Design Example 4
Masonry Shear Wall Building

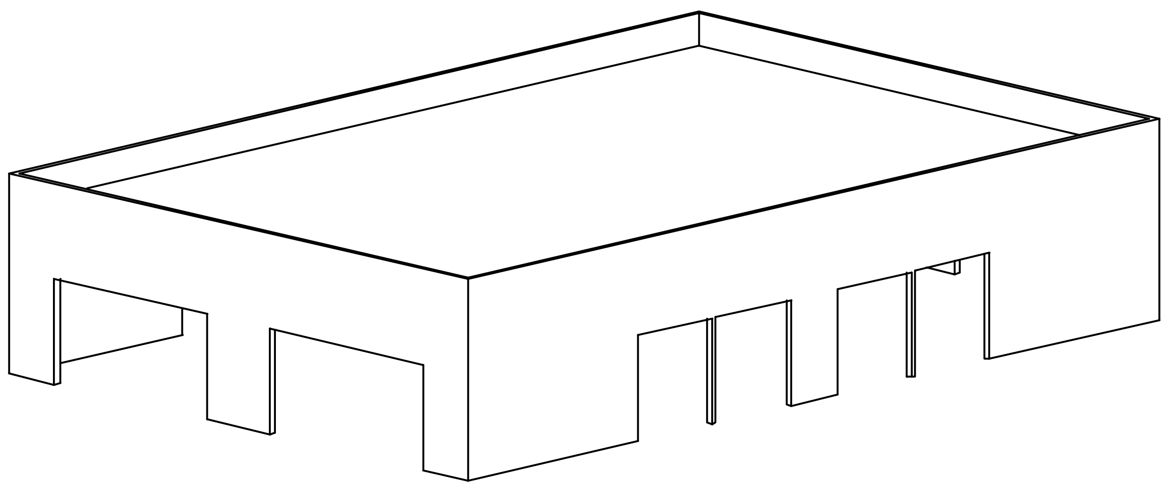

Figure 4-1. Schematic CMU building elevation

Overview

Reinforced concrete block masonry is frequently used in one-story and lowrise construction, particularly for residential, retail, light commercial, and institutional buildings. This type of construction has generally had a good earthquake performance record. However, during the 1994 Northridge earthquake, some one-story buildings with concrete masonry unit (CMU) walls and panelized wood roofs experienced wall-roof separations similar to that experienced by many tilt-up buildings.

This building in this Design Example 4 is typical of one-story masonry buildings with wood framed roofs. The building is characterized as a heavy wall and flexible roof diaphragm "box building." The masonry building for this example is shown schematically in Figure 4-1. Floor and roof plans are given in Figure 4-2 and 4-3, respectively. The building is a one-story bearing wall building with CMU shear walls. Roof construction consists of a plywood diaphragm over wood framing. An elevation of the building on line A is shown in Figure 4-4. A CMU wall section is shown in Figure 4-5, and a plan view of an 8'-0" CMU wall/pier is shown in Figure 4-6.

The design example illustrates the strength design approach to CMU wall design for both in-plane and out-of-plane seismic forces.

Outline

This example will illustrate the following parts of the design process.

1. **Design base shear coefficient.**
2. **Base shear in the transverse direction.**
3. **Shear in wall on line A.**
4. **Design 8'-0" shear wall on line A for out-of-plane seismic forces.**
5. **Design 8'-0" shear wall on line A for in-plane seismic forces.**
6. **Design 8'-0" shear wall on line A for axial and in-plane bending forces.**
7. **Deflection of shear wall on line A.**
8. **Requirements for shear wall boundary elements.**
9. **Wall-roof out-of-plane anchorage for lines 1 and 3.**
10. **Chord design.**

Given Information

Roof weights:

Roofing+ one re-roof	7.5 psf
½" plywood	1.5
Roof framing	4.5
Mech./elec.	1.5
Insulation	1.5
Total dead load	17.0 psf
Roof live load	20.0 psf

Exterior 8-inch CMU walls:

75 psf (fully grouted, light-weight masonry)

$f'_m = 2{,}500$ psi

$f_y = 60{,}000$ psi

Seismic and site data:

