b. In flanged sections, the special boundary element shall include the effective flange width in compression and shall extend at least 12 in. into the web.

c. Special boundary element transverse reinforcement at the wall base shall extend into the support a minimum of the development length of the largest longitudinal reinforcement in the boundary element unless the special boundary element terminates on a footing or mat, where special boundary element transverse reinforcement shall extend at least 12 in. into the footing or mat.

d. Horizontal shear reinforcement in the wall web shall be anchored to develop the specified yield strength, f_y, within the confined core of the boundary element.

6.8 DESIGN EXAMPLE – SHEAR WALL

This section provides a detailed design example based on strength design requirements of MSJC Code Section 3.3.6.

A shear wall computer program is suggested to estimate the location of the neutral axis and determine stresses, loads and moments. The Concrete Masonry Association of California and Nevada has a program available.

Tensile bond strength and modulus of rupture values for the unreinforced masonry is shown in Table 6.2.

EXAMPLE 6-I Shear Wall Design by Strength Methods; Vertical Load, Overturning and Shear.

A nominal 8 in. solid grouted concrete masonry shear wall carries a dead load of 4 kips/ft, live load of 1.5 kips/ft and a lateral force of 45 kips due to wind. (SDC D, but wind governs) f'_m = 1,500 psi, f_y = 60,000 psi.

Determine the required tension and shear steel using factored loads and strength design procedures developed above.

Solution 6-I

Determine factored loads.

$U = 0.9D + 1.6W + 1.6H$ (H = 0 in this case)
(IBC Section 1605.2.1, Equation 16-6)

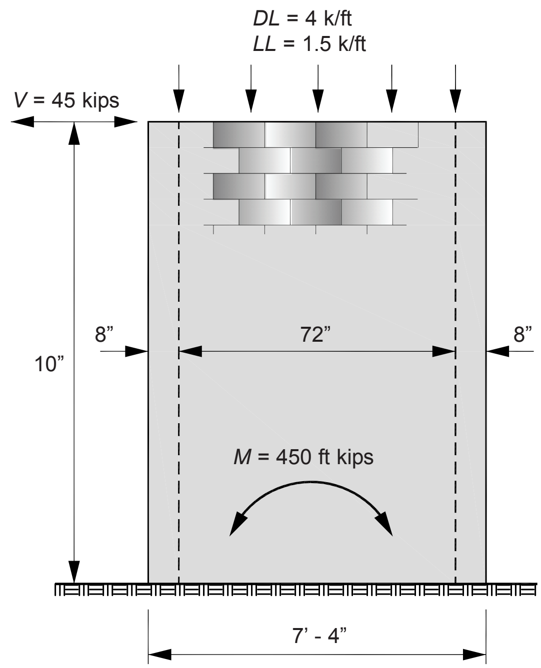


FIGURE 6.27 Load condition example 6-I.

$$P_u = 0.9D = 0.9\ (4)\ 7.33$$
$$= 26.4 \text{ kips}$$

Factored overturning moment

$$M = 1.6W = 1.6\ (45)\ 10$$
$$= 720 \text{ ft kips}$$

Solve for length of stress block a

Determine the constants for the coefficients for the quadratic equations

$$\mathbf{a} = 0.4f'_m t$$
$$= 0.4\ (1.5)\ 7.63 = 4.58$$

$$\mathbf{b} = -0.80f'_m td$$
$$= -0.80\ (1.5)\ 7.63\ (80) = -732.5$$

$$\mathbf{c} = P\left(\frac{l}{2} - d_1\right) + M$$
$$= 26.4\left(\frac{88}{2} - 8\right) + 720\,(12)$$
$$= 9{,}590$$

Solve for length of stress block a

$$a = \frac{-\mathbf{b} - \sqrt{\mathbf{b}^2 - 4\mathbf{ac}}}{2\mathbf{a}}$$

$$a = \frac{-(-732.5) - \sqrt{(-732.5)^2 - 4(4.58)9{,}590}}{2(4.58)}$$
$$= 14.4 \text{ in.}$$

Compression forces

$C = 0.80 f'_m a t$

$= 0.80(1.5)\ (14.4)\ (7.63)$

$= 131.7$ kips

Tension force

$T = C - P$

$= 131.7 - 26.4$

$= 105.3$ kips

Area of overturning tension steel for in-plane flexure

$\phi = 0.9$

$$A_s = \frac{T}{\phi f_y} = \frac{105.3}{0.9\,(60)}$$

$= 1.95$ sq in.

Use 2 - #9 bars each side ($A_s = 2.0$ sq in.)

