MASONRY WALLS

MOST MASONRY STRUCTURAL ELEMENTS ARE WALLS. A WALL IS DEFINED BY SPECIFICATION 1.6 AS A VERTICAL ELEMENT WITH

\[
\frac{L}{t} > 3
\]

WALLS

\[
\begin{align*}
L & = \text{LENGTH OF THE WALL} \\
T & = \text{THICKNESS OF THE WALL}
\end{align*}
\]

STRUCTURAL ELEMENTS WITH ASPECT RATIOS LESS THAN THIS ARE CONSIDERED COLUMNS. THE DISTINCTION BETWEEN WALLS AND COLUMNS IS IMPORTANT BECAUSE RESTRICTIONS FOR EITHER ARE DIFFERENT IN ACI 530.

WALLS HAVE DIFFERENT REINFORCEMENT REQUIREMENTS WHICH ARE GENERALLY LESS RESTRICTIVE RELATIVE TO COLUMNS. THESE RESTRICTIONS INCLUDE

1. SPECIFICATION 1.11 REQUIRES A MINIMUM HORIZONTAL REINFORCEMENT OF 0.0002B TIMES THE GROSS VERTICAL CROSS-SECTIONAL AREA OF THE WALL FOR MOSAICY, OTHER THAN RUNNING BOND.

2. SPECIFICATION 1.15 CONTAIN SPECIAL PROVISIONS REINFORCEMENT RELATIVE TO SEISMIC CATEGORIES DEFINED BY ASCE 7.

NOTE THAT A CLOSELY READING OF THE CODE WILL REVEAL THAT SOME STRUCTURAL ELEMENTS DO NOT FIT THE DEFINITION OF EITHER A WALL OR A COLUMN. IN THIS CASE THE STRUCTURAL ELEMENT IS TYPICALLY DESIGNATED AS A WALL ELEMENT.

WALLS HAVE 3 FUNCTION (SEE FIGURE ON THE NEXT PAGE)

1. RESIST VERTICAL LOADS (WALL LOADS)

2. RESIST BENDING FROM ECCENTRIC VERTICAL LOADS (FLOOR LOADS)

3. RESIST IN-PLANE SHEAR AND BENDING FROM LATERAL LOADS APPLIED TO THE BUILDING SYSTEM
Historically walls were sized in terms of \((h/L)\) ratios. This ratio traditionally was limited to a maximum value of 25. To see where this comes from consider a wall subjected to a wind load of 15 psf.

\[
\begin{align*}
h & \leq 15 \\
wind & = 15 \text{ psf}
\end{align*}
\]
Here the maximum moment at the center of the wall is

\[ M = \frac{wh^2}{B} \leq F_d S \]

where

\[ F_d = \text{historical allowable bending stress} = 50 \text{ psi} \]

\[ S = \text{section modulus per foot of wall} = \frac{t}{6} \]

Thus for

\[ w = 15 \text{ psi} \]

then

\[ \frac{(15 \text{ psi}) h^2}{B} \leq (50 \text{ psi}) \left( \frac{t}{6} \right) \]

\[ \left( \frac{h^2}{t^2} \right) \leq \frac{(50 \text{ psi})(144 \text{ in}^2/\text{ft})(8)}{6 (15 \text{ psi})} \]

\[ \frac{h}{t} \leq 25.3 \]

In addition wall heights were limited to

\[ h < 35 \text{ ft} \]

See spec 5.2 for current limitations on wall height for empirical design. Thus for a maximum wall height

\[ t = \frac{35}{25} = 1.4 = 16.8 \text{ inches} \]
Table 5.5.1 — Wall lateral support requirements

<table>
<thead>
<tr>
<th>Construction</th>
<th>Maximum l/t or h/t</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing walls</td>
<td></td>
</tr>
<tr>
<td>Solid units or fully grouted</td>
<td>20</td>
</tr>
<tr>
<td>All other</td>
<td>18</td>
</tr>
<tr>
<td>Nonbearing walls</td>
<td></td>
</tr>
<tr>
<td>Exterior</td>
<td>18</td>
</tr>
<tr>
<td>Interior</td>
<td>36</td>
</tr>
</tbody>
</table>

In computing the ratio for multiwythe walls, use the following thickness:
1. The nominal wall thicknesses for solid walls and for hollow walls bonded with masonry headers (Section 5.7.2).
2. The sum of the nominal thicknesses of the wythes for non-composite walls connected with wall ties (Section 5.7.3).

Table 5.4.2 — Allowable compressive stresses for empirical design of masonry

<table>
<thead>
<tr>
<th>Construction; compressive strength of unit, gross area, psi (MPa)</th>
<th>Allowable compressive stresses(^1) gross cross-sectional area, psi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Holdwythe masonry of brick and other solid units of clay or shale, sand-lime or concrete brick:</td>
<td></td>
</tr>
<tr>
<td>8000 (55.16) or greater</td>
<td>350 (2.41)</td>
</tr>
<tr>
<td>4500 (31.03) or greater</td>
<td>225 (1.55)</td>
</tr>
<tr>
<td>2500 (17.23) or greater</td>
<td>160 (1.10)</td>
</tr>
<tr>
<td>1500 (10.34) or greater</td>
<td>115 (0.79)</td>
</tr>
<tr>
<td>Grouted masonry of clay or shale, sand-lime or concrete:</td>
<td></td>
</tr>
<tr>
<td>4500 (31.03) or greater</td>
<td>225 (1.55)</td>
</tr>
<tr>
<td>2500 (17.23) or greater</td>
<td>160 (1.10)</td>
</tr>
<tr>
<td>1500 (10.34) or greater</td>
<td>115 (0.79)</td>
</tr>
<tr>
<td>Solid masonry of solid concrete masonry units:</td>
<td></td>
</tr>
<tr>
<td>3000 (20.69) or greater</td>
<td>225 (1.55)</td>
</tr>
<tr>
<td>2000 (13.79) or greater</td>
<td>160 (1.10)</td>
</tr>
<tr>
<td>1200 (8.27) or greater</td>
<td>115 (0.79)</td>
</tr>
<tr>
<td>Masonry of hollow load bearing units:</td>
<td></td>
</tr>
<tr>
<td>2000 (13.79) or greater</td>
<td>140 (0.97)</td>
</tr>
<tr>
<td>1500 (10.34) or greater</td>
<td>115 (0.79)</td>
</tr>
<tr>
<td>1000 (6.90) or greater</td>
<td>75 (0.52)</td>
</tr>
<tr>
<td>700 (4.83) or greater</td>
<td>60 (0.41)</td>
</tr>
<tr>
<td>Hollow walls (noncomposite masonry bonded(^1)):</td>
<td></td>
</tr>
<tr>
<td>Solid units:</td>
<td></td>
</tr>
<tr>
<td>2500 (17.23) or greater</td>
<td>160 (1.10)</td>
</tr>
<tr>
<td>1500 (10.34) or greater</td>
<td>115 (0.79)</td>
</tr>
<tr>
<td>Hollow units:</td>
<td>75 (0.52)</td>
</tr>
<tr>
<td>Stone ashlar masonry:</td>
<td></td>
</tr>
<tr>
<td>Granite</td>
<td>720 (4.96)</td>
</tr>
<tr>
<td>Limestone or marble</td>
<td>450 (3.10)</td>
</tr>
<tr>
<td>Sandstone or cast stone</td>
<td>360 (2.48)</td>
</tr>
<tr>
<td>Rubble stone masonry:</td>
<td></td>
</tr>
<tr>
<td>Coursed, rough, or random</td>
<td>120 (0.83)</td>
</tr>
</tbody>
</table>

\(^1\) Linear interpolation for determining allowable stresses for masonry units having compressive strengths which are intermediate between those given in the table is permitted.
2 Where floor and roof loads are carried upon one wythe, the gross cross-sectional area is that of the wythe under load; if both wythes are loaded, the gross cross-sectional area is that of the wall minus the area of the cavity between the wythes. Walls bonded with metal ties shall be considered as noncomposite walls unless collar joints are filled with mortar or grout.
Recently I have heard the term “Plain Masonry” being used to describe shear walls and other masonry elements. What is plain masonry?