$Z = 0.4$ (Seismic Zone 4) Table 16-I

$I = 1.0$ (standard occupancy) Table 16-K

Seismic source type = A

Distance to seismic source = 5 km

Soil profile type $= S_D$

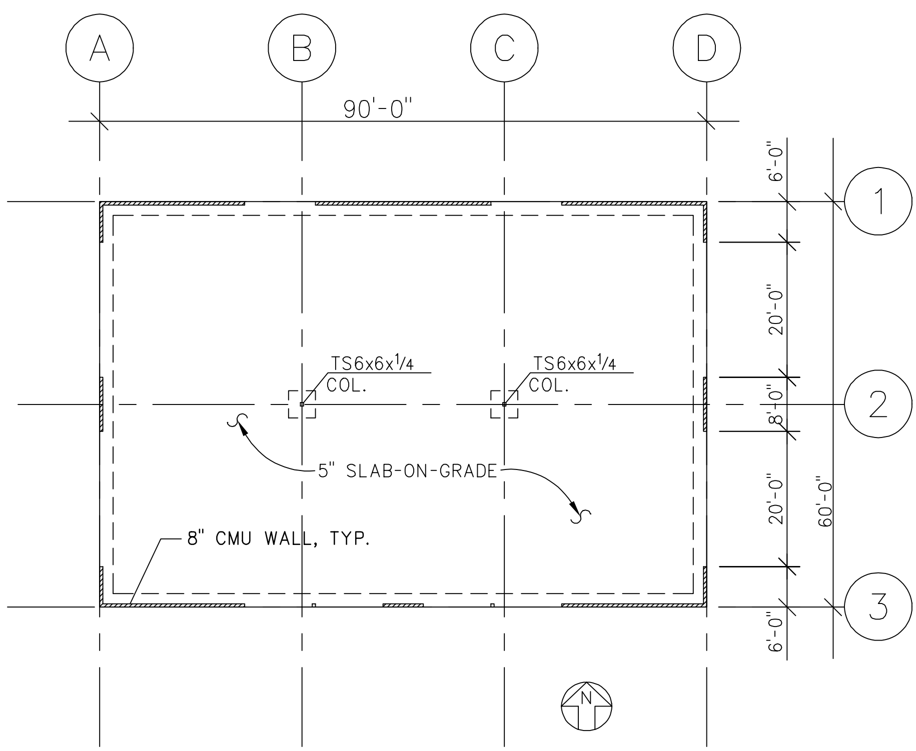

Figure 4-2. Floor plan

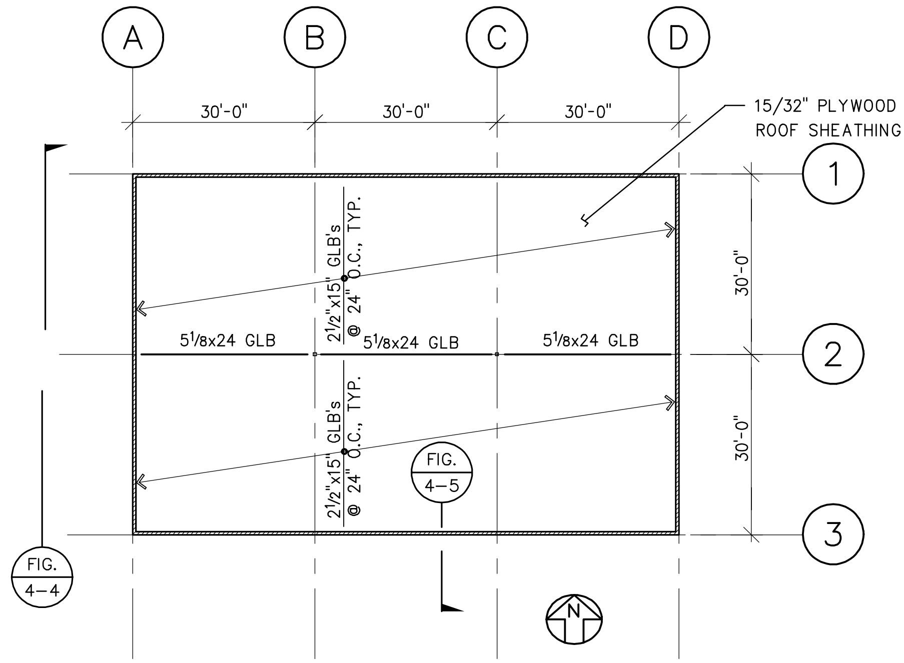

Figure 4-3. Roof plan

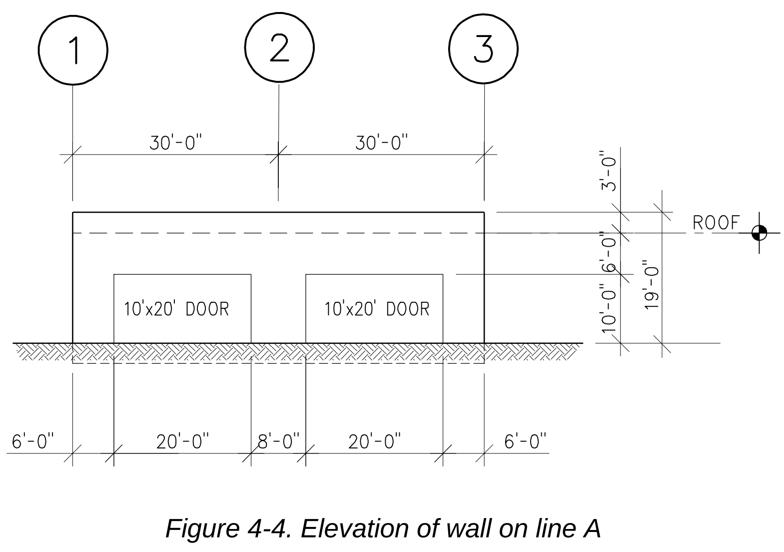

Figure 4-4. Elevation of wall on line A

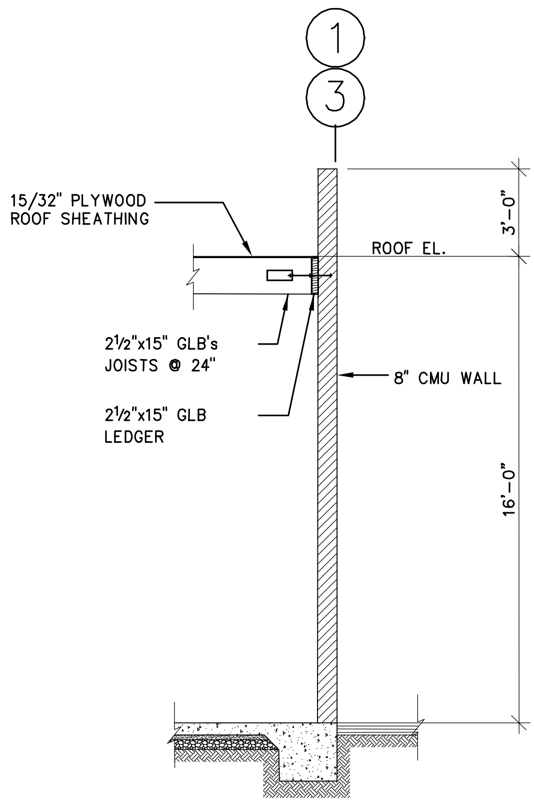

Figure 4-5. Section through CMU wall along lines 1 and 3

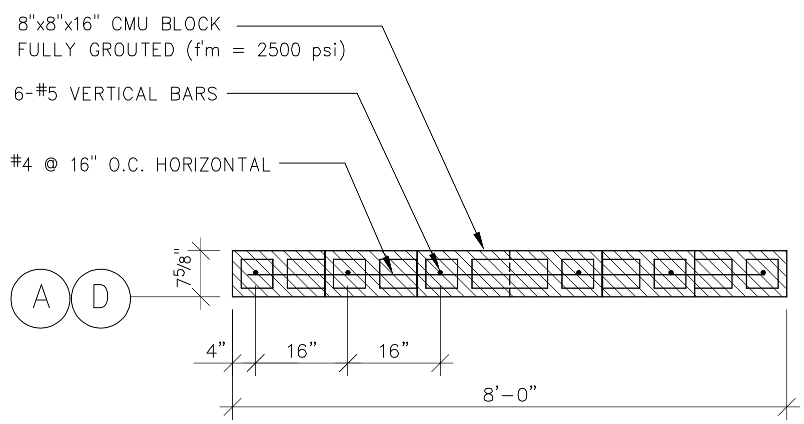

Figure 4-6. Reinforcement in 8'-0" CMU shear walls on lines A and D

Calculations and Discussion	*Code Reference*

1. Design base shear coefficient. §1630.2.2

Period using Method A (see Figure 4-5 for section through structure):

$$T = C_t (h_n)^{3/4} = .020(16 \text{ ft})^{3/4} = 0.16 \sec \qquad (30\text{-}8)$$

Near source factors for seismic source type A and distance to source = 5 km

$N_a = 1.2$ Table 16-S

$N_v = 1.6$ Table 16-T

Seismic coefficients for Zone 4 and soil profile type S_D are:

$C_a = 0.44 N_a = 0.53$ Table 16-Q

$C_v = 0.64 N_v = 1.02$ Table 16-R

The R coefficient for a masonry bearing wall building with masonry shear walls is:

$R = 4.5$ Table 16-N

Calculation of design base shear:

$$V = \frac{C_v I}{RT} W = \frac{1.02(1.0)}{4.5(0.16)} W = 1.417W \qquad (30\text{-}4)$$

but need not exceed:

$$V = \frac{2.5 C_a I}{R} W = \frac{2.5(0.53)(1.0)}{4.5} W = 0.294W \qquad (30\text{-}5)$$

The total design shear shall not be less than:

$$V = 0.11 C_a I W = 0.11(0.53)(1.0)W = 0.058 \qquad (30\text{-}6)$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than:

$$V = \frac{0.8ZN_vI}{R}W = \frac{0.8(0.4)(1.60)(1.0)}{4.5}W = 0.114W \tag{30-7}$$

Therefore, Equation (30-5) controls the base shear calculation and the seismic coefficient is thus:

$$V = \underline{\underline{0.294W}}$$

2. Base shear in transverse direction.

This building has a flexible roof diaphragm and heavy CMU walls (see Figure 4-3). The diaphragm spans as a simple beam between resisting perimeter walls in both directions and will transfer 50 percent of the diaphragm shear to each resisting wall. However, in a building that is not symmetric or does not have symmetric wall layouts, the wall lines could have slightly different wall shears on opposing wall lines 1 and 3 and also on A and D.

The building weight (mass) calculation is separated into three portions: the roof, longitudinal walls, and transverse walls for ease of application at a later stage in the calculations. The reason to separate the CMU wall masses is because masonry walls that resist ground motions parallel to their in-plane directions resist their own seismic inertia without transferring seismic forces into the roof diaphragm. This concept will be demonstrated in this example for the transverse (north-south) direction.

For the transverse direction, the roof diaphragm resists seismic inertia forces originating from the roof diaphragm and the longitudinal masonry walls (out-of-plane walls oriented east-west) on lines 1 and 3, which are oriented perpendicular to the direction of seismic ground motion. The roof diaphragm then transfers its seismic forces to the transverse masonry walls (in-plane walls oriented north-south) located on lines A and D. The transverse walls resist seismic forces transferred from the roof diaphragm and seismic forces generated from their own weight. Thus, seismic forces are generated from three sources: the roof diaphragm; in-plane walls at lines 1 and 3; and out-of-plane walls at lines A and D.

The design in the orthogonal direction is similar and the base shear is the same. However, the proportion of diaphragm and in-plane seismic forces is different. The orthogonal analysis is similar in concept, and thus is not shown in this example.

Roof weight:

$$W_{roof} = 17\,\text{psf}\,(5{,}400\,\text{sf}) = 92\,\text{kips}$$

For longitudinal wall weight (out-of-plane walls), note that the upper half of the wall weight is tributary to the roof diaphragm. This example neglects openings in the top half of the walls.

$$W_{walls,long} = 75\,\text{psf}\,(2\text{ walls})(90\,\text{ft})(19)\left(\frac{19\,\text{ft}}{2}\right)\left(\frac{1}{16\,\text{ft}}\right) = 75\,\text{psf}\,(180\,\text{ft})\frac{(19\,\text{ft})^2}{2(16\,\text{ft})} = 152\,\text{kips}$$

For forces in the transverse direction, seismic inertial forces from the transverse walls (lines A and D) do not transfer through the roof diaphragm. Therefore, the effective diaphragm weight in the north-south direction is:

$$W_{trans.diaph} = W_{roof} + W_{walls,long} = 92\,\text{k} + 152\,\text{k} = 244\,\text{kips}$$

The transverse seismic inertial force (shear force), which is generated in the roof diaphragm is calculated as follows:

$$V_{trans.diaph} = 0.294 W_{trans.diaph} = 0.294(244\,\text{kips}) = 72\,\text{kips}$$

The seismic inertial force (shear force) generated in the transverse walls (in-plane walls) is calculated using the full weight (and height) of the walls (with openings ignored for simplicity).

$$V_{trans.walls} = 0.294\,(75\,\text{psf})(19\,\text{ft})(60\,\text{ft})(2\text{ walls}) = 50\,\text{kips}$$

The design base shear in the transverse direction is the sum of the shears from the roof diaphragm shear and the masonry walls in-plane shear forces.

$$\therefore V_{trans.} = V_{trans.diaph} + V_{trans.walls} = 72\,\text{k} + 50\,\text{k} = \underline{\underline{122\,\text{kips}}}$$

3. Shear wall on line A.

The seismic shear tributary to the wall on line A comes from the roof diaphragm (transferred at the top of the wall) and the in-plane wall inertia force:

$$V_A = \frac{V_{trans.diaph}}{2} + \frac{V_{trans.walls}}{2} = \frac{72\,\text{kips}}{2} + \frac{50\,\text{kips}}{2} = \underline{\underline{61\,\text{kips}}}$$

4. Design 8'-0" shear wall on line A for out-of-plane seismic forces.

In this part, the 8'-0" shear wall on line A (Figure 4-4) will be designed for out-of-plane seismic forces. This wall is a bearing wall and must support gravity loads. It must be capable of supporting both gravity and out-of-plane seismic forces, and gravity plus in-plane seismic forces at different instants in time depending on the direction of seismic ground motion. In this Part, the first of these two analyses will be performed.

The analysis will be done using the "slender wall" design provisions of §2108.2.4. The analysis incorporates static plus $P\Delta$ deflections caused by combined gravity loads and out-of-plane seismic forces and calculates an axial plus bending capacity for the wall under the defined loading.

4a. Vertical loads.