Check strain requirements of MSJC Code Section 3.3.3.5. Since shear walls of Seismic Design Category D must be designed as special reinforced masonry shear walls, MSJC Code Section 3.3.3.5.3 applies and the strain in the extreme fibers must be 4 times the yield strain. Based on the neutral axis, the strain in the extreme fiber can be computed using similar triangles:

$$\varepsilon_s = (d - c)\frac{\varepsilon_m}{c} = \left(80 - \frac{14.4}{0.8}\right)\frac{(0.0025)}{\left(\frac{14.4}{0.8}\right)}$$

$= 0.008611$

$$\frac{\varepsilon_s}{\varepsilon_y} = \frac{0.008611}{0.002069}$$

$= 4.16 > 4$ OK

Satisfies MSJC Code Section 3.3.3.5.3.

Shear design

$V_n = V_m + V_s$ (MSJC Code Eq 3-18)

Consider various options in the design.

Factored lateral load

$V_u = 1.6\ (45) = 72.0$ kips

$\phi = 0.80$ (MSJC Code Section 3.1.4.3)

$$V_n = \frac{V_u}{\phi} = \frac{72}{0.8} = 90 \text{ kips}$$

$d_v = 7$ ft 4 in. - 8 in. = 80 in.

Option 1 - Consider nominal masonry shear strength

$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + 0.25 P_u$$

(MSJC Code Eq 3-21)

$$= \left[4.0 - 1.75\left(\frac{720(12)}{72(80)}\right)\right](7.63)(88)\frac{\sqrt{1500}}{1000} + 0.25(26.4)$$

$= 42.4$ kips

Therefore V_s required is

$V_s = V_n - V_m = 90 - 42.36 = 47.6$ kips

$$V_s = 0.5\left(\frac{A_v}{s}\right) f_y d_v$$ (MSJC Code Eq 3-22)

$$A_v = \frac{2 s V_s}{f_y d_v}$$

$$= \frac{2s(47.6)}{60(80)} = 0.0198 s \text{ in.}^2$$

(area for spacing s)

Try 16 in. spacing.

$A_s = 0.0198(16) = 0.3168$ in.^{2}
(#5 bars, $A_s = 0.31$ in.^{2}, 2% overstress)

Consider minimum steel $0.0013bt$.

$A_s = 0.0013\ (16)\ (7.63)$

$= 0.16$ in.$^2 < 0.31$ in.^{2}

Use #5 at 16 in. o.c. in horizontal bond beams.

Option 2 - Assume wall is in critical hinge area, all shear to be resisted by reinforcing steel.

$$V_s = 0.5\left(\frac{A_v}{s}\right) f_y d_v$$ (MSJC Code Eq 3-22)

$$A_v = \frac{2 s V_s}{f_y d_v}$$

$$= \frac{2s(90)}{60(80)} = 0.0375s \text{ in.}^2 \quad \text{(area for spacing } s\text{)}$$

Try 16 in. spacing.

A_s = 0.0375 (16) = 0.60 in.^{2} (#7 bars @ 16 in.)

Consider steel in primary direction 0.0013*bt*.

A_s = 0.0013 (16) (7.63)

= 0.16 in.^{2} < 0.60 in.^{2}

Use #7 at 16 in. o.c. in horizontal bond beams

Vertical (transverse) shear reinforcement between OTM jamb steel:

MSJC Code Section 3.3.6.2 requires vertical reinforcement of at least half the horizontal reinforcement. Therefore, vertical reinforcement is at least 0.5(0.60) = 0.30 in.^{2}/16 in. (#5 bar A_s = 0.31).

Consider minimum steel 0.0007*bt*.

A_s = 0.0007 (16) (7.63)

= 0.85 in.^{2} < 0.31 in.^{2}

Use #5 bars @ 16 in. spacing between the #9 bars

A_s = 4(1.0) + 4(0.31) = 5.24 sq in.

However, if this same wall is located in SDC D, then MSJC Code Section 1.14.6.3 requires a minimum of 0.002 times the gross section for combined vertical and horizontal reinforcement:

A_s = 0.002*bt*

Considering horizontal and vertical ratio requirements:

(0.002 - 0.0007)*bt* = 0.0013*bt* = 0.0013 (88) (7.63) = 0.87 in.^{2}

0.0007*bt* = 0.0007 (88) (7.63) = 0.47 in.^{2} [which is less than the area of 0.85 in.^{2}]

If the shear wall is required to have plastic hinge considerations, then the masonry component should not be considered as part of the shear strength, V_m.

Additional examples are provided with respect to in-plane shear for shear walls and the use of the criteria based upon *M/Vd* as per MSJC Code Section 3.3.6.