The term Plain Masonry was created to distinguish masonry elements that are structurally unreinforced from those that include nominal reinforcement but which were designed neglecting the contribution of the reinforcement. That sounds a hair convoluted, but let’s describe it with some examples.

Is a concrete masonry wall with joint reinforcement reinforced or not? Well it depends on what you are considering. Yes it contains reinforcement, but if it is designed to span vertically, that joint reinforcement does not contribute to the flexural strength in that direction. It must be designed as unreinforced vertically in conformance with Section 2.2 of the 1999 MSJC Code (ACI 530/ASCE 5/TMS 402 – Building Code Requirements for Masonry Structures). Thus from a structural engineers perspective the wall is unreinforced. From the contractors perspective, however, the wall is reinforced since he is obviously installing reinforcement.

To avoid this confusion, the term Plain Masonry was created since it describes a wall with some reinforcement, but which does not consider that reinforcement in the overall strength of the wall.

It remains to be seen whether this new term will reduce confusion when describing masonry walls, or just add to it. It is, however, important to note that the International Building Code and other codes and standards have begun to use this term extensively to describe shear walls and other masonry elements. The International Building Code, for instance, describes both “ordinary plain masonry shear walls” and “detailed plain masonry shear walls”. Both of these shear wall types are designed neglecting the contribution of any included reinforcement. While there are no reinforcement requirements for ordinary plain masonry shear walls, there are extensive prescriptive minimum reinforcement requirements for detailed plain masonry shear walls as shown in Figure 1, (IBC Sections 2106.1.1.3 and 2106.4.2.3.1). (Response by Phillip Samblanet, The Masonry Society)

Plain Masonry is masonry in which the tensile stress resistance of the masonry is taken into consideration and the effects of stresses in the reinforcement are neglected. (Section 2102.1 of the International Building Code)

---

**Figure 1** - Minimum prescriptive reinforcement required in Detailed Plain Masonry Shear Walls by the International Building Code. The contribution of this reinforcement is neglected when considering the flexural and shear resistance of plain masonry walls. (Courtesy of the National Concrete Masonry Association)
UNREINFORCED MASONRY WALLS

DESIGN OF UNREINFORCED MASONRY WALLS PRIMARILY FALLS UNDER SPEC 2.2 IN ACI 330. SEISMIC CODE CONSIDERATION MUST BE INCLUDED IN THE FOLLOWING:

1. THE UNITY INEQUALITY

\[
\frac{f_a}{f_o} + \frac{f_b}{f_b} \leq 1 \quad \text{Eq. 2-13}
\]

Which has been in masonry codes for a number of years.

2. IN LIEU OF LIMITS ON ASPECT RATIOS (A/H OR L/H) THE MASONRY CODE USES LIMITS ON BUCKLING LOADS

\[
P \leq (1/4) P_e \quad \text{Eq. 2-14}
\]

to more rationally restrict the slenderness ratio. Note that for walls this will be checked on a per foot basis.

3. FLEXURE TENSILE STRESSES CAUSED BY ECCENTRICITY OF AXIAL LOADS, OR BY LATERAL LOADS MUST BE LIMITED TO VALUES LISTED IN TABLE 2.2.3.2. UNDER SPEC 2.2.4 AXIAL TENSILE LOADS MUST BE COUNTER-ACTED BY GRAVITY LOADS, i.e., unreinforced masonry has zero axial tensile load capacity. Consider uplift.

THE UNITY INEQUALITY

\[
\frac{f_a}{f_o} + \frac{f_b}{f_b} \leq 1
\]

can be restated in terms of an axial load per foot of wall (P) by taking

\[
f_a = \frac{P}{A}
\]

\[
f_b = \frac{P_b}{S} = \frac{M}{S}
\]
DEFINE A KINAN ECCENTRICITY AS THE ECCENTRICITY AT WHICH THE EXTREME FIBER STRESS TRANSITIONS FROM COMPRESSION TO TENSION

\[ \frac{P}{A} + \frac{Mc}{I} = 0 \]

\[ \frac{P}{A} + \frac{M}{S} = 0 \]

\[ \frac{P}{A} + \frac{Pe_k}{S} = 0 \]

\[ e_k = \frac{S}{A} \]

NOW, FOR THE GENERAL CASE

\[ 1 \geq \frac{f_a}{f_a} + \frac{f_b}{f_b} \]

\[ 1 \geq \frac{P}{AF_a} + \frac{Pe}{Sf_b} \]

\[ 1 \geq \frac{P}{AF_a} + \frac{Pe}{mc_k f_b} \]
\[
1 \geq \frac{P}{A} \left[ \frac{1}{F_4} + \frac{e}{F_4 \cdot F_5} \right] \\
\geq \frac{P}{AF_4} \left[ 1 + \frac{eF_4}{F_4 \cdot F_5} \right]
\]

Or,

\[
P_{allow} = \frac{AF_4}{\left[ 1 + \frac{eF_4}{F_4 \cdot F_5} \right]}
\]

From the unity check equation (2-13). Note that in this equation

\[e = \frac{M_{max}}{P_{actual}}\]

\[M_{max} = \text{max bending stress in wall due to lateral loads such as wind.}\]

And that

\[P_{actual} \leq P_{allow}\]

Due to restrictions on the slenderness ratio of a wall (Equation 2-14)

\[P_{actual} \leq \left( \frac{1}{F.S. = 4} \right) P_e\]

Where

\[P_e = \frac{\pi^2 E_0 I_m}{h^2} \left[ 1 - (0.577) \left( \frac{e}{t} \right) \right]^3\]

Where

\[e = \frac{M_{max}}{P_{actual}}\]

Which in some respect is a "virtual" eccentricity. Here the eccentricity "e" is the actual eccentricity of the compressive load.

Whereas the code is very specific in regards to a minimum eccentricity for columns, i.e., Spec 2.1.6.2 states

\[e_{min} = (0.1) t\]

Where \(t\) is the column thickness, the code is silent.
On this topic for walls.

However, as the following paper by Colville (2001) indicates an eccentricity of

\[(0.05)h < e < (0.1)h\]

was used to derive the expression

\[F_g = \left( \frac{1}{F_s = 4} \right) f_m' \left( \frac{70r}{h} \right)^2\]

Also note that by Code equation 2-17

\[F_b = \left( \frac{1}{3} \right) f_m'\]

which has a safety factor of 4 built into the expression as follows. Test data indicates that the compressive stress at failure for bending exceeds the compressive stress at failure for axially loaded test specimens by a factor of 3.

\[F_b = \left( \frac{4}{3} \right) \left( \frac{1}{F_s = 4} \right) f_m'\]

\[= \left( \frac{1}{3} \right) f_m'\]
Euler's Buckling Load

It has been shown in the literature that the Euler buckling load for an unreinforced masonry wall is as follows:

\[ P_e = \frac{\pi^2 E_m I_m}{h^2} \left[ 1 - \frac{2}{3} \left( \frac{\ell}{t} \right)^3 \right] \]

This equation assumes no tensile strength in the cross section, a linear relationship between compressive stress and compressive strain, and a fully cracked cross section.

With

\[ r = \left( \frac{I}{A} \right)^{\frac{1}{2}} = \left\{ \left[ \frac{12}{12} \right] - \left[ \frac{1}{\ell (1)} \right] \right\}^{\frac{1}{2}} \]

\[ = \left( \frac{t^2}{12} \right)^{\frac{1}{2}} \]

\[ = (0.289) t \]

Then

\[ P_e = \frac{\pi^2 E_m I_m}{h^2} \left[ 1 - \frac{2}{3} \left( 0.289 \ell \right) \right] \]

\[ = \frac{\pi^2 E_m I_m}{h^2} \left[ 1 - \frac{0.578 \ell}{r} \right] \]

But the radius of gyration assumes a solid rectangular cross section. Although both the first and the last equation for \( P_e \) above give identical answers for solid rectangular cross sections, the ACI-530 committee chose the latter equation.

