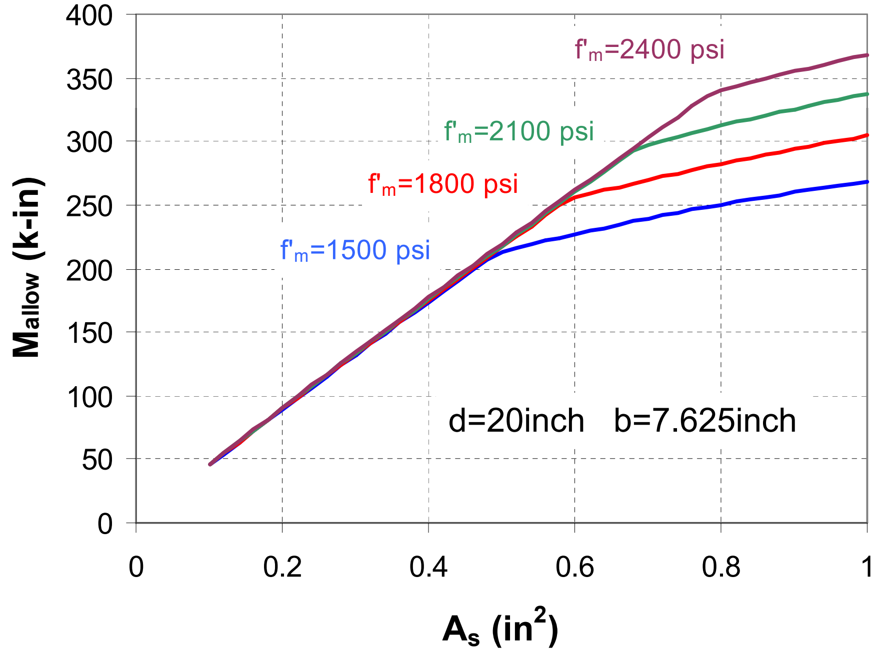
A 32% increase in steel caused a 9% reduction in masonry stress.

Use 1-#8

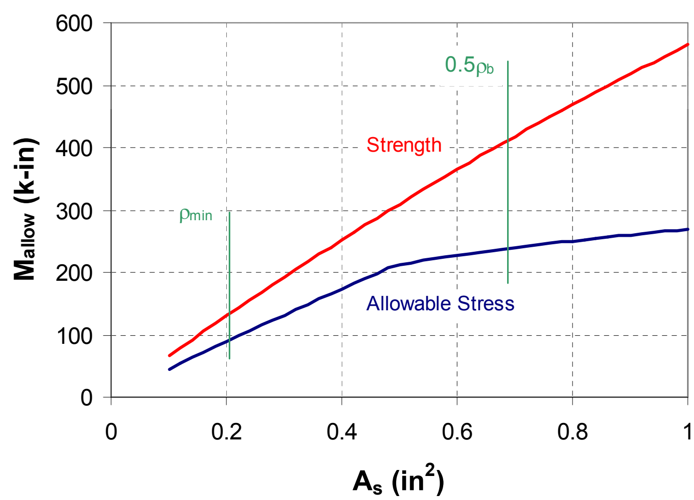
Close enough for government work!

Use stronger block. If blocks are 2500 psi, f'_m=1830psi, E_m=1650ksi, F_b=610psi. Results are shown on last line of table.

Masonry Beam - Parametric Study



Allowable Stress vs. Strength Design



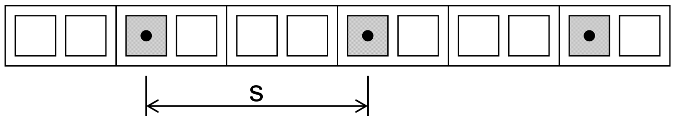
d=20in b=7.625in
f'_m=1.5ksi f_y=60ksi

M_{allow} for strength design obtained as $0.9M_n/1.6$ (assumes all load is live load).

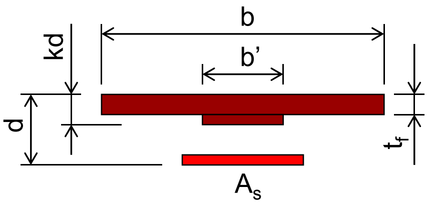
M_n is relatively insensitive to f'_m, although increasing f'_m does change ρ_{max}.