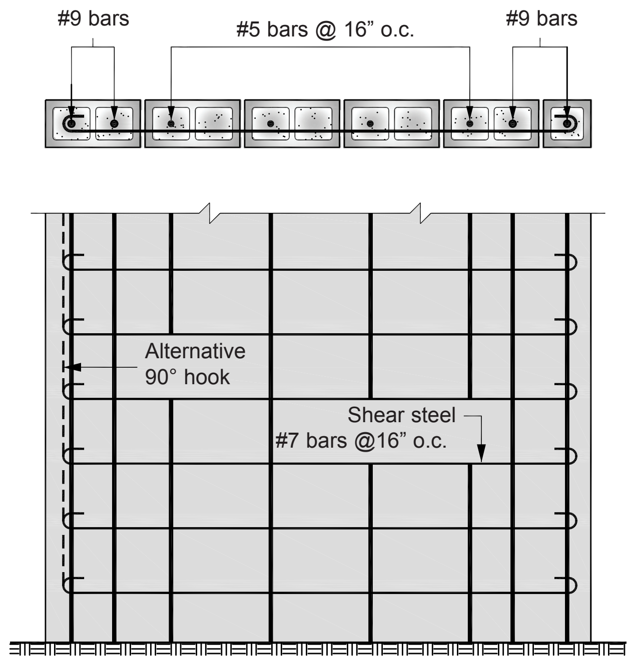


FIGURE 6.28 Layout of final design steel in shear wall (considering plastic hinge criteria).

EXAMPLE 6-J Strength Design of a Shear Wall.

Determine the reinforcing steel for the overturning moment, axial load and shear force on the solid grouted 8 in. concrete masonry wall shown. Type S portland cement-lime mortar is specified. Verify that the wall meets the requirements of MSJC Code Section 3.3.6.

V = 110 kips (Earthquake Load)

P = 200 kips (Dead Load)

M = 1100 ft kips (Earthquake Load)

Wall properties

8 in. CMU = 7.625 in. actual

Given f'_m = 3,000 psi; f_y = 60,000 psi

$E_m = 900f'_m$ = 2,700,000 psi; n = 10.7

From Table SD-24:

Modulus of rupture = 163 psi

Maximum usable masonry strain, e_{mu} = 0.0025 in./in.

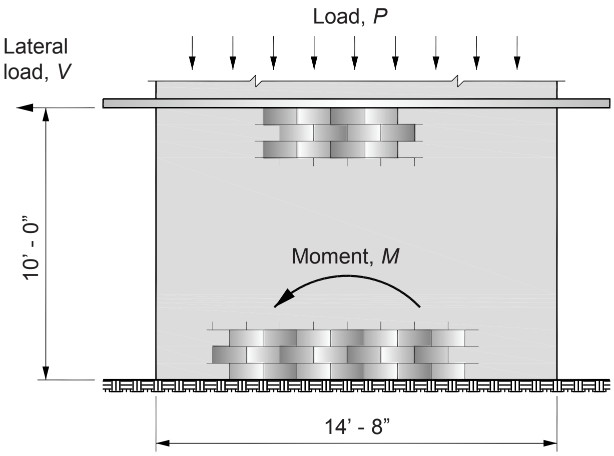


FIGURE 6.29 Masonry shear wall subjected to combined loading and moment.

Load factors (other factors for snow, rain, wind and/or contributory area could apply) from sample combinations:

$U = 1.4D$

$U = 1.2D + 1.6L$

$U = 0.9D + (1.0E \text{ or } 1.6W)$ (Assume that E controls over W for this example)

Strength reduction factors, ϕ

ϕ = 0.9 Axial load and moment (MSJC Code Section 3.1.4.1)

ϕ = 0.80 Shear (MSJC Code Section 3.1.4.3)

Estimate vertical steel requirement for overturning moment (neglecting axial force for this trial determination of an area of steel). For seismic strength design the preferred distribution of steel is uniform distribution at a 16 in. or 24 in. spacing. Thus, a distribution of reinforcement represented in Figure 6.30 is preferred.

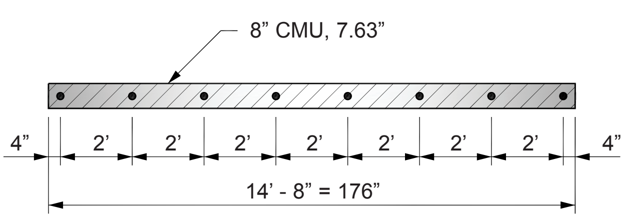


FIGURE 6.30 Shear wall reinforcement locations.