Since no limitations are given in ACI-530 on the use of the latter equation, this equation can be used not only for solidly grouted sections, but also on hollow and partially grouted sections as well.

Now assume for a moment that

\[ E_m = (1000) f_m' \]

\[ \ell = (0.1) \ell \]

F.S. = 4.0
WITH

\[ I_n = \pi_n r^2 \]

THEN

\[ P = \frac{\pi^2 (1000) \frac{f_m'}{h}}{(FS=4) h^2} \left[ 1 - \frac{2(0.1) t}{t} \right]^3 \]

\[ \frac{P}{A_n} = \left[ \frac{\pi^2 (1000)}{4} \right] \frac{f_m'}{h^2} \left( \frac{0.8}{t} \right)^2 \]

\[ F_o = \left( \frac{5053.23}{4} \right) f_m' \left( \frac{r}{h} \right)^2 \]

\[ = \left( \frac{1}{4} \right) f_m' \left[ \left( \frac{71.1}{h} \right) r \right]^2 \]

which is nearly code equation 2-16. Thus we can get to code equation 2-16 and 2-18 from

\[ P_e = \frac{\pi^2 Em}{h^2} \left[ 1 - \frac{2(0.13)}{t} \right]^3 \]

If we look at analyses of axial load tests in the following graph of axial compressive strength as a function of the slenderness ratio \( h / t \)

![Graph showing the relationship between slenderness ratio and axial compressive strength with Test Results](image)

\[ R = \frac{F_o}{f_m} \]

Thus code equations 2-15 and 2-16 are the result of curve fits to experimental data.
Finally, it is interesting to note that for

\[ e = \frac{t}{2} \]

then

\[ P_e = 0 \]

This presents a significant problem when attempting to attach ledger beams to the side of a wall, when

\[ e > \frac{t}{2} \]

and the expression for \( P_e \) goes negative.
DETERMINE THE ALLOWABLE VERTICAL LOAD CAPACITY OF THE REINFORCED CAVITY WALL DEPICTED BELOW. ASSUME:

a.) $f_m' = 2500$ PSI

b.) $f_m' = 1500$ PSI

![Diagram of wall with legend: ungrouted, 8" cmu, face-shell bedding, 1.25", 7.63", 2", 3.63", Pa, 4" brick, metal ties.

Concrete footing 20' (nts)

NOTE THAT THE BRICK WYTHE IS NOT CONSIDERED "LOAD BEARING" IN THIS PROBLEM. FOR THE CONDITIONS CITED ABOVE,

\[
\begin{align*}
I &= 30.0 \text{ in}^4/\text{ft} \text{ of wall} \\
E &= 3009 \text{ in}^4/\text{ft} \text{ of wall} \\
\tau &= 3.21 \text{ in} \text{/ ft} \text{ of wall}
\end{align*}
\]

Thus,

\[
\frac{h}{r} = \frac{12 \times 20}{3.21} = 74.8
\]

Thus,

\[
F_a = \left(\frac{1}{4}\right) f_m' \left[ 1 - \left(\frac{h}{14r} \right)^2 \right]
\]
a.) \[ F_a = \left( \frac{L}{4} \right) (2500) \left\{ 1 - \left[ \frac{12 (20)}{140 (3.21)} \right]^2 \right\} \]

\[ = 447 \text{ psi} \]

\[ P_a = \frac{(447)(30)}{1000} = 13.4 \text{ k/ft} \]

\[ P_e = \frac{\pi^2 E_y I_n}{I_n^2} \left[ 1 - 0.577 \left( \frac{a}{r} \right) \right]^3 \]

\[ = \frac{\pi^2 (900) (2500) (309)}{1000 \left[ (12)^2 (20) \right]^2} \left[ 1 - 0.577 \left( \frac{3}{3.21} \right) \right]^3 \]

\[ = 119 \text{ k/ft} \]

\[ P = \left( \frac{1}{4} \right) P_e = \left( \frac{1}{4} \right) (119) = 29.8 \text{ k/ft} \]

\[ > 13.4 \text{ k/ft} \quad \text{contrasts} \]

b.) \[ F_a = \left( \frac{L}{4} \right) (1500) \left\{ 1 - \left[ \frac{12 (20)}{140 (3.21)} \right]^2 \right\} \]

\[ = 268 \text{ psi} \]

\[ P_a = \frac{268(30)}{1000} = 8.04 \text{ k/ft} \]

\[ P_e = \frac{\pi^2 (900) (1500) (309)}{1000 \left[ (12)^2 (20) \right]^2} = 71.4 \text{ k/ft} \]

\[ P_a = \left( \frac{1}{4} \right) (71.4) = 17.9 \text{ k/ft} \]

\[ > 8.04 \text{ k/ft} \quad \text{contrasts} \]
DETERMINE THE ALLOWABLE VERTICAL LOAD CAPACITY OF THE UNREINFORCED MASONRY WALL DEPICTED BELOW. NOTE THAT

\[ f_m' = 2000 \text{ PSI} \]

\[ A = 30 \text{ in}^2/\text{ft} \]
\[ I = 300 \text{ in}^4/\text{ft} \]
\[ S = 81 \text{ in}^3/\text{ft} \]
\[ r = 3.21 \text{ in} \]

TYPE 3 MORTAR

CHECK FLEXURAL TENSION, FROM TABLE 2.2.3.2

\[ f_t = 25 \text{ PSI} \]

Hence the flexural stress is normal to the bed joint. Thus

\[ -P_a + \frac{f_0}{S} \leq f_t \]

\[ -\frac{P_a}{4} + \frac{P_a e}{S} \leq 25 \]

\[ -\frac{P_a}{30} + \frac{3P_a}{81} \leq 25 \]

\[ P_a \left( \frac{2}{81} - \frac{1}{30} \right) \leq \frac{25}{1000} \]

\[ P_a \leq 6.75 \text{ klf} \]
Next, check compression and bending.

\[ F_B = \left( \frac{1}{3} \right) f_m' = \left( \frac{1}{3} \right) 2000 = 667 \text{ PSI} \]

\[ E_m = (900) f_m' = 900 \times 2000 = 1,800,000 \text{ PSI} \]

\[ \frac{h}{r} = \frac{12 \times (20)}{3.21} = 74.8 < 99 \]

Thus

\[ F_B = \left( \frac{1}{4} \right) f_m' \left[ 1 - \left( \frac{h}{140 r} \right)^2 \right] \]

\[ = \left( \frac{1}{4} \right) (2000) \left[ 1 - \left( \frac{12 \times (20)}{140 	imes (3.21)} \right)^2 \right] \]

\[ = 357 \text{ PSI} \]

Using the unity equation (2-10, page C-25)

\[ \frac{F_B}{F_B} + \frac{F_B}{F_B} \leq 1 \]

\[ \left( \frac{1}{337} \right) \left( \frac{P_a}{30} \right) + \left( \frac{1}{667} \right) \left( \frac{3 P_a}{81} \right) \leq 1 \]

\[ P_a \left[ \frac{1}{337(30)} + \frac{3}{667(81)} \right] \leq \frac{1}{1000} \]

\[ P_a \leq 6.475 \leq 1 \text{ kips} \]