$\rho_{max} = 1.089\ in^2$

Partially Grouted Walls - Allowable Stress



b = width of effective flange = min{s, 6t, 72 in}
(2.3.3.3.1)



A. Neutral axis in flange; design and analysis for solid section
B. Neutral axis in web

$$k = \frac{b}{b'}\sqrt{\frac{t_f^2}{d^2}\left(1-\frac{b'}{b}\right)+(\rho n)^2+2\rho n\left(\frac{t_f}{d}+\frac{b'}{b}-\frac{b'}{b}\frac{t_f}{d}\right)} - \rho n\frac{b}{b'}+\left(1-\frac{b}{b'}\right)\frac{t_f}{d}$$

$$M = C_f j_f d + C_w j_w d \qquad C_f = \frac{f_m}{2}\left(\frac{2kd-t_f}{kd}\right)bt_f \qquad j_f = 1-\frac{t_f}{3d}\left(\frac{3kd-2t_f}{2kd-t_f}\right)$$

$f_m = F_b$ if masonry controlling
$f_m = F_s k/(n(1-k))$ if steel controlling

$$C_w = \frac{f_m}{2}\left(\frac{kd-t_f}{kd}\right)b'(kd-t_f) \qquad j_w = 1-\frac{2t_f+kd}{3d}$$

Partially Grouted Walls - Example

<u>Given:</u> 8 in CMU wall; 12 ft high; Grade 60 steel, f'_m=1350 psi; Lateral load of 20 psf

<u>Required:</u> Reinforcing (place in center of wall)

<u>Solution:</u> $M = \frac{wh^2}{8} = \frac{20lb/ft^2(12in/ft)(12ft)^2}{8} = 4320lb-in/ft = 0.36k-ft/ft$

$$d = \frac{t}{2} = \frac{7.625in}{2} = 3.81in \qquad A_s = \frac{M}{F_s jd} \approx \frac{4.320kip-in/ft}{24ksi(0.9)(3.81in)} = 0.052in^2/ft$$

Try #4 @48 in (0.050in^2/ft)

F_b =	450 psi
F_s =	24000 psi
E_m =	1.215 x 10^6 psi
E_s =	29 x 10^6 psi
n =	E_s/E_m = 23.9

Solve as solid section
ρ = 0.00109
k = 0.204
kd = 0.776 in < t_f = 1.25 in OK
j = 0.932
f_m = 261 psi OK
f_s = 24.3 ksi say OK

Use #4 @ 48 inches

Shear - Allowable Stress Design

A. Masonry carries **all** the shear (2.3.5.2.2)
B. Steel carries **all** the shear (2.3.5.2.3; 2.3.5.3)

Shear strength of cracked beams comes from:
1. Dowel action
2. Aggregate interlock
3. Shear transfer in flexural compression region
4. Tensile strength of uncracked masonry

Aggregate interlock accounts for about 50-70% transfer in reinforced concrete. This will be less in masonry due to possible bond failure between grout and unit, and inherent weakness of mortar joint planes.

Shear - Allowable Stress Design

$$f_v = \frac{V}{bd} \qquad F_v = \sqrt{f'_m} \le 50\,psi$$

If steel is required:

$$A_v = \frac{Vs}{F_s d} \qquad F_v = 3\sqrt{f'_m} \le 150\,psi \qquad s_{\max} = \min\{d/2, 48in.\}$$

Sections within d/2 from face of support can be designed for shear at d/2:
A. Noncantilever beam
B. Reaction introduces compression into end region of member
C. No concentrated load between d/2 and face of support

Compressive Force

Allowable stress design (2.3.3.2.1)

$$P_a = \left(0.25 f'_m A_n + 0.65 A_{st} F_s\right)\left[1 - \left(\frac{h}{140r}\right)^2\right] \qquad \frac{h}{r} \le 99$$

A_{st} is area of laterally tied steel

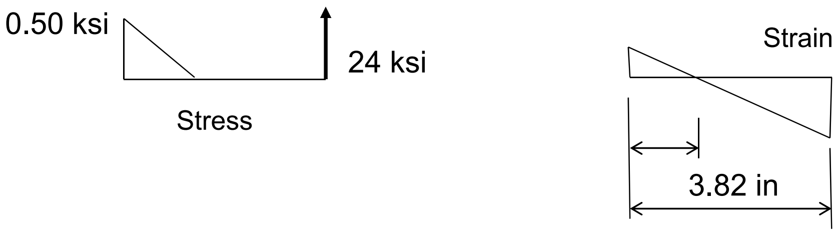
$$P_a = \left(0.25 f'_m A_n + 0.65 A_{st} F_s\right)\left(\frac{70\mathrm{r}}{\mathrm{h}}\right)^2 \qquad \frac{h}{r} > 99$$