Gravity loads from roof framing tributary to the 8'-0" shear wall at line A:

$$P_{DL} = (17\text{ psf})\left(\frac{60\text{ ft}}{2}\right)\left(\frac{30\text{ ft}}{2}\right) = 7{,}650\text{ lb}$$

Live load reduction for gravity loads:

$$R = r(A - 150) \leq 40\text{ percent}$$ §1607.5

$$A = (30\text{ ft})(15\text{ ft}) = 450\text{ sq ft}$$

$$R = 0.8(450\text{ sq ft} - 150\text{ sq ft}) = 24\text{ percent}$$

$$R_{max} = 23.1\left(1 + \frac{DL}{LL}\right) = 23.1\left(1 + \frac{17}{20}\right) = 42.7\text{ percent}$$

$$\therefore R = 24\text{ percent}$$

The reduced live load is:

$$P_{RLL} = (20\text{ psf})\left(\frac{60\text{ ft}}{2}\right)\left(\frac{30\text{ ft}}{2}\right)(100\text{ percent} - 24\text{ percent}) = 6{,}840\text{ lb}$$

Under §2106.2.7, the glulam beam reaction load may be supported by the bearing width plus four times the nominal wall thickness. Assuming a 12-inch bearing width from a beam hanger, the vertical load is assumed to be carried by a width of wall $12\text{ in.} + 4(8\text{ in.}) = 44\text{ in.}$

$$P_{beamD+L} = \frac{(7{,}650\text{ lb} + 6{,}840\text{ lb})}{(44\text{ in.}/12\text{ in.})} = 3{,}952\text{ plf}$$

$$P_{beamD} = \frac{7{,}650\text{ lb}}{(44\text{ in.}/12\text{ in.})} = 2{,}086\text{ plf}$$

Wall load on 8-foot wall (at wall mid-height):

$$P_{wall\,DL} = (75\,\text{psf})(8\,\text{ft})\left(\frac{16\,\text{ft}}{2} + 3\,\text{ft}\right) = 6{,}600\,\text{lb}$$

$$w_{wall\,DL} = \frac{6{,}600\,\text{lb}}{8\,\text{ft}} = 825\,\text{plf}$$

Dead load from wall lintels:

$$P_{Lintel\,D} = (75\,\text{psf})(9\,\text{ft})\left(\frac{20\,\text{ft}}{2}\right) = 6{,}750\,\text{lb}$$

$$l = (96\,\text{in.} - 44\,\text{in.})/2 = 26\,\text{in.}$$

$$w_{LintelD} = \frac{6{,}750\,\text{lb}}{26\,\text{in.}/12\,\text{in.}} = 3{,}115\,\text{plf}$$

Since the lintel loads are heavier than the beam load, and since dead load combinations will control, the loads over the wall/pier length will be averaged.

The gravity loads on the 8'-0" wall from the weight of the wall, the roof beam, and two lintels are:

$$\sum P_{DL} = (6{,}600\,\text{lb} + 7{,}650\,\text{lb} + 6{,}750\,\text{lb} + 6{,}750\,\text{lb}) = \underline{\underline{27{,}750\,\text{lb}}}$$

$$\sum P_{RLL} = \underline{\underline{6{,}840\,\text{lb}}}$$

4b. Seismic forces.

Out-of-plane seismic forces are calculated as the average of the wall element seismic coefficients at the base of the wall and the top of the wall. The coefficients are determined under the provisions of §1632.2 using Equation (32-2) and the limits of Equation (32-3).

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \qquad (32\text{-}2)$$

$$0.7 C_a I_p W_p \le F_p \le 4.0 C_a I_p W_p \qquad (32\text{-}3)$$

At the base of the wall:

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p$$

$$= \frac{(1.0)C_a I_p}{R_p}\left(1 + 3\frac{0\,\text{ft}}{16\,\text{ft}}\right)W_p$$

$$= 0.133 C_a I_p W_p \leq 0.7 C_a I_p W_p$$

$$\therefore \text{Use } 0.7 C_a I_p W_p$$

$$F_p = 0.7\,(.53)(1.0)W_p = 0.37 W_p$$

$$= 0.37(75\,\text{psf}) = 27.8\,\text{psf}$$

At roof:

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p$$

$$= \frac{(1.0)C_a I_p}{R_p}\left(1 + 3\frac{0'}{16'}\right)W_p$$

$$= 1.33 C_a I_p W_p \leq 4.0 C_a I_p W_p$$

$$\therefore \text{Use } 1.33 C_a I_p W_p$$

$$F_p = 1.33(.53)(1.0)W_p = 0.37 W_p$$
$$= 0.70(75\,\text{psf}) = 52.5\,\text{psf}$$

Thus, use the average value of $F_p = (1/2)(27.8\,\text{psf} + 52.5\,\text{psf}) = \underline{\underline{40.2\,\text{psf}}}$

Calculation of wall moments due to out-of-plane forces is done using the standard beam formula for a propped cantilever. See Figure 4-7 for wall out-of-plane loading diagram and Figure 4-8 for tributary widths of wall used to determine the loading diagram.

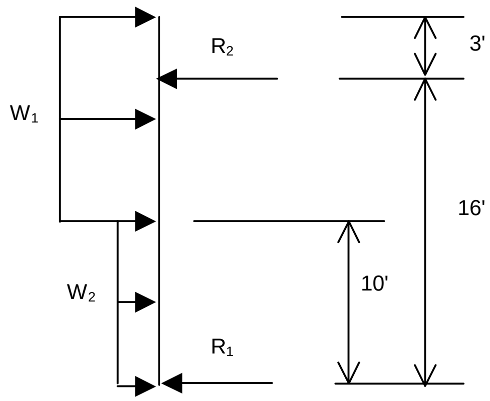

Figure 4-7. Propped cantilever loading diagram

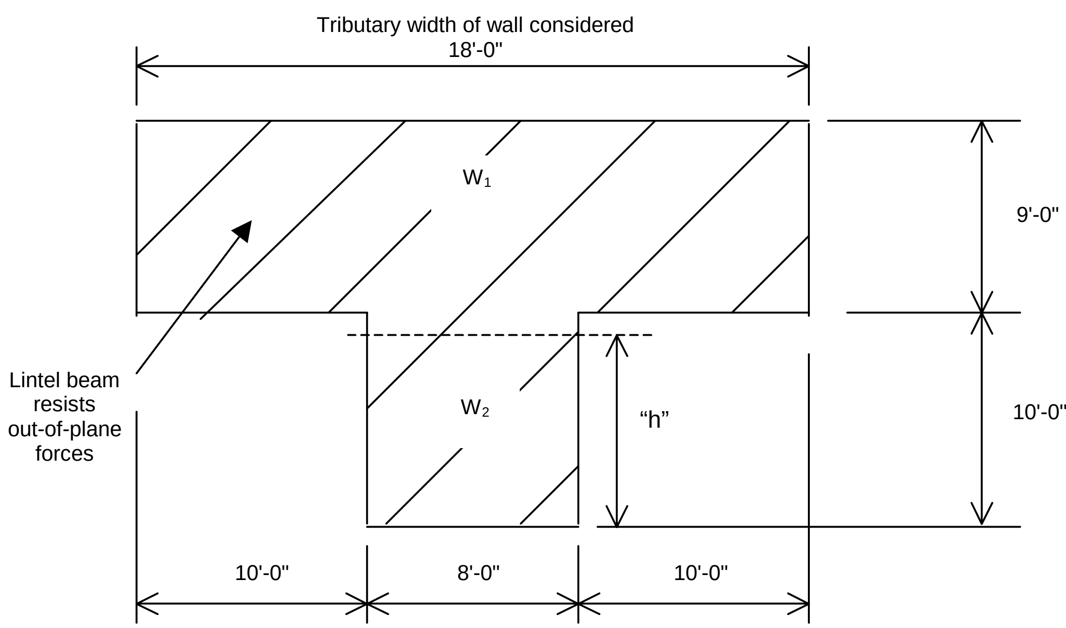

Figure 4-8. Tributary width of wall for out-of-plane seismic inertial force calculations

$$W_1 = (10\text{ ft} + 8\text{ ft} + 10\text{ ft})(40.2\text{ psf}) = 1{,}125\text{ plf}$$

$$W_2 = 8\text{ ft}(40.2\text{ psf}) = 322\text{ plf}$$

Using simple beam theory to calculate moment M_{oop} for out-of-plane forces, the location of maximum moment is at $h = 9.8$ feet:

$$M_{oop} = 15{,}530\text{ lb-ft} = 186{,}360\text{ lb-in.}$$

Comparison of seismic out-of-plane forces with wind (approximately 25 psf) indicate that seismic forces control the design.

4c. Design for out-of-plane forces. §1612.2.1

The wall section shown in Figure 4-6 will be designed. The controlling load combinations for masonry are:

$$1.2D + 1.6L_r \qquad (12\text{-}3)$$

$$1.1(1.2D + 1.0E) = 1.32D + 1.1(E_h + E_v) \qquad (12\text{-}5)$$

$$1.1E_v = 1.1(0.5)C_a ID = 0.55(0.53)(1.0)D = 0.30D$$

Note: Exception 2 of §1612.2.1 requires that a 1.1 factor be applied to the load combinations for strength design of masonry elements including seismic forces. The SEAOC Seismology Committee has recommended that this factor be deleted. However; this example shows use of the factor because it is a present requirement of the code, thus:

$$P_{D+RLL} = 1.2(27{,}750\text{ lb}) + 1.6(6840\text{ lb}) = 44{,}244\text{ lb} \qquad (12\text{-}3)$$

$$P_u = P_{D+L+E} = P_D + 1.1E_v$$

$$= 1.32(27{,}750\text{ lb}) + (0.30)(27{,}750\text{ lb}) = 44{,}955\text{ lb} \qquad (12\text{-}5)$$

The controlling load case by examination is Equation (12-5) for gravity plus seismic out-of-plane forces.