Finally, check buckling

\[ P \leq \left( \frac{1}{4} \right) P_c = \left( \frac{1}{4} \right) \frac{3 E t}{h^2} \left[ 1 - 0.577 \left( \frac{r}{h} \right) \right]^3 \]

\[ \leq \left( \frac{1}{4} \right) \frac{3 \times 2000 \times 2000 \times (200)}{12 \times (20)^2} \left[ 1 - 0.577 \left( \frac{3}{3.21} \right) \right]^3 \]

\[ \leq 2.35 \text{ kips} \]
EVALUATE THE ADEQUACY OF A CMU WALL WITH 12" BLOCK LAYED IN FULL BEDDING. THE WALL IS 15 FT. LONG, AND 10 FT. HIGH. THE WALL SUPPORTS THE FOLLOWING LOADS:

**From Above:**
- $D_L = 6.67 \, k/ft$
- $L_L = 6.67 \, k/ft$

**Floor Loads (one side only):**
- $D_L = 1.67 \, k/ft$
- $L_L = 1.67 \, k/ft$

$f_{m'} = 1500 \, psi$

*Type S Mortar*

**Section Properties for 12" CMU, Full Mortar Bedding:**
- $A = 57.8 \, in^2/ft \, of \, wall$
- $I = 1065 \, in^4/ft \, of \, wall$
- $s = 133 \, in^3/ft \, of \, wall$
- $r = 4.29 \, in$

IF WE CONSIDER THE FULL FLOOR LOADS (NO CHECKERBOARD):

- $P = 12(6.67)(15) + 2(1.67 + 1.67)(15) = 300 \, k$
- $M = 0$

Note that these are total loads, not loads per foot of wall. Hence,

$$\frac{h}{r} = \frac{10(12)}{4.29} = 28 < 99$$

Thus,

$$F_x = \frac{1}{4} f_{m'} \left[ 1 - \left( \frac{h}{140} \right)^2 \right]$$

$$= \frac{1}{4} (1500) \left[ 1 - \left( \frac{28}{140} \right)^2 \right]$$

$$= 360 \, psi$$
\[ f_c = \frac{P}{A} = \frac{300 \text{ (1000)}}{57.8 \text{ (15)}} = 346 \text{ psi} \]

\[ < 360 \text{ psi \ OK} \]

Check Spec Eq. 2-11.

\[ P_e = \frac{\pi^2 EI}{k^4} \left[ 1 - (0.527) \left( \frac{e}{k} \right) \right] \]

\[ = \frac{\pi^2 \times 900 \text{ (1500)} \times 1000 \times (10 \text{ (12)} \right)}{1000 \times 100 \times 12} \left[ 1 - 0 \right] \]

\[ = 24,635 \text{ kN} \]

By Spec 2.2.3.1

\[ P = \frac{P_e}{4} = \frac{24,635}{4} = 6159 \text{ kN} \]

\[ \geq 300 \text{ kN \ OK} \]

Next, consider checkboard loading. Note

\[ P = 300 - \text{Floor live load} \]

\[ = 300 - 25 = 275 \text{ kN} \]

Calculate eccentricity

\[ e = \frac{t}{2} - \frac{4.03}{3} \]

\[ = \frac{11.98}{2} - \frac{4}{3} \]

\[ = 4.48 \text{ in} \]

Compute the moment from the eccentric axial live load. Note that the dead load will still come into the wall along the center line because it is balanced on both sides of the wall, thus the moment transmitted to the wall is

\[ M = 25 \times (4.48) = 112 \text{ kN} \cdot \text{in} \]
The axial stress under the reduced line load is

\[ f_x = \frac{P}{A} = \frac{225,000}{57.8 \text{ (15)}} = 3,172 \text{ psi} \]

The tensile bending stress is

\[ f_{bt} = \frac{112 \text{ (1000)}}{(183 \text{ (15})} = 40.8 \text{ psi} \]

\[ < 190.3 \text{ psi} = f_x \]

Thus there is no net tensile stress in the wall. Check interaction equation (SPEC 2.2.3.1, EQUATION 2-13)

\[ \frac{f_x}{f_y} + \frac{f_{bt}}{f_b} = \frac{3,172}{360} + \frac{40.8}{500} \]

\[ = 0.88 + 0.08 \]

\[ = 0.96 \]

\[ < 1.0 \text{ OK} \]

\[ e = \frac{M}{P} = \frac{112}{275} = 0.407 \text{ in.} \]

"virtual" e

Check equation 2-18

\[ P_c = \frac{\pi^2 EI}{h^2} \left[ 1 - (0.577) \left( \frac{e}{h} \right) \right]^3 \]

\[ = \frac{\pi^2 (900)(1500)(1065)(15)}{1000 \left[ (10)(12) \right]^2} \left[ 1 - (0.577) \left( \frac{0.407}{4.29} \right) \right]^3 \]

\[ = 1,498 \text{ k} \]

By SPEC 2.2.3.1

\[ P \leq \frac{P_c}{4} = \frac{1,498}{4} = 374.5 \text{ k} \]

\[ > 275 \text{ k} \text{ OK} \]
EVALUATE THE ADEQUACY OF 12" CMU BLOCK WALL IN THE LAST PROBLEM. BUT THIS TIME ASSUME THE BLOCK IS PLACED WITH FACE SHELL BEDDING. ONCE AGAIN

\[ f_m' = 1500 \text{ PSI} \]

TYPE 5 MORTAR

**SECTION PROPERTIES FOR A 12" CMU BLOCK WALL, FACE SHELL BEDDING ARE**

\[ A = 36.0 \text{ in}^2/\text{ft of wall} \]
\[ t = 9.29 \text{ in}^4/\text{ft of wall} \]
\[ s = 100 \text{ in}^3/\text{ft of wall} \]
\[ r = 5.08 \text{ in} \]

IN THIS PROBLEM

\[ \frac{h}{r} = \frac{10(12)}{5.08} = 23.4 < 99 \]

**Thus by 2-15**, \[ F_a = \left(\frac{1}{4}\right) f_m' \left[ 1 - \left(\frac{h}{140r}\right)^2 \right] \]

\[ = \left(\frac{1500}{4}\right) \left[ 1 - \left(\frac{23.4}{140}\right)^2 \right] \]

\[ = 364.3 \text{ PSI} \]

**CHECK APPLIED STRESS**

\[ f_a = \frac{P}{A} = \frac{300(1000)}{36(15)} = 555.5 \text{ PSI} \]

\[ > 364.3 \text{ PSI NO GOOD} \]

**TRY**

\[ f_m' = 2500 \text{ PSI} \]

FOR HOMEWORK (FACE SHELL BEDDING)
AVOIDING REINFORCEMENT CONGESTION

1.3-2 I have heard that clumping steel in jambs of shear walls and in the bottoms and tops of lintels is not good practice. I understand that clumping the steel can cause congestion problems, but are there any other reasons why I should avoid clumping the steel? If I distribute the steel more, won't I decrease the moment capacity and thus need more steel reinforcement? Isn't this counterproductive?

Concentrating flexural reinforcement at the extreme fibers of flexural elements has one advantage, but several disadvantages.

- The advantage is that it can maximize the efficiency of use of the reinforcement, by placing it the greatest possible distance apart. This increases flexural capacity for a given amount of steel, and also slightly increases flexural ductility.

- The disadvantages are that it can lead to congestion of reinforcement; that it can decrease resistance to shear failure along a section perpendicular to the axis of the member (for example, along the bed joints of walls; and that it can lead to more rapid deterioration of the compression toe of the wall.

The principal negative consequences of congestion are inadequate bond (because of the inability to get the grout to flow around congested bars, and an increased tendency toward longitudinal splitting because of the tensile forces produced around the bars, and the small cover around a zone of congested reinforcement.