To estimate the reinforcement required compute the area of the bar size required from:

$$A_s = \frac{M}{\left(\sum d_i\right) f_y}$$

$$= \frac{1100(1.0)(12)}{(168 + 144 + 120 + 96 + 72 + 48 + 24)(60)}$$

$= 0.33 \text{ in.}^2$ (8 - #5 bars, A_s = 2.62 sq in.)

This value is close to the size of a #5 bar (0.31 in.^{2}), but since the combined stresses including axial loading requires the next larger size, use #6 bars (0.44 in.^{2}).

Try 8 - # 6 bars (A_s = 3.52 sq in. > 2.62 sq in.)

Analyze the shear wall by:

1. Plotting the interaction diagram for the wall.
2. Determining the cracking moment, $M_n \geq M_{cr}$.
3. Checking loading conditions for vertical load and moment.
4. Checking the requirements for boundary elements and confinement.
5. Determining the shear reinforcement.
6. Comparing the design to wall designed by the allowable stress method.

Solution 6-J

1. Plot interaction diagram

Where

P_o = Nominal axial strength

P_u = ϕ times the nominal axial strength or the factored axial load on the wall

M_n = Nominal moment strength

M_u = ϕ times the nominal moment strength or the factored moment on the wall

P_b = Balanced axial strength

P_{bu} = ϕ times the balanced axial strength

M_b = Balanced moment strength

M_{bu} = ϕ times the balanced moment strength

a) Nominal axial load P_o

$P_o = 0.80f'_m(A_n - A_s) + f_y A_s$

$A_n = 7.625\ (176) = 1{,}342 \text{ in.}^2$

A_s = 8 (0.44) = 3.52 in.^{2}

P_o = 0.80 (3) (1,342 - 3.52) + 60 (3.52)

= 3,212 + 211 = 3,424 kips

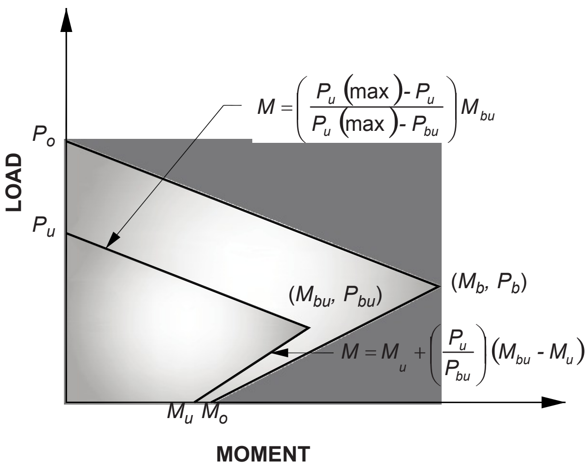


FIGURE 6.31 Simplified generic interaction diagram.

Nominal maximum axial compressive strength for the upper limit of axial force:

Where P_o is the theoretical upper limit; however, the upper permissive axial force is governed by MSJC Code Equations 3-16 or 3-17 depending upon the *h/r* ratio of greater or less than 99.

Thus, from Table GN-8b, horizontal section properties for solid grouted masonry spanning vertically, the radius of gyration, *r*, is 2.21 in., so the *h/r* ratio is determined by:

$$\frac{h}{r} = \frac{10\,(12)}{2.21} = 54.3 \leq 99 \text{ therefore MSJC Code}$$

Equation 3-16 for $h/r \leq 99$ applies

$$P_n = 0.80\left[0.80 f'_m \left(A_n - A_s\right) + f_y A_s\right]\left[1 - \left(\frac{h}{140r}\right)^2\right]$$

(MSJC Code Eq 3-16)

P_n = 0.80[0.80(3)(1,342-3.52) + 60(3.52)]

$$\left[1 - \left(\frac{10(12)}{140(2.21)}\right)^2\right]$$

= 2,324 kips

Therefore, the P_n obtained from the above equation used with $P_u = \phi P_n$ provides the upper limit to the final axial load capacity for the interaction curves.

$P_u = \phi P_n$ = 0.90 (2,324) = 2,091 kips

b) Factored axial load, P_u

P_u = 1.4 (200) = 280 k

Check $P_u \leq \phi P_n$ (conservatively check for bearing with ϕ = 0.6 in MSJC Code Section 3.1.4.5)

280 = 0.6 (2,324)

280 $\leq$ 1,394 kips O.K.

c) Nominal moment strength, M_n

Solve for location of the neutral axis (NA) so that sum of vertical forces equals zero.

Assume location for NA; *c* = 10 in., use trials based upon strain compatibility and equilibrium.

Use maximum allowable CMU strain = 0.0025

Solution by iteration.