Slender wall design of masonry walls with an axial load of $0.04 f_m'$ or less are designed under the requirements of §2108.2.4.4.

Check axial load vs. $0.04 f_m'$ using unfactored loads:

$$\frac{P_w + P_f}{A_g} \le 0.04 f_m$$

$$\frac{27{,}750\,\text{lb}}{(7.625\,\text{ft})(8\,\text{ft})(12\,\text{in.})} = 38\,\text{psi} \le 0.04\,(2500\,\text{psi}) = 100\,\text{psi}$$

$\therefore$ *o.k.*

Calculate equivalent steel area A_{se}:

$$A_{se} = \frac{A_s f_y + P_u}{f_y} \qquad (8\text{-}24)$$

$$= \frac{(0.31\,\text{in.}^2)(6\,\text{bars})(60{,}000\,\text{psi}) + 44{,}955\,\text{lb}}{60{,}000\,\text{psi}} = 2.61\,\text{in.}^2$$

Calculate I_{cr}:

$$a = \frac{(P_u + A_s f_y)}{.85 f'_m\, b} = \frac{44{,}955\,\text{lb} + (1.86\,\text{in.}^2)(60{,}000\,\text{psi})}{.85(2500\,\text{psi})(96\,\text{in.})} = 0.77\,\text{in.} \qquad (8\text{-}25)$$

$$c = \frac{a}{.85} = 0.86\,\text{in.}$$

$$n = \frac{E_s}{E_m} = \frac{29{,}000{,}000\,\text{psi}}{1{,}875{,}000\,\text{psi}} = 15.46 \qquad \S 2106.2.12.1$$

$$I_{cr} = \frac{bc^3}{3} + nA_{se}(d-c)^2$$

$$= \frac{96\,\text{in.}(0.90\,\text{in.})^3}{3} + (15.46)(2.62\,\text{in.}^2)(3.81\,\text{in.} - 0.90\,\text{in})^2 = 365.0\,\text{in.}^4$$

Calculate M_{cr} using the value for f_r from §2108.2.4.6, Equation (8-31):

$$M_{cr} = S_g f_r = \left(\frac{96\,\text{in.}(7.625\,\text{in.})^2}{6}\right)(4.0)(2{,}500)^{1/2} = 186{,}050\,\text{lb - in.} \qquad (8\text{-}30)$$

Calculate I_g:

$$I_g = \frac{(96\text{ in.})(7.625\text{ in.})^3}{12} = 3546.6\text{ in.}^4$$

Calculate M_u based on Equation (8-20) of §2108.2.4.4:

First iteration for moment and deflection (note that eccentric moment at mid-height of wall is one-half of the maximum moment):

$$M_u = M_{out-of-plane} + M_{eccentric} = 1.1E + 1.1(1.2D) + 1.1(1.6)(L = 0)$$

$$M_u = M_{out-of-plane} + M_{eccentric} = 1.1(186{,}360\text{ lb - in.}) + 1.32(7{,}650\text{ lb})(6\text{ in.})/2 = 235{,}960\text{ lb - in.} \quad (8\text{-}20)$$

$$\Delta_u = \frac{5M_{cr}h^2}{48E_mI_g} + \frac{5(M_u - M_{cr})h^2}{48E_mI_{cr}} \quad (8\text{-}28)$$

$$\Delta_u = \frac{5(186{,}050\text{ lb - in.})(192\text{ in.})^2}{48(1{,}875{,}000\text{ psi})(3{,}546.6\text{ in.}^4)}$$

$$+ \frac{5(235{,}290\text{ lb - in.} - 186{,}050\text{ lb - in.})(192\text{ in.})^2}{48(1{,}875{,}000\text{ psi})(365.0\text{ in.}^4)} = 0.11\text{ in.} + 0.28\text{ in.} = 0.38\text{ in.}$$

Note: The deflection equation used is for uniform lateral loading, maximum moment at mid-height, and pinned-pinned boundary conditions. For other support and fixity conditions, moments and deflections should be calculated using established principals of mechanics. Beam deflection equations can be found in the AITC or AISC manuals or accurate methods can be derived.

Second iteration for moment and deflection:

$$M_u = 235{,}290\text{ lb - in.} + 44{,}955\text{ lb}(0.38\text{ in.}) = 252{,}540\text{ lb - in.}$$

$$\Delta_u = 0.11\text{ in.} + \frac{5(252{,}540\text{ lb - in.} - 186{,}050\text{ lb - in.})(192\text{ in.})^2}{48(1{,}875{,}000\text{ psi})(365.0\text{ in.}^4)}$$

$$= 0.11\text{ in.} + 0.37\text{ in.} = 0.48\text{ in.}$$

Third iteration for moment and deflection:

$$M_u = 235{,}290\,\text{lb - in.} + 44{,}955\,\text{lb - in.}(0.48\,\text{in.}) = 256{,}891\,\text{lb - in.}$$

$$\Delta_u = 0.11\,\text{in.} + \frac{5(256{,}891\,\text{lb - in.} - 186{,}050\,\text{lb - in.})(192\,\text{in.})^2}{48(1{,}875{,}000\,\text{psi})(365.0\,\text{in.}^4)}$$

$$= 0.11\,\text{in.} + 0.40\,\text{in.} = 0.51\,\text{in.}$$

Final moment (successive iterations are producing moments within 3 percent, therefore convergence can be determined):

$$M_u = 235{,}290\,\text{lb - in.} + 44{,}955\,\text{lb}\,(0.51\,\text{in.}) = 258{,}217\,\text{lb - in.}$$

Calculation of wall out-of-plane strength:

$$\phi M_u = \phi A_{se} f_y \left(d - \frac{a}{2}\right)$$

$$= 0.80(2.47\,\text{in.}^2)(60{,}000\,\text{psi})\left(3.81\,\text{in.} - \frac{0.73\,\text{in.}}{2}\right)$$

$$= 408{,}439\,\text{lb - in.} \geq 258{,}217\,\text{lb - in.}$$

Since the wall strength is greater than the demand, the wall section shown in Figure 4-4 is okay.

Note that out-of-plane deflections need to be checked using same iteration process, but with service loads per §2108.2.4.6, (i.e., $P_D = 27{,}750\,\text{lbs}$). Since ultimate deflections are within allowable, there is no need to check service deflections in this example. The limiting deflection is $0.007h$ per §2108.2.4.6 is $0.007(16' \times 12") = 1.34"$. The deflection from this analysis is 0.50 inches. Thus the deflection is within allowable limits.

Check that the wall reinforcement is less than 50 percent of balanced reinforcement per §2108.2.4.2:

$$\rho_b = \frac{.85\beta_1 f'_m}{f_y} + \frac{87{,}000}{87{,}000 + f_y} = 0.0178$$

$$\rho = \frac{(6)(0.31\,\text{in.}^2)}{(3.81\,\text{in.})(96\,\text{in.})} = 0.0051 \leq 0.0089$$

$\therefore$ *o.k.*

Check the unbraced parapet moment:

$a_p = 2.5$ Table 16-0

$R_p = 3.0$

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p = \frac{(2.5)(.53)(1.0)}{(3.0)}\left(1 + 3\left(\frac{16\,\text{ft}}{16\,\text{ft}}\right)\right)W_p$$

$$= 1.76W_p = 1.76(75\,\text{psf}) = 132.5\,\text{psf}$$

$$M_u (132.5\,\text{psf})(3\,\text{ft})^2 / 8 = 596\,\text{lb - ft} = 7{,}155\,\text{lb - in.} \leq 408{,}439\,\text{lb - in.}$$

$\therefore$ Wall section is okay at parapet.

5. Design 8'-0" shear wall on line A for in-plane seismic forces.

5a. Shear force distribution.

The shear force on line A must be distributed to three shear wall piers (6', 8', and 6' in width, respectively) in proportion to their relative rigidities. This can be accomplished by assuming that the walls are fixed at the tops by the 9-foot-deep lintel. Reference deflection equations are given below for CMU or concrete walls with boundary conditions fixed top or pinned top. For this Design Example, the fixed/fixed equations are used because the deep lintel at the wall/pier tops will act to fix the tops of wall piers.

$$\Delta_i = \frac{V_i h^3}{12E_m I} + \frac{1.2V_i h}{AG} \text{ for walls/piers fixed top and bottom}$$

$$\Delta_i = \frac{V_i h^3}{3E_m I} + \frac{1.2V_i h}{AG} \text{ for walls/piers pinned top and fixed at bottom}$$

$G = 0.4E_m$ for concrete masonry under §2106.2.12.13 (6-6)

Relative rigidity is thus $\frac{1}{\Delta}$ where Δ is the deflection under load V_i. Using the fixed/fixed equation, the percentage shears to each wall are shown in Table 4-1.

Table 4-1. Distribution of line A shear to three shear walls.

Wall Length (ft)	Moment Deflection (in.)	Shear Deflection (in.)	Total Deflection (in.)	Rigidity (1/in.)	Distribution to Piers (%)	Wall Shear (k)
6	1.17E-05	3.50E-07	1.20E-05	83,28	26.6%	16.2
8	6.56E-06	2.62E-07	6.82E-06	146,63	46.8%	28.6
6	1.17E-05	3.50E-07	1.20E-05	83,28	26.6%	16.2
Totals				313,20	100%	61.0

The seismic shear force E_h to the 8-foot pier is $(0.468)61\,\text{k} = 28.6\,\text{k}$.