(Response by Richard E. Klingner, University of Texas, Austin)

Code Requirements to try to Prevent Reinforcement Congestion

Numerous general requirements and recommendations on ways to prevent reinforcement congestion have been used in the past, and some requirements are given in masonry codes. One "rule-of-thumb" recommendation given to many beginning designers is to limit bar designation (in US units only) to the nominal wall thickness (I in inches, or the wall thickness minus one. Thus, the maximum bar size would be a No. 8 (or a No. 7 if you used r-1) for an 8 in. masonry, a No. 6 (or a No. 5 for r-1) for a 6 in. wall, etc. These easy to apply "rules-of-thumb" generally limit congestion effectively, although they are not code requirements for all masonry. The following summarizes some of the main code requirements to limit reinforcement congestion. References cited are designated as the MSJC-99 for the 1999 ACI 530/ASCE 5/TMS 402, Building Code Requirements for Masonry Structures, MSJC-02 for the draft provisions being considered by the MSJC, and the IBC-00 for the 2000 Edition of the International Building Code. While not logical, requirements in these codes and standards often differ for masonry designed by the Working Stress Design Method and the Strength Design Method. The reader should review the applicable requirements carefully to ensure the appropriate provisions are being applied to the job.

Maximum Bar Size
No. 11 (M #36) (1.12.2.1 of MSJC-99)
No. 9 (M #29) (for Strength Design by MSJC-02, 3.2.3.1)

Maximum Bar Size Per Wall Thickness (See Table 1)
1/8 nominal wall thickness (IBC 2107.2.4, IBC 2108.9.2.1, MSJC-02 3.2.3.1 for Strength Design)

Maximum Bar Size Per Cell Thickness (See Table 2)
1/2 clear cell or collar joint thickness (MSJC-99, 1.12.2.2)
1/4 clear cell or collar joint thickness (IBC 2107.2.4, IBC 2108.9.2.1, MSJC-02 3.2.3.1 for Strength Design)

Maximum Bar Area Per Grout Space Area (See Table 3)
6% of Grout Space Area (MSJC-99, Table 1.5.2, Footnote 4) Applies to Vertical Reinforcement
4% of Grout Space Area (IBC 2107.2.4, IBC 2108.9.2.1, MSJC-02 3.2.3.1 for Strength Design)

Table 1 - Bar Size Limitations Recommended by Many and Required by Some Codes

<table>
<thead>
<tr>
<th>Nominal Wall Thickness, t (in.)</th>
<th>4</th>
<th>6</th>
<th>8</th>
<th>10</th>
<th>12</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/8 t (in.)</td>
<td>0.5</td>
<td>0.75</td>
<td>1.0</td>
<td>1.25</td>
<td>1.5</td>
</tr>
<tr>
<td>Bar Designation meeting 1/8t and other requirements</td>
<td>No. 4</td>
<td>No. 6</td>
<td>No. 8</td>
<td>No. 9</td>
<td>No. 11</td>
</tr>
</tbody>
</table>

1Based on MSJC-02 Section 3.2.3.1 and IBC Sections 2107.2.4 and 2108.9.2.1
2While 4 in. nominal hollow clay masonry units are available, most 4 in. nominal hollow concrete masonry is nearly solid, and thus cannot be reinforced and grouted. Check with local concrete masonry unit supplier on unit availability
3No. 9 is listed since the nominal diameter of a No. 10 bar is 1.27, which slightly exceeds 1/8"
4Maximum bar size in masonry is No. 11.
5MSJC-02, 3.2.3.1 would limit bar size to No. 9.
CRACK CONTROL OF CMU
NOW THAT TYPE REQUIREMENTS ARE
NO LONGER IN ASTM C 90

1.3-3 Were Type I and Type II requirements for CMU removed from ASTM C 90? If so, why?

Yes. Prior to 2000, ASTM C 90 included two different type designations for concrete masonry units: Type I units were defined as moisture-controlled units, Type II units were referred to as non-moisture-controlled units. Requirements for these different unit types were identical in all respects with the exception: the moisture content of the unit at the time of delivery. Historically, ASTM C 90 stipulated a maximum moisture content for Type I units at the time of delivery. Conversely, no such moisture content requirements were specified for Type II units.

The magnitude of permissible moisture for Type I units was determined from a table in ASTM C 90 and was based on the tested shrinkage potential of the unit (ASTM C 425) and the mean annual relative humidity of the area where the unit was to be used. The moisture content requirements for Type I units were intended to limit the potential residual drying shrinkage of the unit and thus reduce the potential for wall cracking. Hence a Type I unit with low potential drying shrinkage was permitted to have a higher moisture content at the time of delivery and likewise, a unit to be used in a humid environment was also permitted to be at a higher moisture content at the time of delivery.

The phrase “at the time of delivery” is key to this discussion. ASTM C 90 is a manufacturing specification. The concrete masonry producer is responsible for supplying units that comply with the requirements of the specification. The moisture content of a concrete masonry unit changes based on their environment and thus may rise above and fall below Type I moisture content requirements several times before being laid. ASTM C 90 only requires Type I units to comply with moisture content requirements at the time they are delivered, and does not address the moisture content of the units at the time they are laid since this is out of the control of the manufacturer. Rarely are the units verified in the field to determine compliance at the exact time of construction since this is difficult, expensive, and will vary considerably based on the daily conditions at the jobsite.

Manufacturers in some geographical areas with humid environments have informed specifiers for years that they could not produce Type I units because of the inability or cost of completely protecting units from their environment. Because specifying Type I units did not ensure the moisture content of the units at the time of placement, the confusion regarding these limits, and the difficulty in complying with the requirements in many areas, the provisions have been removed from ASTM C 90.

(continued on Page 4)
Adequate protection against shrinkage cracking in concrete masonry can be obtained without Type I specifications. Historically, cracking has been alleviated using control joints, and reinforcement has also often been used to control cracking. Control joints are placed in concrete masonry to relieve stress in the wall and reduce cracking that results from unit shrinkage. Reinforcement does not prevent cracks, but rather keeps them acceptably narrow. At one time, the concrete masonry industry had different control joint and horizontal reinforcement recommendations for walls depending on whether they were constructed with Type I or II units. In 1998, those recommendations were revised to that shown in Figure 1. These recommendations are conservative enough to apply to units of any moisture content, and thus negate the need to distinguish between Type I and II. NCMA TEK 10-2A and the recently released 3rd Edition of the Masonry Designers’ Guide describe movement control of concrete masonry in more depth, and are recommended to readers looking for more information on this topic. (Response by Robert D. Thomas, National Concrete Masonry Association)

![Diagram](image)

**Figure 1 - Recommendations for Crack Control of Concrete Masonry Walls from NCMA**

---

**Disclaimer**

This document is intended to provide explanation of typical and not-so-typical questions regarding masonry design, construction, evaluation and repair. It is intended for masonry design professionals, architects, engineers, inspectors, contractors, manufacturers, building officials, students, and others interested in masonry. It is not intended to cover every aspect of the discussed topics, but rather to focus on key issues that should be considered and addressed. This document should not be used as the sole guide for designing, constructing, evaluating or repairing masonry. It is imperative to refer to relevant building codes, standards and other industry-related documents. As such, TMS assumes no liability for any consequences that may follow from the use of this document. In addition, the opinions, ideas and suggestions given herein are those of the respondent, and not necessarily those of The Masonry Society.

This document is produced bimonthly by:

The Masonry Society
3970 Broadway, Suite 201-D
Boulder, CO 80304-1135
Phone: (303) 939-9700,
Fax: (303) 541-9215
Website: www.masonrysociety.org

Oversight: TMS Design Practices Committee, William A. Wood, chair
Editors: Edwin T. Huntin, Vilas Mujumdar, Phillip J. Sammut and William A. Wood

Questions, ideas, suggestions and differing opinions may be sent to TMS for consideration for inclusion in future issues of TMS Responds.
INTERPRETING KENL DISTANCES FOR URM WALLS

ASSUME THERE IS NO NET APPLIED TENSILE STRESS IN AN UNREINFORCED MASONRY (URM) WALL SUBJECTED TO BOTH MOMENT AND AXIAL LOAD.

\[ P_0 + f_{bt} = 0 \]

\[ -\frac{P}{A} + \frac{Mc}{I} = 0 \]

\[ A = bt \quad I = \frac{bt^3}{12} \]

\[ -\frac{P}{bt} + \frac{P_e (t/2)}{(bt^3/12)} = 0 \]

\[ \epsilon = \frac{t}{b} \]

IF LOAD IS WITHIN THE KENL, THEN THERE IS NO NET TENSILE STRESS.