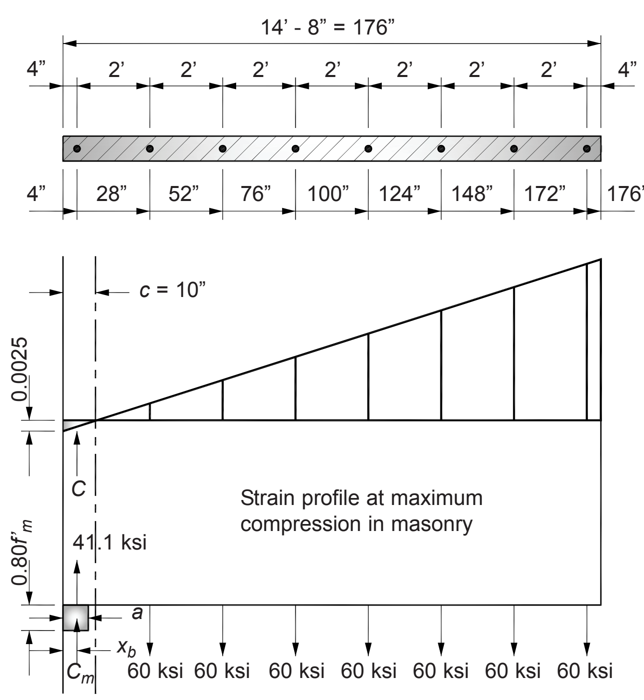


FIGURE 6.32 Steel location, strain condition and force equilibrium diagrams.

Take the sum of the moments about the extreme compression fiber at the end of wall.

a = Depth of compression stress block

= 0.80 c = 0.80 (10) = 8.0 in.

x_b = 4.0 < 8.0 in.

Tension force

$T = A_s f_y$

= 0.44 (7) (60)

= 184.8 kips

Compression force

$$f_s = E_s\left(\frac{c-4}{c}\right)\varepsilon_{mu} \text{ (by proportion)}$$

$$= 29{,}000\left(\frac{10-4}{10}\right)0.0025$$

= 43.5 ksi

$C = A_s f_s + 0.80f'_m ba$

= 0.44 (43.5 - 0.80 (3)) + 0.80 (3) (7.625) (8.0)

= 18.1 + 146.4 = 164.5 kips

T - C = 184.8 - 164.5 = 20.3 kips (need more C force, therefore, try c = 11 in.)

a = 0.80 (11) = 8.8 in.

$$f_s = 29{,}000\left(\frac{11-4}{11}\right)0.0025 = 46.14 \text{ ksi}$$

$C = A_s f_s + 0.85f'_m ba$

= 0.44 (46.14 - 0.80(3)) + 0.80 (3) (7.625) (8.8)

= 19.2 + 161.0 = 180.2 kips (just shy by 4.6 kips, try c = 11.3 in.)

a = 0.80(11.3) = 9.04 in.

$$f_s = 29{,}000\left(\frac{11.3-4}{11.3}\right)0.0025 = 46.84 \text{ ksi}$$

$C = A_s f_s + 0.85f'_m ba$

= 0.44 (46.84 - 0.80(3)) + 0.80 (3) (7.625) (9.04)

= 19.6 + 165.4 = 185.0 kips - reasonably close

Thus, use c = 11.3 in.

Nominal bending moment, M_n

Sum of moments about left edge of wall

M_n = T (moment arm) - C (moment arm)

= $A_s f_y$ (moment arm) - [$0.80f'_m ba$ (moment arm) + $A_s f_s$ (moment arm)]

= 0.44[(60)(28 + 52 + 76 + 100 + 124 + 148 + 172)]
- [0.80(3)(7.625)(9.04)(4.52)]
- 0.44[46.84 - 0.8(3)](4)

= 18,480 - 747.8 - 78.2

= 17,654 in. kips = 1,471 ft kips

d) Design bending moment, M_u

$M_u = \phi M_n$ = 0.90 (1,471)

= 1,324 ft kips

e) Nominal balanced design axial load, P_b

Compression capacity, $C_m = 0.80f'_m ba_b$

Where balanced stress block, $a_b = 0.80c$

$$c_b = \left(\frac{e_{mu}}{e_{mu} + \dfrac{f_y}{E_s}}\right)d$$

$$= \left(\frac{0.0025}{0.0025 + \dfrac{60{,}000}{29{,}000{,}000}}\right)d = 0.547d$$

c_b = 0.547 (172) = 94.1 in. (neutral axis for balanced design)

$a_b = 0.80c_b = 0.80(0.547)d$

= 0.438d

a_b = 0.438(172)