Calculation of reliability/redundancy factor ρ is shown below. For shear walls the maximum element story shear ratio r_i is determined as: §1630.1.1

$$r_{i8} = (28.6\,\text{k})(10)/\,8\,\text{ft}\,/\,122\,\text{k} = 0.29\ \text{for}\ 8\,\text{ft segment}$$

$$r_{i6} = (16.2\,\text{k})(10)/\,6\,\text{ft}\,/\,122\,\text{k} = 0.22\ \text{for}\ 6\,\text{ft segment}$$

$$\therefore r_{max} = 0.29$$

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_B}} = 2 - \frac{20}{(0.29)\sqrt{5{,}400\,\text{ft}^2}} \qquad (30\text{-}3)$$

$$\therefore \rho = \underline{\underline{1.06}}$$

The strength design shear for the 8'-0" wall is:

$$\therefore V_{8'\,wall} = 1.06(28.6\,\text{k}) = \underline{\underline{30.3\,\text{k}}}$$

5b. Determination of shear strength.

The in-plane shear strength of the wall must be determined and compared to demand. The strength of the wall is determined as follows. Vertical reinforcement is #5@16 inches o.c. Try #4@16 inches o.c. horizontally. Note that concrete masonry cells are spaced at 8-inch centers, thus reinforcement arrangements must have spacings in increments of 8 inches (such as 8 inches, 16 inches, 24 inches, 32 inches, 40 inches, and 48 inches). Typical reinforcement spacings are 16 inches and 24 inches for horizontal and vertical reinforcement.

Calculate M/V_d :

$$\frac{M}{V_d} = \frac{151.5\,\text{k - ft}}{(30.3\,\text{k})(8\,\text{ft})} = 0.625$$

From Table 21-K and by iteration, the nominal shear strength coefficient $C_d = 1.8$

$$V_n = V_m + V_s \qquad (8\text{-}36)$$

$$V_m = C_d A_{mv} \sqrt{f_m} = (1.80)(7.625\,\text{in.})(96\,\text{in.})\sqrt{2{,}500\,\text{psi}} = 65.9\,\text{k} \qquad (8\text{-}37)$$

$$V_s = A_{mv} \rho_n f_y \qquad (8\text{-}38)$$

for $\phi = 0.80$, with #4@16" o.c. horizontally:

$$\phi V_s = \phi A_{mv} \rho_n f_y = (0.80)(7.625\,\text{in.})(96\,\text{in.})\left[\frac{(0.20\,\text{in.}^2)}{(7.625\,\text{in.})(16\,\text{in.})}\right](60{,}000\,\text{psi}) = 57.6\,\text{k}$$

for $\phi = 0.60$, with #4@16" o.c. horizontally:

$$\phi V_s = \phi A_{mv} \rho_n f_y = (0.60)(7.625\,\text{in.})(96\,\text{in.})\left[\frac{(0.20\,\text{in.}^2)}{(7.625\,\text{in.})(16\,\text{in.})}\right](60{,}000\,\text{psi}) = 43.2\,\text{k}$$

Thus, conservatively, using $\phi = 0.60$

$$\phi V_n = 0.6(65.9\,\text{k}) + 43.2\,\text{k} = 82.7\,\text{k}$$

The designer should check the failure mode. If failure mode is in bending, $\phi = 0.80.$ If failure mode is in shear, $\phi = 0.60.$ For this example, we will conservatively use $\phi = 0.60.$ The method of checking the failure mode is to check how much moment M_u is generated when the shear force is equal to shear strength V_n with $\phi = 1.0.$ Then that moment is compared with the wall P_n and M_n with a $\phi = 1.0.$ If there is reserve moment capacity, there will be a shear failure. If not, there will be a bending failure. Later in the example this will be checked.

The reason the failure mode should be checked is to understand whether a brittle shear failure will occur or a ductile bending failure. Since the bending failure is more desirable and safer, the ϕ factor is allowed to be higher.

$$V_u = 1.1(30.3\,\text{k}) = 33.3\,\text{k} \le \phi V_n = 82.7\,\text{k}, \text{ for } 0.60, \therefore\ \textit{o.k.}$$

$\therefore$ Use #4@16" horizontal reinforcement in the wall/pier.

6. **Design 8'-0" shear wall on line A for combined axial and in-plane bending actions.**

Part 5 illustrated the design of the wall for shear strength. This Part illustrates design for wall overturning moments combined with gravity loads. A free body diagram of the wall/pier is needed to understand the imposed forces on the wall.

The load combinations to be considered are specified in §1612.2.1. These are as follows (with the 1.1 factor of Exception 2 applied):

$$1.1(1.2D + 0.5L + 1.0E)(\text{floor live load}, L = 0) \quad (12\text{-}5)$$

$$1.1(0.9D - 1.0E) \quad (12\text{-}6)$$

$$E = \rho E_h + E_v \quad (30\text{-}1)$$

$$E_v = 0.5C_a ID = 0.5(0.53)(1.0)D = 0.27D \quad \S1630.1.1$$

The resulting Equation (12-5) is:

$$1.1(1.2D + 0.27D + 1.0E_h) = 1.61D - 1.1E_h$$

The resulting Equation (12-6) is:

$$1.1(0.9D + 0.27D + 1.0E_h) = 0.63D - 1.1E_h$$

$$E_h = V_{8'-0"\,wall} = 1.1(30.3\,\text{k}) = 33.3\,\text{k}$$

Axial loads P_u are calculated as P_{u1} and P_{u2} for load combinations of Equations (12-5) and (12-6):

$$P_{u1} = 1.61(27{,}750\,\text{lb}) = 44.7\,\text{kips}$$

$$P_{u2} = 0.63(27{,}750\,\text{lb}) = 17.5\,\text{kips}$$

By performing a sum of moments about the bottom corner at point A (Figure 4-9):

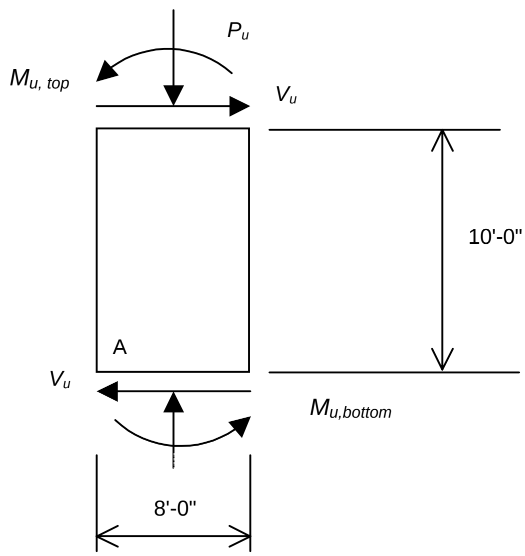

Figure 4-9. Free body diagram of 8'-0" shear wall

$$\sum M_A = 0 = 2M_u - V_u(10\text{ ft})$$

$$M_{u,top} \approx M_{u,bottom} = \frac{(33.3\text{ k})(10\text{ ft})}{2} = 166.5\text{ k - ft}$$

The reader is referred to an excellent book for the strength design of masonry *Design of Reinforced Masonry Structures,* by Brandow, Hart, Verdee, published by Concrete Masonry Association of California and Nevada, Sacramento, CA, Second Edition, 1997. This book describes the calculation of masonry wall/pier strength design in detail.

The axial load vs. bending moment capacity (P-M) diagram for the wall must be calculated. For this, the designer must understand the controlling strain levels that define yielding and ultimate strength. At yield moment, the steel strain is the yielding strain (0.00207 in./in. strain) and the masonry strain must be below 0.002 in./in. (for under-reinforced sections). At ultimate strength, the masonry has reached maximum permissible strain (0.003 in./in.) and the steel strain is considered to have gone beyond yield strain level (see§2108.2.1.2 for a list of design assumptions). See Figure 4-10 for concrete masonry stress-strain behavior. A representation of these strain states is shown in Figures 4-11 and 4-12 (the pier width is defined as h).

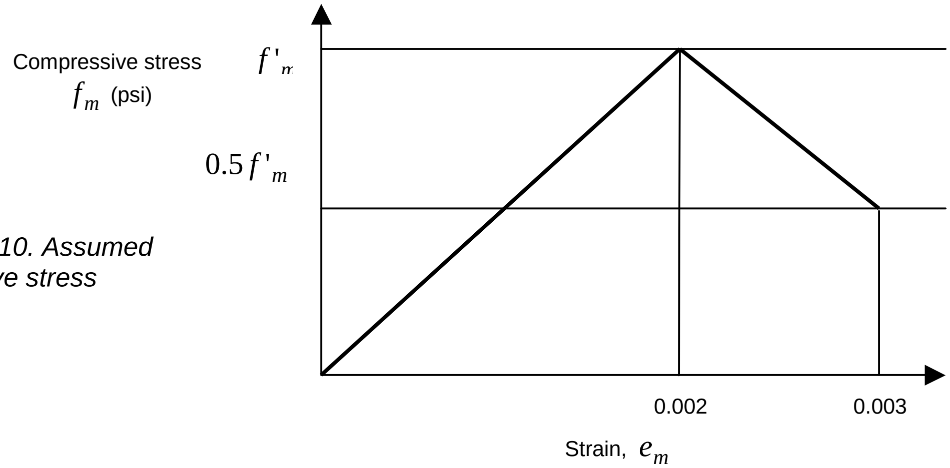

Figure 4-10. Assumed masonry compressive stress versus strain curve

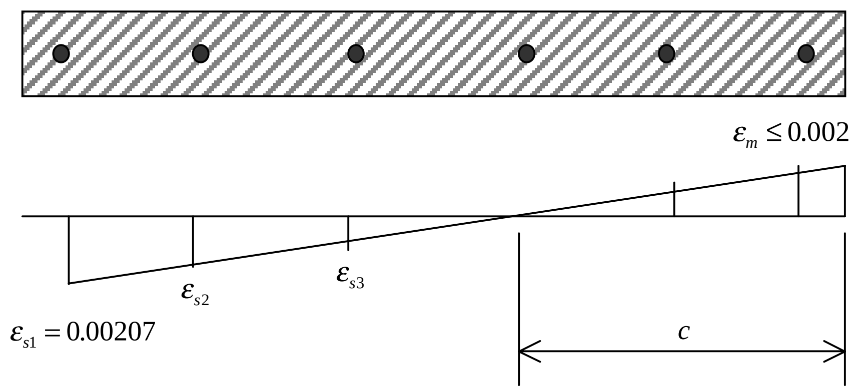

Figure 4-11. Strain diagram at yield moment; steel strain =0.00207 in./in.; masonry strain is less than yield for under-reinforced sections