NOTE: THIS KENL DISTANCE, i.e., the KENL DISTANCE ASSOCIATED WITH ZERO NET TENSILE STRESS IS REPORTED IN THE CMU TABLES THAT APPEARED EARLIER IN THE NOTES.
NEVER ASSUME THE NET TENSILE STRESS IN AN URM WALL IS EQUAL TO THE ALLOWABLE TENSILE STRENGTH ($F_{a}$)

\[-f_a + f_b = F_{a}\]

\[-\frac{P}{A} + \frac{Mc}{I} = F_{b}\]

\[-\frac{P}{bt} + \frac{Pe (t^2)}{(bt^3/12)} = F_{a}\]

\[-1 + \frac{6e}{t} = \frac{F_{a} (bt)}{P}\]

\[\frac{6e}{t} = \frac{F_{a} (bt)}{P} + 1\]

\[e = \frac{F_{a} + bt^2}{6P} + \frac{t}{6}\]

IF THE LOAD IS WITHIN THE KEEP THEN THE NET TENSILE STRESS IS LESS THAN $F_{a}$, AND

\[f_m = \frac{P}{A} + \frac{Mc}{I}\]

\[-f_a + f_o = F_{a}\]

\[-\frac{P}{bt} + \frac{Pe (t^2)}{(bt^3/12)} = \left(\frac{P}{bt}\right) \left[ 1 + \frac{6e}{t} \right]\]

\[< F_{o} = \frac{1}{3} f_m\]

\[\frac{b}{3} + \frac{F_{a} t^2 b}{3P}\]

\[\frac{t^3}{3} + \frac{F_{a} t^2 b}{3P}\]

<table>
<thead>
<tr>
<th>CASE</th>
<th>ECCENTRICITY</th>
<th>AXIAL LOAD</th>
<th>LIMIT CONDITIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SMALL</td>
<td>LARGE</td>
<td>$F_{a}$</td>
</tr>
<tr>
<td>2</td>
<td>LARGE</td>
<td>SMALL</td>
<td>$F_{o}$</td>
</tr>
<tr>
<td>3</td>
<td>LARGE</td>
<td>LARGE</td>
<td>$F_{b}$</td>
</tr>
<tr>
<td>4</td>
<td>SMALL</td>
<td>SMALL</td>
<td>$F_{a}$</td>
</tr>
</tbody>
</table>
Consider an URM wall supported by pin-pin end conditions subjected to eccentric axial loads and a transverse wind load.

For large $P$ and small $W$, the critical section is at the top of the wall and

$$M_{\text{max}} = P_e$$

For small $P$ and large $W$, the critical section is at the midheight where

$$M = \frac{P_e}{2} + \frac{W h^2}{B}$$
Determine masonry compressive strength and the type of mortar required for the following 8" CMU wall laid with face shell bedding.

4.0 k/ft of wall (includes dead and live loads)

8" CMU - Face Shell Bedding

$A = 30 \text{ in}^2/\text{ft of wall}$

$I = 309 \text{ in}^3/\text{ft of wall}$

$s = 81 \text{ in}^3/\text{ft of wall}$

$r = 3.21 \text{ in}$

For this wall

$$\frac{h}{r} = \frac{12(12)}{3.21} = 44.8 < 99$$

By Equation 2-15

$$F_4 = \frac{f_m'}{4} \left[ 1 - \left(\frac{h}{140r}\right)^2 \right]$$

$$= (0.224) f_m'$$

By Equation 2-17

$$F_b = (0.333) f_m'$$

With

$$F_4 = \frac{P}{A} = \frac{4.0 \times (1000)}{30} = 133.33 \text{ PSI}$$
The applied bending stress at the wall mid-height (assumed worst case) is computed based on the following moment:

\[ M = \frac{Pe}{2} + \frac{wL^2}{8} \]

\[ = \frac{4(3)}{2} + \frac{50(12)^2}{1000(8)} \]

\[ = 12.48 \text{ k-ft/ft or wall} \]

Thus,

\[ f_b = \frac{M}{S} = \frac{12.48(1000)}{81} = 154 \text{ PSI} \]

\[ > 133.3 \text{ PSI} \]

Wall is in tension,

\[ f_t = 154 - 133.3 = 20.8 \text{ PSI} \]

\[ < 25.0 \text{ PSI} \rightarrow \text{TYPE M OR C MORTAR} \]

By Spec 2.1.2.3, since we are considering (O+N)

\[ f_t = \frac{20.8}{1.33} = 15.6 \text{ PSI} \]

Check current version of the Ohio Building code to insure use of Spec 2.1.2. Now check unity equation, i.e., code equation 2-10:

\[ \frac{f_b}{f_t} + \frac{f_o}{f_o} = \left[ \frac{133.3}{(0.324) f_m'} + \frac{154}{(0.333) f_m'} \right] \left( \frac{1}{1.33} \right) \]

\[ < 1.0 \]

Solving for \( f_m' \) yields

\[ f_m' = \left( \frac{133.3}{0.224} + \frac{154}{0.333} \right) \left( \frac{1}{1.33} \right) \]

\[ = 792.8 \text{ PSI} \]

Now check buckling,

\[ P < \frac{Pe}{4} \]
\[ 4 \times 1000 \leq \frac{\pi^2 EI}{4 \cdot \frac{h^2}{16}} \left[ 1 - 0.577\left(\frac{c}{h}\right)\right]^3 \]

\[ \leq \frac{\pi^2 \cdot 900 \cdot f_m'(3099)}{4 \cdot 1212^2} \left[ 1 - 0.577\left(\frac{3}{1212}\right)\right]^3 \]

\[ f_m' \geq 1236 \text{ PSI} \quad \text{MINIMUM} \quad f_m' \]

**SPECIFY**

**TYPE M OR S MORTAR**

\[ f_m' = 1500 \text{ PSI} \]
CHECK THE ADEQUACY OF AN UNREINFORCED MASONRY WALL FOR A THREE STORY OFFICE BUILDING. THE MASONRY WALL IS COMPOSED OF 12" CMU BLOCKS Laid IN FACE BRICK BEDDING.

ASSUME

$f_m = 1500$ PSI

3 Bond Beams Per Story

16 Courses of Block Per Story

Wind Load = 70 PSF (Suction)

USE THE FOLLOWING STORY LOADS

- **Roof DL** = 900 LBS/FT OF WALL
- **Roof LL** = 1,500 LBS/FT OF WALL

- **Third Floor DL** = 900 LBS/FT OF WALL
- **Third Floor LL** = 1,000 LBS/FT OF WALL

- **Second Floor DL** = 600 LBS/FT OF WALL
- **Second Floor LL** = 1,800 LBS/FT OF WALL
ESTIMATE WALL LOADS

3 courses of bond beam = 3(133)(\frac{8}{12}) = 266 \text{ lbs/ft of wall}

13 courses of block = 13(8)(\frac{8}{12}) = 540 \text{ lbs/ft of wall}

8" thick concrete slab = (\frac{8}{12})(150)(1) = 100 \text{ lbs/ft of wall}

Miscellaneous (vertical grout, spccl, etc.) = 88 \text{ lbs/ft of wall}

\text{Total} = 1000 \text{ lbs/ft of wall}

---

ESTIMATE PARAPET WALL LOADS

1 course bond beam = 133(8)/12 = 89 \text{ lbs/ft of wall}

2 courses of block = \left[\frac{63(8)}{12}\right]^2 = 84 \text{ lbs/ft of wall}

Miscellaneous

\text{Total} = 127 \text{ lbs/ft of wall}

\text{Assume the bond beams provide enough stiffness such that the walls act as fixed-fixed beams between floors. The moment diagram from the wind loads are:}