Example – 8 in. CMU Bearing Wall

<u>Given:</u> CMU bearing wall, Type S masonry cement mortar; Grade 60 steel in center of wall; #4 @ 48 in.
<u>Required:</u> Construct interaction diagram – ASD; present results in terms of capacity per foot

Example – 8 in. CMU Bearing Wall, ASD

Choose strain/stress distribution (alternatively kd)
Balanced conditions (allowable stress)



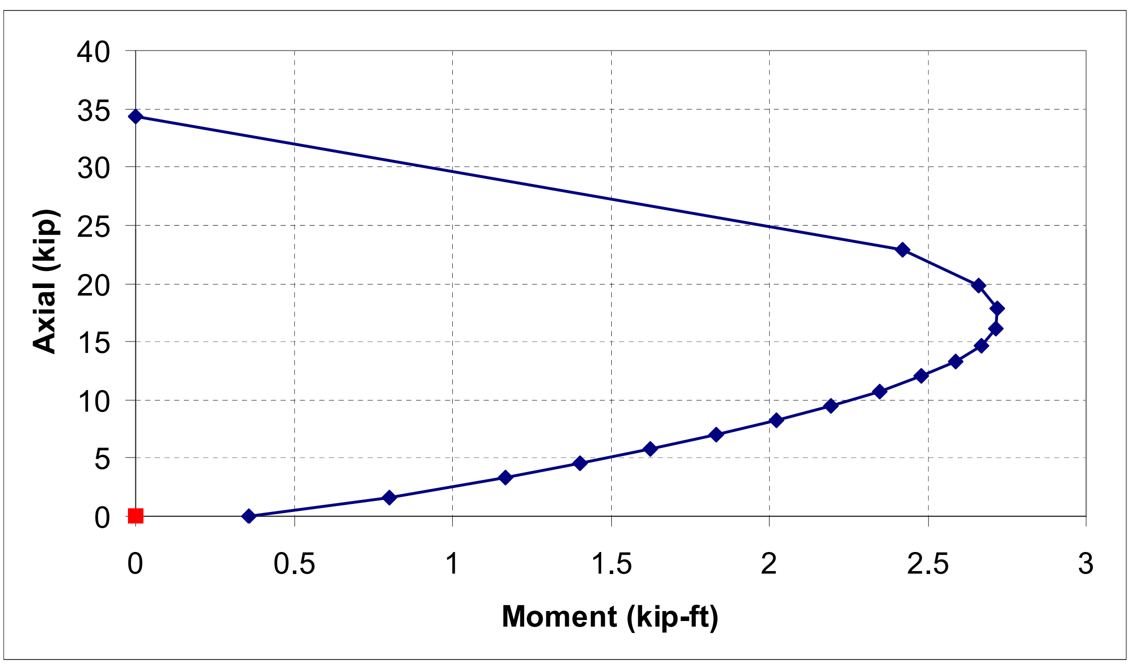
$$C_m =$$

$$T =$$

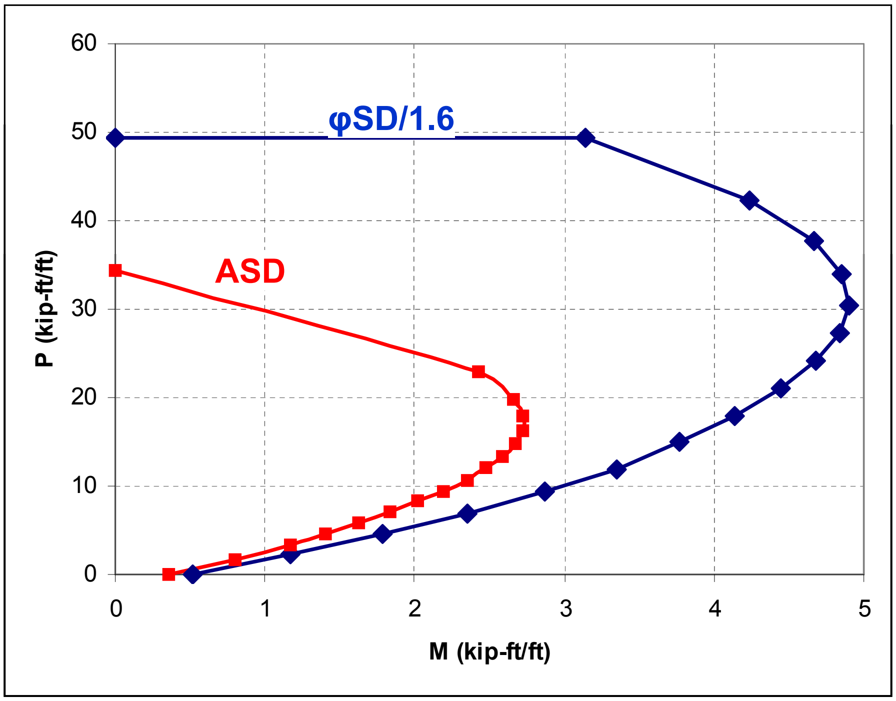
$$P = C_m - T =$$

$$M =$$

Interaction Diagram: ASD



Interaction Diagram: Comparison



Partially Grouted Bearing Wall

- Small ________ forces
  - Partially grouted walls act as _______ walls
  - Compression area is in _______________
- ASD interaction diagram
  - Difficult when neutral axis not in ___________
    - Trapezoidal stress distribution in flange
  - Three point approximation
    - ________________________________
    - _____________________________
    - _____________________________

Allowable Stress - Design Procedure

1. **Select trial size.** For initial guess, consider axial load and moment independently. Size for axial load alone and moment alone.

2. **Assume steel is in tension.** Assume compression controls and neglect compression steel. Determine kd.

$$kd = 3\left[\frac{d}{2} - \sqrt{\left(\frac{d}{2}\right)^2 - \frac{2(P(d-h/2)+M)}{3F_b b}}\right]$$

If kd>d, check compression or increase size of member.
Compare k to k_b, k for balanced conditions.
If k ≥ k_b compression controls.

$$k_b = \frac{F_b}{F_b + \frac{F_s}{n}}$$

$$A_s = \frac{\frac{3F_b(kd)b}{6} - P}{nF_b\left(\frac{1}{k} - 1\right)}$$