= 75.3 in., depth of compression stress block

$$x_b = \frac{176}{2} - \frac{75.3}{2} = 50.35 \text{ in.}$$

Tension force

$T = A_s f_s$

$= 0.44(4.5 + 23 + 41.5 + 60)$

$= 0.44\ (129) = 57$ kips

Compression force

$C = A_s f_s + 0.80f'_m ba_b$

$= 0.44\ [(14.0 + 32.4 + 50.9 + 60) - 4\ (0.80)\ (3)]$
$+ 0.80(3)(7.625)(75.3)$

$= 65 + 1{,}378 = 1{,}443$ kips

Sum of vertical forces

$P_b = C - T$

$= 1{,}443 - 57$

$= 1{,}386$ kips

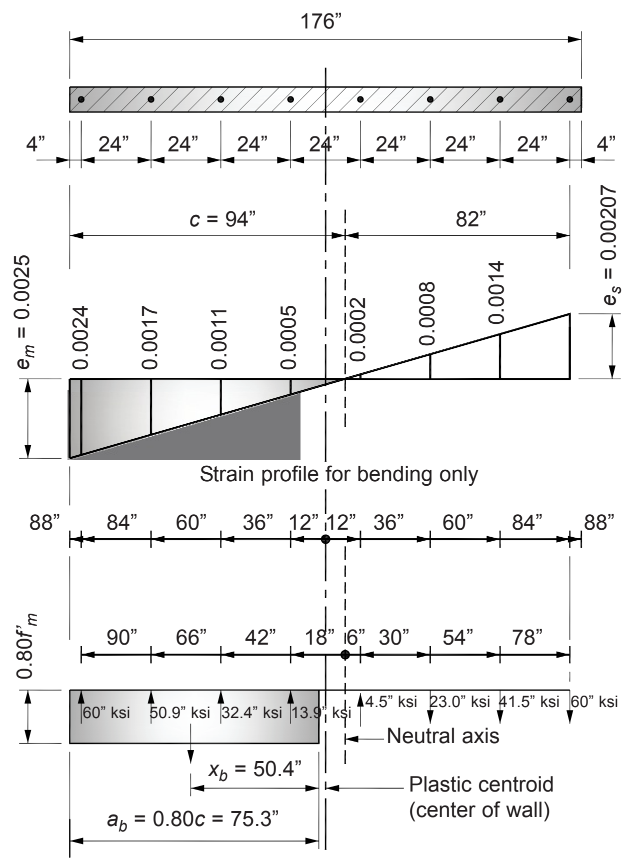


FIGURE 6.33 Balanced design load condition.

f) Design balanced axial load, P_{bu}

$P_{bu} = \phi P_b$

$= 0.9\ (1386) = 1247$ kips

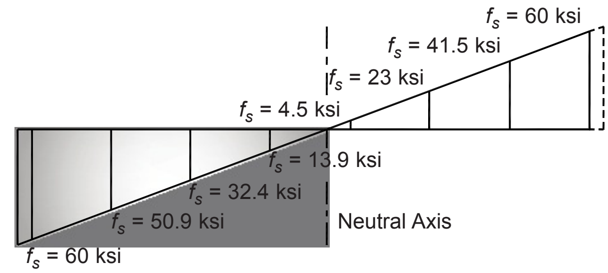


FIGURE 6.34 Stress distribution.

g) Nominal balanced design moment strength, M_b

Take moments about plastic centroid which is the center of the wall as it is symmetrical for masonry and steel

$$M_b = A_s f_s\ (\text{moment arm}) + 0.80f'_m a_b b x_b$$

$$= 0.44\ [60(84) + 41.5\ (60) + 23.0\ (36) + 4.5(12) + 13.9(12) + 32.4(36) + 50.9(60) + 60(84)] + 0.80(3)(75.3)(7.625)(50.35)$$

$$= 7{,}650 + 69{,}380$$

$$= 77{,}030 \text{ in. kips} = 6{,}419 \text{ ft kips}$$

h) Design balanced moment strength, M_{bu}

$$M_{bu} = \phi M_b$$

$$= 0.9(6{,}419) = 5{,}777 \text{ ft kips}$$

i) Plot the interaction diagram

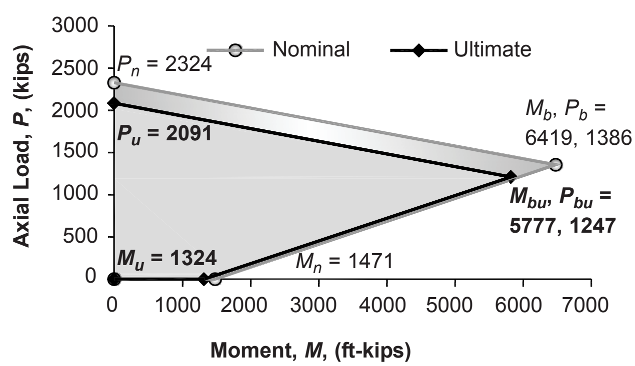


FIGURE 6.35 Interaction diagram for wall, Example 6-J for the assumed reinforcing steel.