\text{Parapet Wall}

\text{Root:} \quad \frac{9WL^2}{8} = \frac{(20)(1)(12)^2(12)}{8} = 270 \text{ lbs-in}

\text{Mid-Height:} \quad \frac{9WL^2}{24} = \frac{(20)(1)(12)^2(12)}{24} = 1440 \text{ lbs-in}

\text{Third Floor:} \quad \frac{9WL^2}{12} = \frac{(20)(1)(12)^2(12)}{12} = 2880 \text{ lbs-in}

\text{Second Floor:} \quad 2880 \text{ lbs-in}

\text{Second Floor:} \quad \frac{9WL^2}{128} = \frac{(20)(1)(12)^2(12)}{128} = 2430 \text{ lbs-in}
The floor and roof slabs will introduce moment via load eccentricity. For this type of wall construction the amount of eccentricity is difficult to determine. So rely on Spec 2.3.3.4.3 and assume a uniform 4" bearing length. The load eccentricity would be

\[ e = \frac{11.625}{2} \]

\[ = 3.8125" \]

The moments induced by the eccentricity of the gravity loads are:

**Roof**

\[ = (900 + 1500)(3.8125) \]

\[ = 9150 \text{ LBS-IN} \]

**Third Floor**

\[ = (900 + 1000)(3.8125) \]

\[ = 7,244 \text{ LBS-IN} \]

**Second Floor**

\[ = (600 + 800)(3.8125) \]

\[ = 9150 \text{ LBS-IN} \]

Both diagrams assume pin connections at the floor section properties are as follows:

<table>
<thead>
<tr>
<th>Stress Computations (Minimum Net Properties)</th>
<th>Stiffness Computations (Average Net Properties)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(A = 36 \text{ in}^2/\text{ft} )</td>
<td>(69 \text{ in}^2/\text{ft} )</td>
</tr>
<tr>
<td>(E = 929 \text{ in}^4/\text{ft} )</td>
<td>(1180 \text{ in}^4/\text{ft} )</td>
</tr>
<tr>
<td>(S = 100 \text{ in}^3/\text{ft} )</td>
<td>(203 \text{ in}^3/\text{ft} )</td>
</tr>
</tbody>
</table>
AT THE ROOF LEVEL

\[ P = 300 + 900 + 1500 = 2700 \text{ lbs/ft} \]
\[ M = 2800 + 9150 = 12,030 \text{ lbs-in./ft} \]

USE AVERAGE NET SECTION PROPERTIES FOR STABILITY CALCULATIONS.

\[ \frac{h}{r} = \left( \frac{1}{4} \right)^{\frac{1}{2}} \left( \frac{1180}{69} \right)^{\frac{1}{2}} = 4.14 \text{ in.} \]

Thus

\[ \frac{h}{r} = \frac{12(12)}{4.14} = 34.82 < 90 \]

By SPEC. 2.2.3.1

\[ F_a = \frac{P_o}{A} \left\{ 1 - \left( \frac{h}{(140)r} \right)^2 \right\} \]
\[ = \frac{1500}{4} \left\{ 1 - \left[ \frac{12(12)}{(140)(4.14)} \right]^2 \right\} \]
\[ = 351 \text{ psi} \]

AND

\[ F_0 = \frac{P_o m}{3} = \frac{1500}{3} = 500 \text{ psi} \]

With

\[ f_o = \frac{M}{S} = \frac{12,030}{160} = 75 \text{ psi} \]

AND

\[ F_a = \frac{P}{A} = \frac{2700}{36} = 75 \text{ psi} \]

THEN

\[ \frac{F_a}{F_o} + \frac{f_o}{F_a} = \frac{75}{351} + \frac{75}{75} = 0.36 < 1.33 \quad \text{OK} \]

Note that since we are considering (D+L+W) TUBA, THE ALLOWABLE STRESSES HAVE BEEN INCREASED BY ONE-THIRD (SPEC. 2.1.2.3)
CHECK END BUCKLING

\[ P_e = \frac{\pi^2 E I_n}{h^2} \left[ 1 - (0.577) \left( \frac{\pi}{h} \right)^3 \right] \]

\[ = \frac{\pi^2 \times 900 \times 1500}{1000 \times (12/4)^2} \left[ 1 - (0.577) \left( \frac{4.46}{4.14} \right)^3 \right] \]

\[ = 41.1 \text{ k/ft} \]

THUS

\[ \frac{P_e}{4} = \frac{41.1}{4} = 10.27 \text{ k/ft} > 2.7 \text{ k/ft} \text{ OK} \]

AT ROOF LEVEL IS OK, CHECK LOWER STORY

\[ P = (300 + 900 + 1500) + (1000 + 900 + 1000) + (1000 + 600 + 1800) \]

\[ = 2700 + 2900 + 3400 = 9000 \text{ lbs/ft} \]

\[ = 9.0 \text{ k/ft} \]

\[ M = 2880 + 9150 \]

\[ = 12,030 \text{ lbs-in/ft} \]

THUS

\[ f_b = \frac{M}{3} = \frac{12,030}{3} = 4010 \text{ psi} \]

AND

\[ f_a = \frac{P}{A} = \frac{9000}{36} = 250 \text{ psi} \]

THEN

\[ \frac{f_a}{f_b} + \frac{f_b}{f_a} = \frac{250}{35.1} + \frac{35.1}{250} = 0.86 < 1.33 \text{ OK} \]

WITH

\[ e = \frac{M}{P} = \frac{12,030}{9.0 \times 1000} = 1.34 \text{ in} \]
\[ P_e = \frac{\pi^2 E_m I_n}{h^2} \left[ 1 - \left(0.577 \left( \frac{L}{r} \right) \right) \right]^3 \]

\[ = \frac{\pi^2 (900)(1500)(1180)}{1000 \left[ \frac{12(12)}{3} \right]^2} \left[ 1 - \left(0.577 \left( \frac{1.34}{4.14} \right) \right) \right]^3 \]

\[ = 407.8 \text{ k/ft} \]

Thus

\[ \frac{P_e}{4} = \frac{407.8}{4} = 102 \text{ k/ft} \geq 9.0 \text{ k/ft} \text{ OK} \]

Finally, at the base of the wall

\[ P = 9000 + 1000 = 10,000 \text{ lbs/ft} \]

\[ = 10 \text{ k/ft} \]

Thus

\[ P_a = \frac{10,000}{36} = 278 \text{ psi} \leq P_a^* = 351 \text{ psi} \text{ OK} \]

With

\[ M = 0 \]

Then

\[ e = 0 \]

And by equation 2.18

\[ P_e = \frac{\pi^2 E_m I_n}{h^2} \]

\[ = \frac{\pi^2 (900)(1500)(1180)}{1000 \left[ \frac{12(12)}{3} \right]^2} \]

\[ = 758 \text{ k/ft} \]

And

\[ \frac{P_e}{4} = \frac{758}{4} = 190 \text{ k/ft} \geq 10.0 \text{ k/ft} \text{ OK} \]
Elastomeric sealants meeting ASTM C920 should be applied in accordance with ASTM C962. Some of the most desirable and successfully used elastomeric sealants for this purpose are the polysulfides, polyurethanes, and silicones. A nonag or gunnable sealant should be used in joints on vertical surfaces. Tooling is essential to force the sealant into the joint and to match the tooled mortar joints in the masonry (Fig. 4-26). Care must be taken to avoid smearing sealant onto the face of the wall.

**Location of Control Joints**

No exact rules can be stated for the location of control joints. Each job must be studied individually to determine where joints can be placed without endangering structural integrity. It has been demonstrated in practice that control joints should be not more than 20 ft. apart in exterior walls with frequent openings. In walls without openings the joint spacing may be a little greater but should never be more than 25 ft. to be most effective. A control joint should be located within 10 or 15 ft. of a corner and preferably one header or stretcher unit from the corner. Flexible ties at the corner should be installed to develop the load carrying capacity of the wall, but allow proper movement.