Can result in negative area of steel. Use minimum steel in that case.

Allowable Stress - Design Procedure

If k < k_b tension controls. Use iterative procedure. Start with kd from compression controlling.

$$M' = P\left(\frac{h}{2} - \frac{kd}{3}\right)$$

$$A_s = \frac{M - M'}{F_s d\left(1 - \frac{k}{3}\right)}$$

$$\zeta = \frac{(P + A_s F_s)n}{F_s b}$$

$$(kd)_2 = \sqrt{\zeta^2 + 2\zeta d} - \zeta$$

Iterate. Use $(kd)_2$ as new guess and repeat.

Strength Design - Design Procedure

Neglect compression steel, and assume steel yields

1. Determine a, depth of compressive stress block

$$a = d - \sqrt{d^2 - \frac{2\left[P_u\left(d - h/2\right) + M_u\right]}{\phi\left(0.8 f'_m b\right)}}$$

2. Solve for A_s

$$A_s = \frac{0.8 f'_m b a - P_u / \phi}{f_y}$$

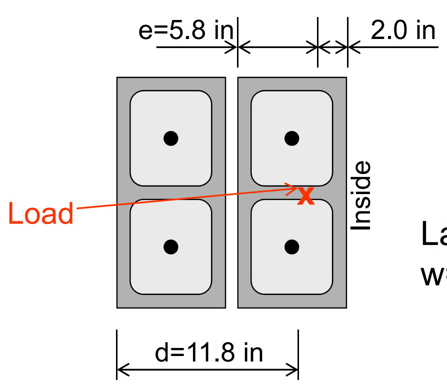
This neglects reduction in axial load due to slenderness effects. A value can be assumed, with 0.9 being reasonable. Design for 1/0.9 or about 10% greater axial load than actually present.

Example - Pilaster Design

Given: Nominal 16 in. wide x 16 in. deep CMU pilaster; f'_m=1500 psi; Grade 60 bar in each corner, center of cell; Effective height = 24 ft; Dead load of 9.6 kips and snow load of 9.6 kips act at an eccentricity of 5.8 in. (2 in. inside of face); Wind load of 16 psf (pressure and suction) and uplift of 5.1 kips (e=5.8 in.); Pilasters spaced at 16 ft on center; Wall is assumed to span horizontally between pilasters; No ties.

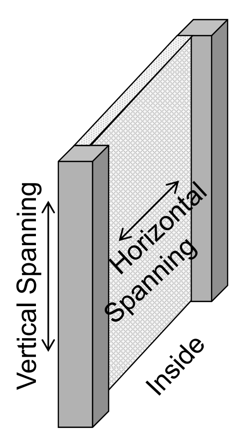
Required: Determine required reinforcing using allowable stress design.