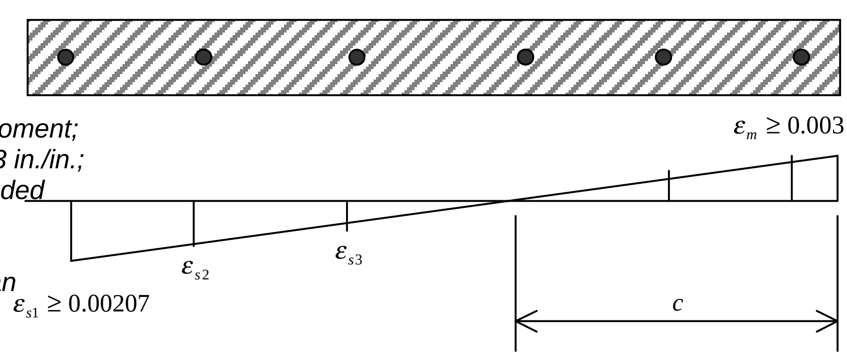

Figure 4-12. Strain diagram at ultimate moment; masonry strain =0.003 in./in.; steel strain has exceeded 0.00207 in./in.; the Whitney stress block analysis procedure can be used to simplify calculations

Note that masonry strain may continue to increase with a decrease in stress beyond strains of 0.002 in./in. at which time stresses are at f'_m. At strains of 0.003, masonry stresses are $0.5f'_m$. With boundary element confinement, masonry strains can be as large as 0.006 in./in.

By performing a summation of axial forces F, the axial load in the pier is calculated as:

$$\sum F = P = C_1 = T_1 = T_2 = T_3$$

The corresponding yield moment is calculated as follows:

$$M_y = T_1\left(d_1 - \frac{h}{2}\right) + T_2\left(d_2 - \frac{h}{2}\right) + T_3\left(d_3 - \frac{h}{2}\right) + C\left(\frac{h}{2} - \frac{c}{3}\right)$$

The ultimate moment is calculated as:

$$M_u = T_1\left(d_1 - \frac{h}{2}\right) + T_2\left(d_2 - \frac{h}{2}\right) + T_3\left(d_3 - \frac{h}{2}\right) + C\left(\frac{h}{2} - \frac{a}{2}\right)$$

Strength reduction factors, ϕ, for in-plane flexure are determined by Equation (8-1) of §2108.1.4.1.1

$$\phi = 0.80 - \frac{P_u}{(A_e f'_m)},\ 0.6 \le \phi \le 0.8 \tag{8-1}$$

Strength reduction factors for axial load, $\phi = 0.65.$ For axial loads, ϕP_n, less than $0.10 f'_m A_e$, the value of ϕ may be increased linearly to 0.85 as axial load, ϕP_n, decreases to zero.

The balanced axial load, P_b, is determined by Equations (8-2) and (8-3).

$$P_b = 0.85 f'_m\, b a_b \tag{8-2}$$

$$a_b = 0.85d\left(\frac{e_m}{e_m + \frac{f_y}{E_s}}\right) \tag{8-3}$$

$$P_b = 0.85(2{,}500)(7.625\text{ in.})(0.85)(92\text{ in.})(0.003/0.00507) = 750\text{ kips}$$

$$\phi P_b = 0.65(750\text{ kips}) = 487\text{ kips}$$

A P-M diagram can thus be developed. The P-M diagrams were calculated and plotted using a spreadsheet program. By observation, the design values P_u and $M_u (P_u = 43\text{k}, M_u = 167\text{k-ft})$ are within the nominal strength limits of ϕP_n, ϕM_n values shown in Figure 4-13. Plots for P_n vs. M_n can be seen in Figure 4-13 and for ϕP_n vs. ϕM_n in Figure 4-14.

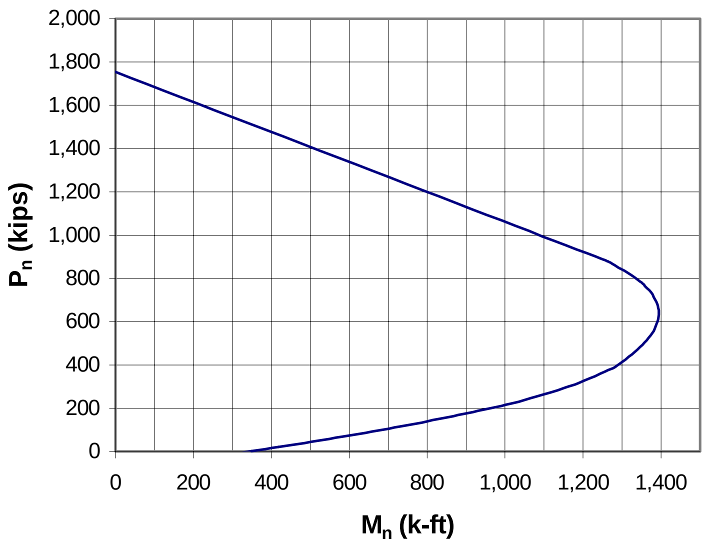

Figure 4-13. The P_n-M_n nominal strength curve with masonry strain at 0.003 in./in.

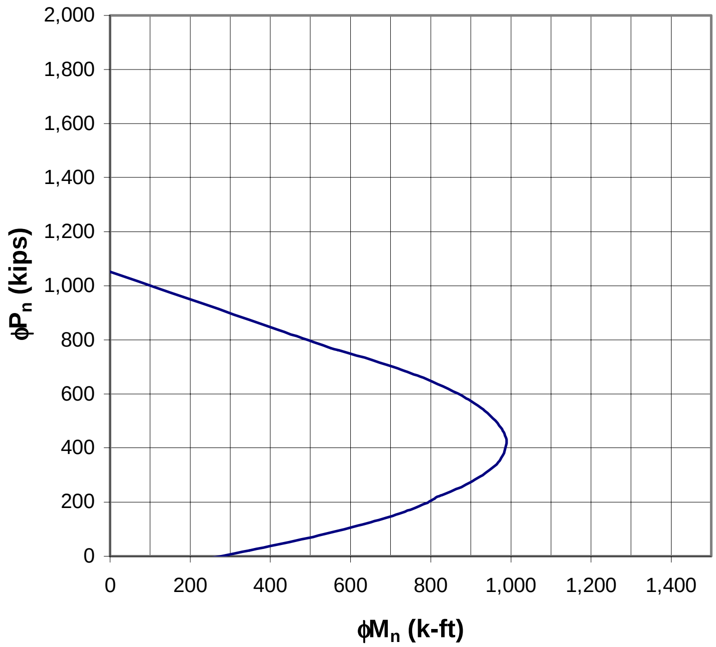

Figure 4-14. The ϕP_n-ϕM_n design strength curve with masonry strain at 0.003 in./in.

Check for type of wall failure by calculating wall moment at shear V_n:

$$M_u = \frac{V_n(10')}{2} = \frac{\left(\dfrac{82.7\,\text{k}}{0.60}\right)(10')}{2} = 689\,\text{k-ft}$$

$$P_u = 43.7\,\text{k}$$

By looking at the $P_n - M_n$ curve, this $P_u - M_u$ load is just outside the P_n, M_n curve. The shear wall failure will likely be a bending failure. However, the designer might still consider a $\phi = 0.60$ for shear design to be conservative.

7. **Deflection of shear wall on line A.** §1630.10

In this part, the deflection of the shear wall on line A will be determined. This is done to check actual deflections against the drift limits of §1630.10.

Deflections based on gross properties are computed as:

$$\Delta_s = \frac{V_i h^3}{12 E_m I} + \frac{1.2 V_i h}{AG} \quad \text{for wall/piers fixed top and bottom}$$

$$\Delta_s = \frac{(28.6\,\text{k})(120\,\text{in.})^3}{12\,(1{,}875\,\text{ksi})(562{,}176\,\text{in}^3)} + \frac{1.2\,(28.6\,\text{k})(120\,\text{in.})}{(732\,\text{in}^2)\,(750\,\text{ksi})} = 0.011\,\text{in.}$$

Assume cracked section properties and $I_{cr} = 0.3 I_g$ (approximately):

$$\Delta_s = \frac{(28.6\,\text{k})(120\,\text{in.})^3}{12\,(1{,}875\,\text{ksi})(168{,}652\,\text{in}^3)} + \frac{1.2\,(28.6\,\text{k})(120\,\text{in.})}{(732\,\text{in}^2)\,(750\,\text{ksi})} = 0.021\,\text{in.}$$

$$\Delta_m = 0.7 R \Delta_s = 0.7\,(4.5)(0.021\,\text{in.}) = 0.066\,\text{in.} \qquad (30\text{-}17)$$

Thus, deflections are less than $0.025h = 3.0$ in.