2. Cracking moment, M_{cr}

Using gross section properties and linear elastic theory:

$$f_r = \frac{M_{cr}}{S} - \frac{P}{A}$$

Where

A = area of cross-section, bl

= 7.625 (176) = 1,342 sq in.

S = section modulus = $\dfrac{bl^2}{6}$

$$= \frac{7.625(176)^2}{6} = 39{,}365 \text{ in.}^3$$

from MSJC Code Table 3.1.8.2.1, f_r = 163 psi

P = dead load = 200 kips

$$M_{cr} = S\left(\frac{P}{A} + f_r\right)$$

$$= 39{,}365\left(\frac{200{,}000}{1{,}342} + 163\right)\frac{1}{1{,}000}$$

= 12,283 in. kips = 1,024 ft kips

3. Analyze two loading conditions for combined loading, vertical load and moment

a) The load condition for dead load is:

$U = 1.4D$

From Table GN-3a for a fully grouted normal weight 8 in. concrete masonry wall, the wall dead load is 84 psf. The ultimate axial load is:

$P_u = 1.4P_{DL}$

$P_{DL} = P + hl$ (wall weight per sq ft surface area)

$$P_u = 1.4\left[200 + \frac{10(14.67)(84)}{1{,}000}\right]$$

= 297.3 kips < P_{bu}

b) The load condition for dead load plus seismic load is:

$U = 0.9D + 1.0E$

$P_u = 0.9\ P_{DL}$

$$P_u = 0.9\left[200 + \frac{10(14.67)(84)}{1{,}000}\right]$$

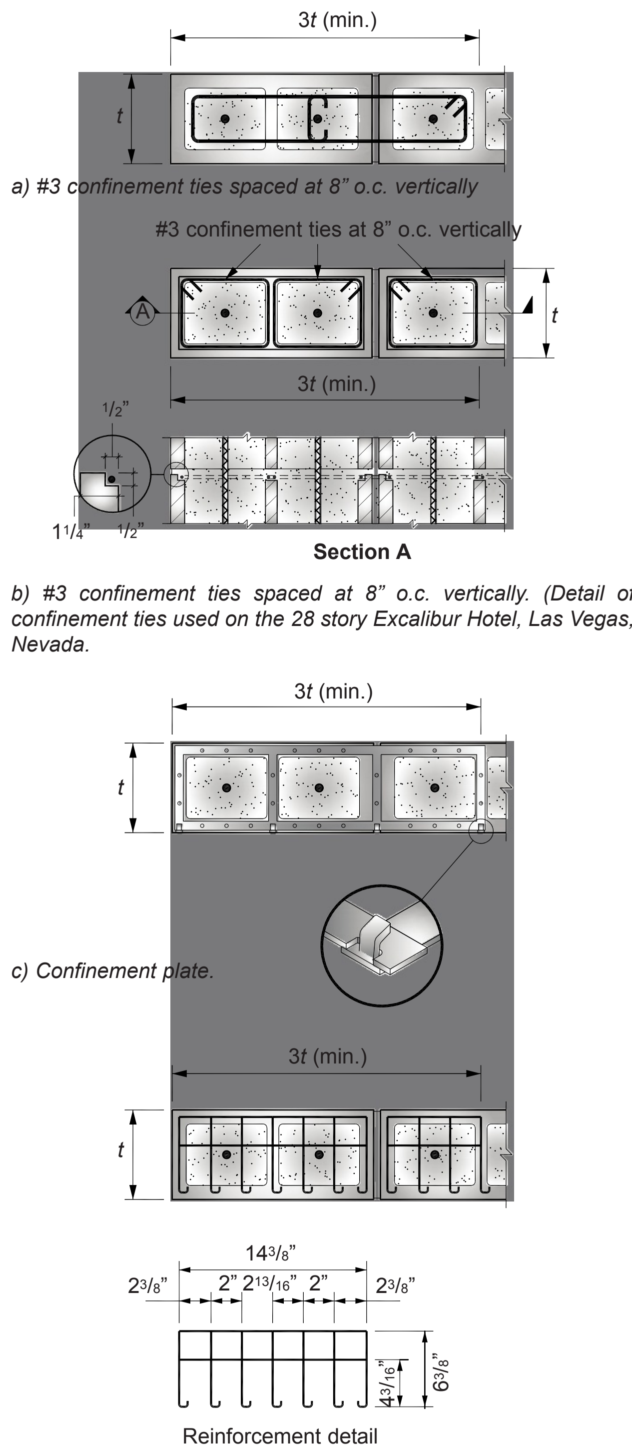
= 191.1 kips

M_u = 1.0 (1,100)