Solution:



E_m = 1350ksi
n = 21.5

Lateral Load
w=16psf(16ft)=256lb/ft



Example - Pilaster Design

D + S

Critical location is top of pilaster. P = 19.2 kips M = 19.2kips(5.8in) = 111.4 kip-in

$$kd = 3\left[\frac{d}{2} - \sqrt{\left(\frac{d}{2}\right)^2 - \frac{2(P(d-h/2)+M)}{3F_b b}}\right]$$

$$=$$

$$k = \frac{kd}{d} = \frac{4.71in}{11.8in} = 0.399$$

$$k_b = \frac{F_b}{F_b + \frac{F_s}{n}} = \frac{0.500ksi}{0.500ksi + \frac{24ksi}{21.5}} = 0.309$$

$k > k_b$
Compression controls

$$A_s = \frac{\frac{3F_b(kd)b}{6} - P}{nF_b\left(\frac{1}{k} - 1\right)} =$$

Example - Pilaster Design

Why the negative area of steel?
Sufficient area from just masonry to resist applied forces.
Determine kd from just compression.

$$kd = \frac{2P}{bF_b} = \frac{2(19.2kip)}{15.6in(0.500ksi)} = 4.92in$$

Find the moment

$$M = P\left(\frac{t}{2} - \frac{kd}{3}\right) = 19.2kip\left(\frac{15.6in}{2} - \frac{4.92in}{3}\right) = 118.3kip - in$$

M_{app} = 111.4 kip-in

Sufficient capacity from just masonry. No steel needed.

Example - Pilaster Design

D + W Check wind suction

At top of pilaster. P = M = 4.5kips(5.8in) = 26.1kip-in

$$M_{max} = \frac{M}{2} + \frac{wL^2}{8} + \frac{M^2}{2wL^2} \qquad x = \frac{L}{2} - \frac{M}{wL}$$

If x<0 or x>L, M_{max}=M

$$x = \frac{L}{2} - \frac{M}{wL} =$$

$$M_{max} = \frac{M}{2} + \frac{wL^2}{8} + \frac{M^2}{2wL^2} =$$

Find axial force at this point. Include weight of pilaster.

$$P = 4.5k + 0.20k/ft(139.8in)1ft/12in = 6.8k$$

Design for P=6.8k, M=234k-in

Weight of pilaster:
Weight of fully grouted 8 in wall (lightweight units) is 75 psf. Pilaster is like a double thick wall. Weight is 2(75psf)(16in)(1ft/12in)=200lb/ft

Example - Pilaster Design

D + W P=6.8k, M=234k-in

kd=7.12in k=0.603 Compression controls A_s=2.96in^2

D + 0.75W + 0.75S At top: P=13.0k, M=75k-in

x=128in M=206k-in P=15.1k

kd=7.28in k=0.617 Compression controls A_s=1.99in^2

0.6D + W At top: P=0.7k, M=4k-in

x=144in M=223k-in P=2.1k

kd=6.07in k=0.514 Compression controls A_s=2.13in^2

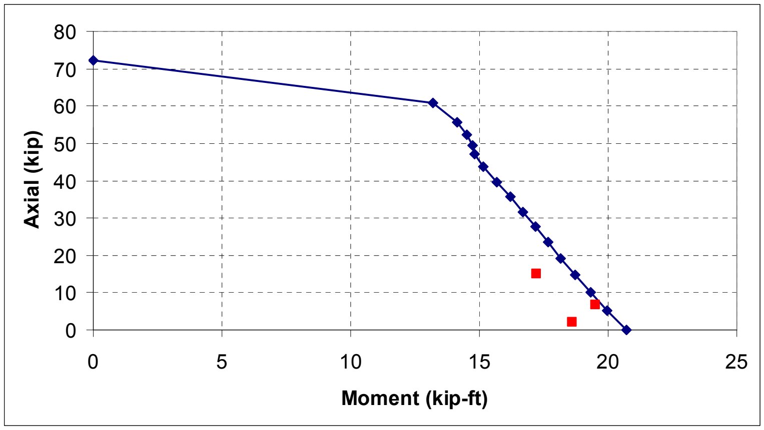
Required steel = 2.96in^2
Use 2-#11 each face, A_s=3.12in^2
Total bars, 4-#11, one in each cell

Example - Pilaster Design

4-#11 is a lot of steel.
Masonry is controlling.

Several options are examined for reducing amount of steel:

1. Increasing f'_m
2. Adding ties
3. Size of pilaster
4. Location of bars



Increasing f'_m: Use an f'_m of 2000 psi; requires units of 2800 psi. Use 4-#8

Adding ties: Steel is effective in compression. Use 4-#9

Size of pilaster: Increase width, make nominal 24in. wide by 16in. deep. Use 4-#7
Increase depth, make nominal 16in. wide by 24in. deep. Use 4-#5

Location of bars: Move bars to 2.5 from face, d = 13.1in. Use 4-#8