$\therefore$ *o.k.*

8. Requirements for shear wall boundary elements. §2108.2.5.6

Section §2108.2.5.6 requires boundary elements for CMU shear walls with strains exceeding 0.0015 in./in. from a wall analysis with $R = 1.5$. The intent of masonry boundary elements is to help the masonry achieve greater compressive strains (up to 0.006 in./in.) without experiencing a crushing failure.

The axial load and moment associated with this case is:

$$P_u = 44.7\,\text{kips}$$

$$M_u = \frac{4.5}{1.1} = \frac{(166.5\,\text{k - ft})}{1.1} = 619\,\text{k - ft}$$

This P-M point is not within the P-M curve using a limiting masonry strain of 0.0015 in./in. (see Figure 4-15). From an analysis it can be determined that the maximum c distance to the neutral axis is approximately 22 inches. For this example, boundary ties are required. Note that narrow shear wall performance is greatly increased with the use of boundary ties.

The code requires boundary elements to have a minimum dimension of $3\times$ wall thickness, which is 24 inches due to yield moments. After yield moment capacity is exceeded, the c distance is reduced. Thus, if boundary element ties are provided at each end of the wall/pier extending 24 inches inward, the regions experiencing strain greater than 0.0015 in./in. are confined. Space boundary ties at 8-inch centers. The purpose of masonry boundary ties is not to confine the masonry for compression, but to support the reinforcement in compression to prevent buckling. Tests have been performed to show that masonry walls can achieve 0.006 in./in. compressive strains when boundary ties are present.

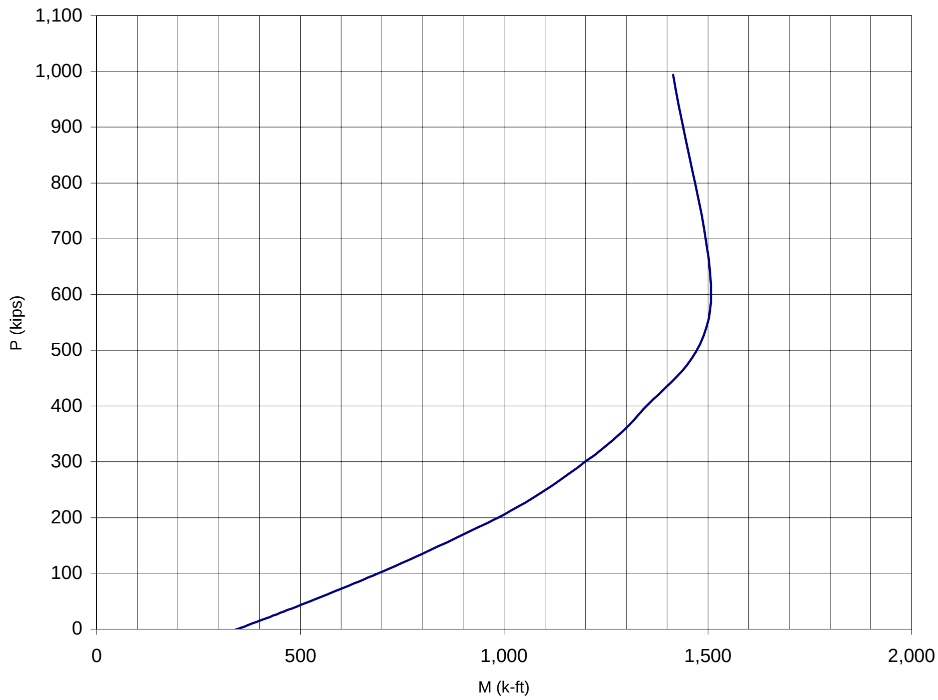

Figure 4-15. P-M curve for boundary element requirements; masonry strain is limited to 0.0015 in./in.

The P-M curve shown in Figure 4-15 is derived by setting masonry strain at the compression edge at 0.0015 in./in. and by increasing the steel tension strain at the opposite wall reinforcement bars. Moments are calculated about the center of the wall pier and axial forces are calculated about the cross-section. P-M points located at the outside of the denoted P-M boundary element curve will have masonry strains exceeding the allowable, and thus will require boundary element reinforcement or devices.

It can be seen that boundary reinforcement is required for the point $\left(P_u = 45\,\text{k},\ M_u = 619\,\text{k}\right)$. Boundary element confinement ties may consist of #3 or #4 closed reinforcement in 10-inch and 12-inch CMU walls. At 8-inch CMU walls pre-fabricated products such as the “masonry comb” are the best choice for boundary reinforcement because these walls are too narrow for reinforcement ties (even #3 and #4 bars). The boundary reinforcement should extend around three vertical #4 bars at the ends of the wall.

9. Wall-roof out-of-plane anchorage for lines 1 and 3.

CMU walls should be adequately connected to the roof diaphragm around the perimeter of the building. In earthquakes, including the 1994 Northridge event, a common failure mode has been separation of heavy walls and roofs leading to partial collapse of roofs. A recommended spacing is 8’-0" maximum. However, 6'-0" or 4'-0" might be more appropriate and should be considered for many buildings. This anchorage should also be provided on lines A and D, which will require similar but different details at the roof framing perpendicular to wall tie condition. UBC §1633.2.9 requires that diaphragm struts or ties crossing the building from chord to chord be provided that transfer the out-of-plane anchorage forces through the roof diaphragm. Diaphragm design is presented in Design Example 5, and is not presented in this example.

Per §1633.2.8.1, elements of the wall out-of-plane anchorage system shall be designed for the forces specified in §1632 where $R_p = 3.0$ and $a_p = 1.5$.

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \qquad (32\text{-}2)$$

$$F_p = \frac{1.5(.53)(1.0)}{3.0}\left(1 + 3\times\frac{16'}{16'}\right)W_p = 1.06W_p$$

or:

$f_p = 1.06w_p$, where w_p is the panel weight of 75 psf (see Figure 4-16) loading.

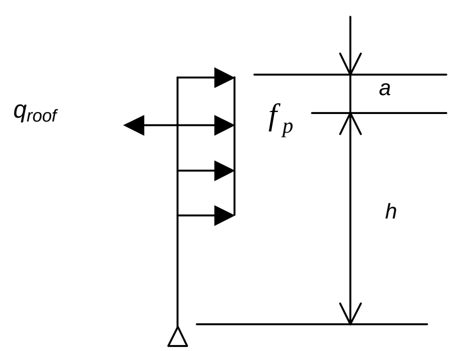

Figure 4-16. Wall-roof connection loading diagram

Calculation of the reaction at the roof level is:

$$q_{roof} = \frac{w_p (h+a)^2}{2h} = \frac{(1.06)(75\,\text{psf})(16\text{ ft} + 3\text{ ft})^2}{2(16\text{ ft})} = 897\,\text{plf}$$

Section 1633.2.8.1 requires a minimum wall-roof anchorage of $q_{roof} = 420\,\text{plf}$

$$q_{roof} = 897\,\text{plf} \geq 420\,\text{plf}$$

$\therefore$ use $q_{roof} = 897\,\text{plf}$

The design anchorage reaction at different anchor spacings is thus:

at 4'-0" centers, $q_{roof} = 3{,}588\,\text{lb}$

at 6'-0" centers, $q_{roof} = 5{,}382\,\text{lb}$

at 8'-0" centers, $q_{roof} = 7{,}175\,\text{lb}$

Therefore, choose wall-roof anchors that will develop the required force at the chosen spacing. The roof diaphragm must also be designed to resist the required force with the use of subdiaphragms (or other means). The subject of diaphragm design is discussed in Design Example 5.

For this example, a double holdown connection spaced at 8'-0" centers will be used (see Figure 4-19). This type of connection must be secured into a solid roof framing member capable of developing the anchorage force.

First check anchor capacity in concrete block of Tables 21-E-1 and 21-E-2 of Chapter 21. Alternately, the strength provisions of §2108.1.5.2 can be used.

The required tension, T, for bolt embedment is $T = E/1.4 = 7{,}175\,\text{lbs}/1.4 = 5{,}125\,\text{lb}$. For ¾-inch diameter bolts embedded 6 inches, $T = 2{,}830\,\text{lb}$ per Table 21-E-1 and 3,180 lb per Table 21-E-2. These values are for use with allowable stress design (ASD).