= 1,100 ft kips and the M_n is greater than the M_{cr} (Controlling load condition)

4. Check requirements for boundary and confinement condition.

For this example assume that confinement of vertical steel is not required, but the designer may specify confinement devices in boundary elements for 32 in. on each side at 8 in. vertical spacing.



a) #3 confinement ties spaced at 8" o.c. vertically

b) #3 confinement ties spaced at 8" o.c. vertically. (Detail of confinement ties used on the 28 story Excalibur Hotel, Las Vegas, Nevada.

c) Confinement plate.

d) Open wire mesh bed joint reinforcement.

FIGURE 6.36 Confinement devices for masonry boundary members.

5. Shear Design

a) Shear requirement from controlling load condition

$U = 0.9D + 1.0E \qquad V_u = 1.0\ V_{service}$

$= 1.0\ (110) = 110$ kips

b) Shear strength of wall is determined by:

$V_n = V_m + V_s$ (MSJC Code Eq 3-18)

Shear strength of masonry only:

$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + 0.25P_u$$

(MSJC Code Eq 3-21)

where in the above equation the term $M_u/V_u d_v$ need not be taken greater than 1.0

$d_v = 172$ in.

$M_u = 1{,}100$ ft kips and $V_u = 110$ kips

$$\frac{M_u}{V_u d_v} = \frac{1{,}100(12)}{110(172)} = 0.698$$

$A_n = bl = 7.625(176) = 1{,}342$ in.^{2}

From Table SD-26 and Diagram SD-26,

for $\frac{M_u}{V_u d_v} = 0.698$ and $f'_m = 3{,}000$ psi

$v_m = 152$ psi

$V_m = v_m A_n + 0.25P_u$

Where:

$$v_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right]\sqrt{f'_m}$$

$V_m = 152(1342) + 0.25(191.1)$

$= 204{,}032$ lbs

$= 204$ kips > 110 kips

From $C_d = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right]$

or $C_d = [4 - 1.75(0.698)] = 2.78$

$$V_m = \left[(C_d)(A_n)\sqrt{f'_m}\right] + 0.25P_u$$

$$V_m = 2.78(1{,}342)\sqrt{3{,}000} + 0.25(191{,}100)$$

$= 204{,}342 + 47{,}775 = 252{,}117$ lbs (or 252.1 kips)

$\phi V_m = 0.80(2252.1) = 201.7$ kips $> V_u = 110$ kips

Check design strength requirement of MSJC Code Section 3.1.3

The design shear strength shall exceed the shear corresponding to 125% of the nominal flexural strength, in order to provide an overstrength factor for the critical shear capacity of the wall over the flexural capacity of the wall during a seismic event.

$$\phi V_n \geq 1.25 V_{Mn} = 1.25\left(\frac{M_n}{h}\right) = 1.25\left(\frac{1{,}471}{10}\right)$$

$= 183.9$ kips < 201.7 kips OK

Note that the V_n computed from the nominal flexural strength need not exceed 2.5 times the required shear strength, such that:

$2.5\phi V_u > \phi V_n > 1.25 V_{Mn}$

Shear reinforcement is not required, except for the nominal prescriptive reinforcement required by MSJC Code Section 1.14 depending upon shear wall type.

6.9 WALL FRAMES

6.9.1 GENERAL

Masonry walls are normally considered solid elements with few openings.

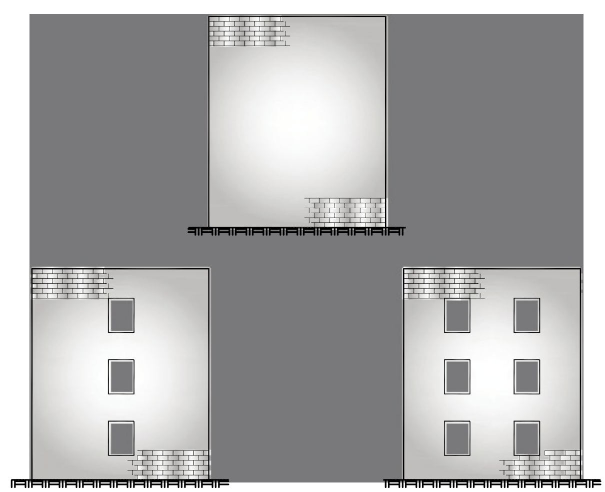


FIGURE 6.37 Shear walls with few small openings.