Control joints should also be located at the following points of weakness or high stress concentrations:

1. At all abrupt changes in wall height.
2. At all changes in wall thickness, such as those at pipe or duct chases and those adjacent to columns or pilasters (Figs. 4-27 and 4-28).
3. Above joints in foundations and floors.
4. Below joints in roofs and floors that bear on the wall.
5. At a distance of not over one-half the allowable joint spacing from bonded intersections or corners.
6. At one or both sides of all door and window openings unless other crack control measures are used, such as joint reinforcement or bond beams.

All large openings in walls should be recognized as natural and desirable joint locations. Although some adjustment in the established joint pattern may be required, it is effective to use vertical sides of wall openings as part of the control joint layout. Under windows the joints usually are in line with the sides of the openings. Above doors and windows the joints must be offset to the end of the lintels. To permit movement, the bearing of at least one end of the lintel should be built to slide (Fig. 4-29). Plastic or bituminous sheets or other suitable material should be used as a slip plate.

Openings less than 6 ft. wide require a control joint along one side only, but openings of more than 6 ft. should have joints along both sides (Fig. 4-30). A control joint between two windows should be avoided since it will not function properly (Fig. 4-31).

To avoid the occurrence of cracks due to differential movement between concrete masonry and structural framing members, such as columns and pilasters, a space should be allowed between the masonry and...
member to allow free movement. One or more control joints should be located at the column or pilaster (Fig. 4-32).

When a concrete masonry wall is reduced in thickness across the face of a column, a control joint should be placed along one or both sides of the column. Thin concrete masonry across the column face should be tied to the column by means of dovetail anchors (Fig. 4-32) or another suitable device.

**Fig. 4-32.** Control joints at columns and pilasters. Joint stabilizing anchors in Fig. 4-24 also are used to attach walls to columns or pilasters while allowing in-plane movement.
Where bond beams are provided only for crack control, control joints should extend through them. If there is a structural reason for a bond beam, a dummy groove or raked joint should be provided to control the location of the anticipated crack.

A concrete masonry or cast-in-place concrete foundation having both sides backfilled does not usually require control joints. However, long concrete masonry basement walls may require control joints, continuous metal ties (joint reinforcement), or reinforcing bars.

Where concrete masonry units are used as a backup for another material with masonry bond, the control joints should extend through the facing. Control joints need not extend through the facing when using flexible metal ties.

Control joints should extend through plaster applied directly to concrete masonry units. Plaster applied on lath that is furred out from the base requires control joints over previous joints in concrete masonry.

The design, detailing, and spacing of control joints should be by mutual agreement of architect and structural engineer. Both parties should consider: availability of units in project area; engineering aspects as to stress concentrations and requirements for concrete masonry; experience with performance of masonry structures; and esthetics. The engineer should explore alternatives to moisture-controlled units not be available in the project area. Alternatives available include reducing length of wall and adjusting/decreasing joint reinforcement spacing requirements.

### Joint Reinforcement

Although concrete masonry walls can be built essentially free of cracks, it is the infrequent crack for which joint reinforcement (Fig. 4-12) is provided. The function of joint reinforcement is not to eliminate cracking in concrete masonry walls but merely to prevent the formation of conspicuous shrinkage cracks. Joint reinforcement does not become effective until the wall begins to crack. After cracking occurs the stresses are transferred to and redistributed by the steel. The result is evenly distributed, very fine cracks that are hardly visible to the naked eye.

The effectiveness of joint reinforcement depends on the type of mortar and the bond between the mortar and the longitudinal wires. The better the bond strength, the more efficient the reinforcement in arresting any cracking. In-service experience has shown that Types M, S, and N mortar should be considered for use with joint reinforcement.

After the joint reinforcement is placed on top of the bare masonry course, the mortar is applied to cover the face shells and joint reinforcement. Minimum mortar cover from the exterior surface to the joint reinforcement should be 3/8 in.; this mortar cover should be 3/8 in. on the interior as shown in Fig. 4-5c.

Prefabricated or job-fabricated corner and T-type joint reinforcement should be used around corners and to anchor abutting walls and partitions (Fig. 4-12). Prefabricated corners and tees are considered superior because they are more accurately formed, fully welded, and easier to install. A 6-in. lapping of side wires at splices is essential. Continuity of the reinforcement must occur so that tensile stress will be transmitted.

The vertical spacing of joint reinforcement is dependent on the spacing of control joints. In addition, joint reinforcement should be located as follows:

1. In the first and second bed joints immediately above and below wall openings. The reinforcement should extend not less than 24 in. past either side of the opening or to the end of the panel, whichever is less.
2. In the first two or three bed joints above floor level, below roof level, and near the top of the wall.

Joint reinforcement need not be located closer to a bond beam than 24 in. It should not extend through control joints unless specifically called for and detailed in the plans.

### Layout of Structural Features

#### Modular Planning

Modular planning is a method of coordinating the dimensions of various building components to simpli-
Empirical Crack Control Criteria and Engineered Crack Control Criteria, present detailed recommendations for control joint spacing. The empirical criteria is based on historical guidelines that have proven successful over many years of experience for a broad geographic distribution. These recommendations are intended to provide the most straightforward guidelines for those cases where detailed properties of the concrete masonry are not known at the time of design.

The engineered criteria has been developed and is based on a Crack Control Coefficient. In general, this rational approach results in more effective crack control, but requires more detailed knowledge of the masonry characteristics.

**EMPIRICAL CRACK CONTROL CRITERIA**

The empirical crack control criteria has been developed based on successful, historical performance over many years in various geographic conditions. Local experience may justify an adjustment to control joint spacing over those presented in Table 1.

To illustrate these criteria, consider a 20 ft (6.10 m) tall warehouse with walls 100 ft (30.48 m) long. Using Table 1 results in control joints being spaced every 25 ft (7.62 m). In this example, the maximum spacing of 25 ft (7.62 m) governs over the maximum length to height ratio of 1 1/4 times 20 ft (6.10 m) which equals 30 ft (9.14 m). For masonry parapet height in determining the length to height ratio, consider the parapet as part of the masonry wall below if it is connected by masonry materials such as a bond beam or a soap unit.

Footnotes to Table 1 provide further information on applying the recommendations. These control joint spacings have been developed based on the use of horizontal reinforcement to keep unplanned cracks closed. The minimum area of reinforcement given, 0.025 in.$^2$/ft (52.9 mm$^2$/m) of height, translates to horizontal joint reinforcement W1.7 (9 gage) (MW 11) or larger at 16 in. (406 mm) oc, one #4 (#14) bar at 48 in. (1219 mm) oc, one #6 (#16) bar at 96 in. (2348 mm) o.c., or one #8 (#18) bar (or larger) at 144 in. (3658 mm) o.c.

**Table 1 — Recommended Control Joint Spacing for Above-Grade Exposed Concrete Masonry Walls**

<table>
<thead>
<tr>
<th>Length to Height Ratio</th>
<th>Minimum Joint Spacing in Ft (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4</td>
<td>25 (7.62)</td>
</tr>
<tr>
<td>1/3</td>
<td>20 (6.10)</td>
</tr>
<tr>
<td>2/3</td>
<td>15 (4.57)</td>
</tr>
</tbody>
</table>

1. Table values are based on the use of horizontal reinforcement having an equivalent area of not less than 0.025 in.$^2$/ft (52.9 mm$^2$/m) of height to keep unplanned cracks closed.
2. Criteria applies to both Type I and Type II concrete masonry units.
3. This criteria is based on experience over a wide geographical area. Control joint spacing should be adjusted up or down where local experience justifies but no further than 25 ft (7.62 m).