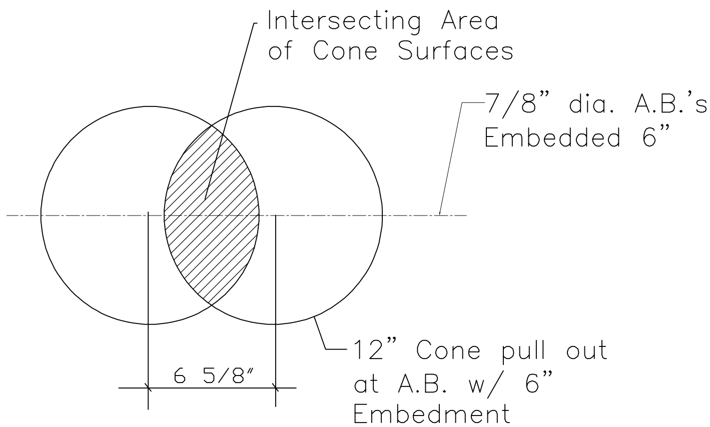

Figure 4-17. Intersection of anchor bolt tension failure cones

The anchor bolts are spaced at 6-5/8 inches center to center (considering purlin and hardware dimensions) and have 12-inch diameter pull-out failure cones. Thus, the failure surfaces will overlap (Figure 4-17). In accordance with §2108.1.5.2, the maximum tension of this bolt group may be determined as follows:

Calculate B_{tn} per bolt using the strength provisions of Equation (8-5):

$$B_{tn} = 1.04 A_p \sqrt{f'_m} = 1.04\left(113\,\text{in.}^2\right)(50\,\text{psi}) = 5{,}876\,\text{lb} \tag{8-5}$$

Calculate one-half the area of intersection of failure surfaces from two circles with radius 6 inches and centers (2-1/16" + 2½" + 2-1/16") 6 5/8" apart. $A_p = 37.8\,\text{in.}^2$ from Equations (8-7) and (8-8). Thus the bolt group tension can be calculated as:

$$(1.0)\left(2 \times 113\,\text{in.}^2 - 2 \times 37.8\,\text{in.}^2/2\right)(50\,\text{psi}) = 9{,}410\,\text{lb}$$

$$\phi B_{tn} \geq B_{tu} \therefore 0.8(9{,}410\,\text{lb}) = 7{,}528\,\text{lb} \geq 7{,}175\,\text{lb}$$

$\therefore$ *o.k.*

By choosing a pair of pre-fabricated holdown brackets with adequate capacity for a double shear connection into a 2½-inch glued-laminated framing member, the brackets are good for $2 \times 3{,}685\,\text{lb} = 7{,}307\,\text{lb}$ (ASD) $> 7{,}175\,\text{lb} \times 1.4$ steel element factor/1.4 ASD factor $= 7{,}175\,\text{lb}$. Thus, the brackets are okay.

Also check bolt adequacy in the double shear holdown connection with metal side plates (2½-inch main member, 7/8-inch bolts) per NDS Table 8.3B.
$T = 2 \times 3{,}060\,\text{lb} \times 1.33 = 8{,}140\,\text{lb} > 7{,}175\,\text{lb}$, if the failure is yielding of bolt (Mode III_s or IV failure). If the failure is in crushing of wood (Mode I_m failure), the required force is $0.85 \times 5{,}125\,\text{lb} = 4{,}356\,\text{lb}$. Therefore, the double shear bolts and pre-fabricated holdown brackets can be used.

Thus, use two holdown brackets on each side of a solid framing member connecting the masonry wall to the framing member with connections spaced at 8'-0" centers.

Verify that the CMU wall can span laterally 8'-0" between anchors. Assume a beam width of 6'-0" (3' high parapet plus an additional three feet of wall below roof) spanning horizontally between wall-roof ties.

$$w = q_{roof} = 897\,\text{plf}$$

$$M_u = \frac{wl^2}{8} = \frac{(897\,\text{plf})(8\,\text{ft})^2}{8} = 7{,}176\,\text{lb-ft}$$

The wall typically has #4@16-inch horizontal reinforcement, therefore a minimum 4-#4 bars in 6'-0" wall section.

$$a = \frac{A_s f_y}{.85 f'_m b} = \frac{4\left(.20\,\text{in.}^2\right)(60{,}000\,\text{psi})}{.85(2{,}500\,\text{psi})(72\,\text{in.})} = 0.314\,\text{in.}$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

$$\phi M_n = 0.8(4)\left(.20\,\text{in.}^2\right)(60{,}000\,\text{psi})\left(3.81\,\text{in.} - \frac{.314\,\text{in.}}{2}\right)\left(\frac{1}{12\,\text{in.}}\right) = 11{,}689\,\text{lb-ft} \le 7{,}176\,\text{lb-ft}$$

$\therefore$ *o.k.*

Per §1633.2.8.1, item 5, the wall-roof connections must be made with 2½-inch minimum net width roof framing members (2½-inch GLB members or similar) and developed into the roof diaphragm with diaphragm nailing and subdiaphragm design.

Anchor bolt embedment and edge distances are controlled by §2106.2.14.1 and §2106.2.14.2. Section 2106.2.14.1 requires that the shell of the masonry unit wall next to the wood ledger have a hole cored or drilled that allows for 1-inch grout all around the anchor bolt. Thus, for a 7/8-inch diameter anchor bolt, the core hole is 2-7/8-inch in diameter at the inside face masonry unit wall. Section 2106.2.14.2 requires that the anchor bolt end must have 1½ inches clearance to the outside face of masonry. The face shell thickness for 8-inch masonry is 1¼ inches, thus the anchor bolt end distance to the inside face of the exterior shell is 7-5/8"-1¼"-6" = 3/8". It is recommended that the minimum clear dimension is ¼-inch if fine grout is used and ½-inch if coarse pea gravel grout is used (Figure 4-18).

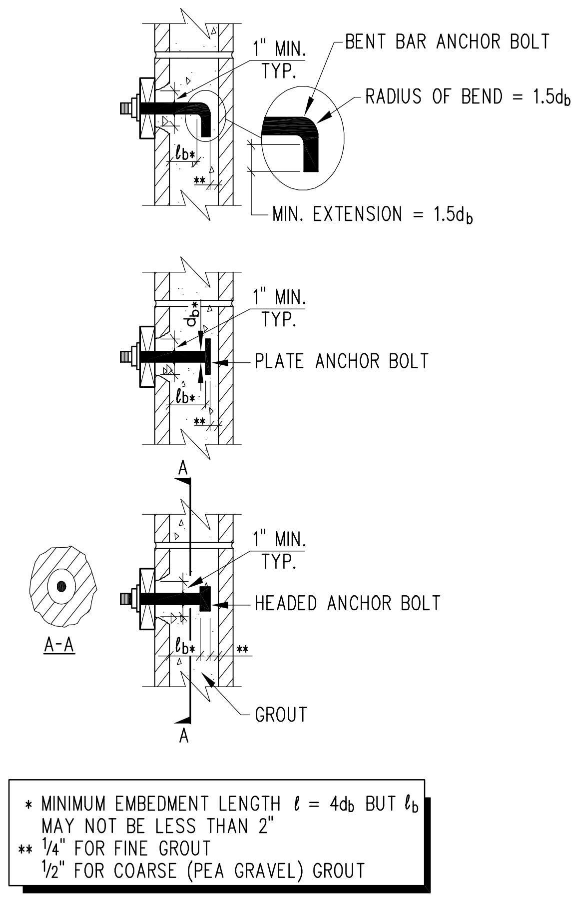

Figure 4-18. Embedment of anchor bolts in CMU walls (MIA, 1998)

10. Chord design.

Analysis of transverse roof diaphragm chords is determined by calculation of the diaphragm simple span moment $\left(wl^2/8\right)$ divided by the diaphragm depth.

$$w_{diaph,\,trans.} = \frac{(72\,\text{k} + 50\,\text{k})}{90'} = 1{,}356\,\text{plf}$$

Modify w for $R = 4.0$ by factor (4.5/4.0) = 1.125 §1633.2.9, Item 3

$$M_{diaph.} = wl^2/8 = 1.125(1{,}356\,\text{plf})(90\,\text{ft})^2/8 = 1{,}545\,\text{k - ft}$$

$$T_u = C_u = 1{,}545\,\text{k - ft}/60\,\text{ft} = 25.7\,\text{kips}$$

Using reinforcement in the CMU wall for chord forces:

$$A_{s,\,required} = \frac{T_u}{\phi f_y} = \frac{25.7\,\text{k}}{(0.80)(60\,\text{ksi})} = 0.54\,\text{in.}^2$$

Thus 2-#5 chord bars $\left(A_s = 0.62\,\text{in.}^2\right)$ are adequate to resist the chord forces. Place chord bars close to the roof diaphragm level. Since roof framing often is sloped to drainage, the chord placement is a matter of judgment.

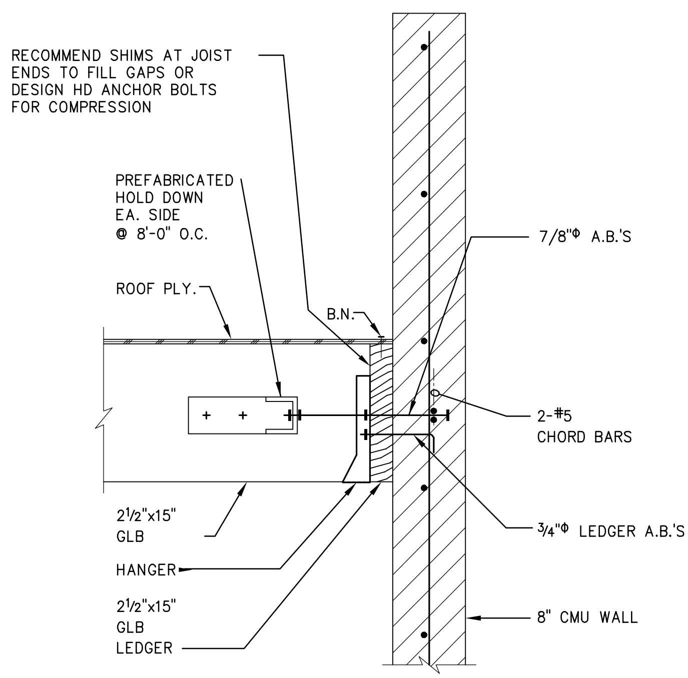

Figure 4-19. CMU wall section at wall-roof ties